Load Factor Design Applied to Truss Members in Design of Greater New Orleans Bridge No. 2

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The application of load factor design principles to the design of truss bridges is illustrated. The recommendations presented were developed during preliminary design of Greater New Orleans Bridge No. 2 and were applied during final design. Significant savings in construction cost resulted. A specification format version of these recommendations is currently before the American Association of State Highways and Transportation Officials Subcommittee on Bridges and Structures for possible adoption as a "guide specification".

The following general description of the load factor design (LFD) method as it applies to beam and girder bridges of moderate span is taken from the Highway Structures Design Handbook of U.S. Steel Corporation (1):

> Members designed by the Load Factor method are proportioned for multiples of the design loads. They are required to meet certain criteria for three theoretical load levels: 1) Maximum Design Load, 2) Overload, and 3) Service Load. The Maximum Design Load and Overload requirements are based on multiples of the service loads with certain other coefficients necessary to ensure the required capabilities of the structure. Service loads are defined as the same loads as used in working stress design.
>
> The Maximum Design Load criteria ensures the structure's capability of withstanding a few passages of exceptionally heavy vehicles (simultaneously in more than one lane), in times of extreme emergency, that may induce significant permanent deformations without failure.
>
> The Overload criteria ensures control of permanent deformations in a member, caused by occasional overweight vehicles equal to 5/3 the design live and impact loads (simultaneously in more than one lane), that would be objectionable to riding quality of the structure.
>
> The Service Load criteria ensures that the live load deflection and fatigue life (for assumed fatigue loading) of a member are controlled within acceptable limits.
>
> Moments, shears and other forces are determined by assuming elastic behavior of the structure, except for a continuous beam of compact section where negative moments over supports, determined by elastic analysis, may be reduced by a maximum of 10%. This reduction, however, must be accompanied by an increase in the maximum positive moment equal to the average decrease of the negative moments in the span.
>
> The moments, shears or forces to be sustained by a stress-carrying steel member are computed from the following formulas for the three loading levels. For Group I Loading:
>
> Service Load: D+(L+I)
>
> Overload: D+5/3(L+I)
>
> Maximum Design Load: 1.30 [D+5/3(L+I)]
>
> where: D = Dead Load
> L = Live Load
> I = Impact Load
>
> The factor 1.30 is included to compensate for uncertainties in strength, theory, loading, analysis, material properties and dimensions. The factor 5/3 is incorporated to allow for overloads.

The feature that most distinguishes LFD from service load design is the use of different multipliers on the dead and live loadings. Structural members designed by LFD will have a more uniform capacity for live load (in terms of multiples of live loads) than the same members designed by the service load method. The same is true of structures of various span lengths.

Section 1.2.22 of the American Association of State Highway and Transportation Officials (AASHTO) bridge specifications (2) states that "when long span structures are being designed by load factor design, the 'multipliers' should be increased if in the engineer's judgment, anticipated loads, service conditions or materials of construction are different than anticipated by the specification." In the case of long-span structures, for most elements of the structure the ratio of dead load to total load is greater than it is in moderate-length structures. Furthermore, the current AASHTO specifications do not fully treat the evaluation of truss member capacity. Therefore, design criteria that deal with proposed load factors and methods of computing member capacities are required before truss design by LFD can proceed.

SELECTION OF LOAD FACTORS

The formula for Group I "multipliers", or "load factors", given above for bending problems {maximum design load = 1.3 [D + 5/3(L + I)]} is shown as curve A in Figure 1, which relates factor of safety for bending and tension members to the percentage of total load--either dead load (upper scale) or live load plus impact (lower scale). The conventional factor of safety against first yield in the service load method is 1.82, and this is shown as curve B. It has not been uncommon in long-span bridge design to allow 10 percent overstress in members that carry mostly dead load. This corresponds to a factor of safety of 1.65. The transition to 10 percent allowable overstress often used by Modjeski and Masters occurs when the dead load is more than 75 percent of the total load. This is shown as curve C. The Group I load factors proposed here were developed by starting with a line that would intercept (a) the point corresponding to a factor of safety of 1.65 at 75 percent dead load and (b) the point at which the AASHTO service load and LFD methods have the same factor of safety--i.e., 40 percent dead load. With some rounding off, the proposed load factors result in

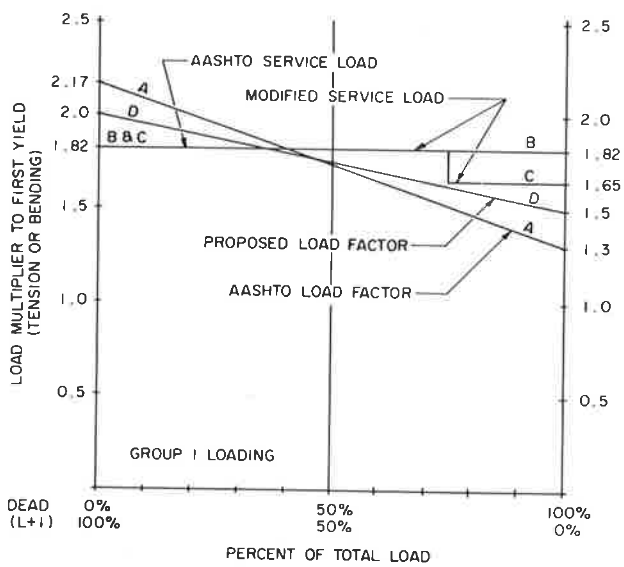
$$\text{Maximum design load} = 1.5\,[D + 4/3\,(L + I)] \leq \text{capacity} \quad (1)$$

The corresponding overload provision is

$$\text{Overload} = D + 4/3\,(L + I) \leq 80 \text{ percent of first yield capacity} \quad (2)$$

Comparison of the maximum design load and overload provisions shows that the overload provision does not control.

Figure 1. Load multiplier to first yield versus relative proportions of dead load and live load.



It is felt that the load factor relation proposed here is more appropriate for those truss members that carry high percentages of dead load--i.e., more than 75 percent. Similarly, the improbability of live load positioned on the structure so as to maximize member loads supports the somewhat lower total capacity required by the proposed method for members that carry high percentages of live load.

The proposed load factors for groups other than Group I have been selected to yield essentially the same results as service load design, as given below (case IIA is specifically intended for lateral truss members):

Group	Basic Factor of Safety/ Group Overstress Factor	Load Factor
II	1.82/1.25 = 1.46	1.46(D + W)
IIA	-	1.60W
III	1.82/1.25 = 1.46	1.46 [D + (L + I) + 0.3W + WL + LF]
XI	1.82/1.60 = 1.14	1.14(D + HW)

COMPUTATION OF MEMBER CAPACITY

Tension

The capacity of tension members is evaluated by using the two interaction equations shown below:

$$[P/(F_y)(A_n)] + [M/(S_n)(F_y)(f)] \leq 1.0 \qquad (3)$$

$$[P/(F_u)(A_{\bar{n}})] + [M/(S_{\bar{n}})(F_u)(f)] \leq 1.0 \qquad (4)$$

where

P = factored axial load;
F_y = yield point;
A_n = net area (2, Section 1.7.15);
M = factored dead load moment;
S_n = net section modulus (2, Section 1.7.15);
f = plastic shape factor computed on the basis of gross effective properties ($f = \Sigma A_y/S_g$);
A_y = statical moment of gross effective areas;
S_g = gross section modulus;
F_u = tensile strength of steel;
$A_{\bar{n}}$ = net area, all holes removed; and
$S_{\bar{n}}$ = net section modulus, all holes removed.

This interaction equation contains two simplifying assumptions. The first is that the shape of the interaction equation is a straight line joining the points ($P = P_y$, $M = 0$) and ($P = 0$, $M = M_p$). This is known to be a conservative assumption for wide-flange shapes bent about their major axis and rectangular shapes. All shapes under consideration can be considered in this range. The second assumption is that the plastic shape factor for the net section (2, Section 1.7.15), the gross effective section, and the net section with all holes removed is the same. These are reasonable assumptions, especially since the moment portion of the interaction curve is usually less than 5 percent of the total.

Compression

Two interaction equations are used, basically as discussed in Section 1.7.69(B)(1) of the AASHTO specifications (2):

$$(P/0.85\ A_{ge}\ F_{cr}) + \left(MC/M_u\{1 - [P/(A_{ge})(F_e)]\}\right) \leq 1.0 \qquad (5)$$

$$(P/0.85\ A_{ge}\ F_y) + (M/M_p) \leq 1.0 \qquad (6)$$

where

A_{ge} = gross effective area;
F_{cr} = critical load [2, Section 1.7.69(A)] with a suitable effective length factor (K);
C = equivalent moment factor taken as 0.85 or 1.00, as appropriate;
M_u = maximum bending strength, reduced for lateral buckling as indicated in the next section;
F_e = $(0.85)(\pi^2)(E)/(KL/r)^2$ in plane of bending;
M_p = $(F_y)(f)(S_{ge})$; and
S_{ge} = section modulus at end, reduced for access holes, if any.

Computation of Bending Strength

Box Members

Typical box-shaped truss members have such high lateral-torsional stiffness that the reduction in bending strength arising from lack of lateral support is minimal. The bending capacity can be computed as follows:

$$M_u = F_y S_{ge}\left\{1 - 0.0641\ [F_y S_{ge} L \sqrt{\Sigma(s/t)}/EA\sqrt{I_y}]\right\} \qquad (7)$$

where

S_{ge} = gross effective section modulus about bending axis,
L = length of member,
s/t = length of a side divided by its thickness,
A = area enclosed within center lines of plates of box members, and
I_y = moment of inertia about the nonbending axis ("vertical axis").

H-Shaped Members Bent About Axis Parallel to Flange

H-shaped sections bent about their major axis (the axis parallel to the flanges) are very susceptible to lateral torsional buckling. The elastic critical stress at which buckling is imminent is

$$\sigma_{cr} = (1/S_{ge})\sqrt{[(\pi^2 E I_y G J)/(KL)^2] + [(\pi^4 h^2 I_y^2 E)/4\,(KL)^4]} \qquad (8)$$

where

S_{ge} = gross effective section modulus about major axis;

I_y = minor-axis moment of inertia;
G = shear modulus;
J = St. Venant torsional constant, approximately $\Sigma\ bt^3/3$;
K = effective length factor for column buckling about weak axis; and
h = depth of web plate plus flange thickness.

If $\sigma_{cr} < 1/2\ F_y$, then $M_u = \sigma_{cr}\ S_{ge}$; if $\sigma_{cr} > 1/2\ F_y$, then $M_u = F_y\ S_{ge}\ [1 - (F_y/4\sigma_{cr})]$.

The expression for σ_{cr} above can also be used for modified H-shaped members composed of two channels (as flanges) and a web plate.

H-Shaped Members Bent About Axis Parallel to Web

H-shaped members bent about their minor axis do not exhibit lateral-torsional buckling, and their full plastic capacity may be used. Therefore, in this case,

$$M_u = 1.5\ F_y\ S_{ge} \tag{9}$$

Width-Thickness Ratios for Plates

Critical elastic buckling stress for plates can be written as

$$\sigma_{cr} = K\pi^2\ E/12\ (1 - \mu^2)\ (b/t)^2 \tag{10}$$

Substituting E = 29 million psi and μ = 0.3 and solving for b/t yields

$$b/t = (5120\ \sqrt{K})/\sqrt{\sigma_{cr}} \tag{11}$$

AASHTO shifts the curve defined by this equation to account for the observed behavior of plates, which indicates that residual stresses and out-of-flatness reduce the strength of plates of intermediate slenderness below that which would be indicated by simple elastic stability analysis. This shift is accomplished by multiplying the equation above by 0.6, which results in

$$b/t = (3072\ \sqrt{K})/\sqrt{\sigma_{cr}} \tag{12}$$

For the case of a simply supported plate, the minimum value of K is 4.0. This value of K and the introduction of a factor of safety that results in a working stress of 0.55 σ_{cr}/1.25 yield

$$b/t = 4073/\sqrt{\sigma_{cr}} \tag{13}$$

For main plates of truss members, the AASHTO specifications use

$$b/t = 4000/\sqrt{\sigma_{cr}} \tag{14}$$

For LFD, the maximum compressive stress is 0.85 σ_{cr}, which leads to

$$b/t = 5660/\sqrt{\sigma_{cr}} \tag{15}$$

The exact values of K, to be used for plate components of members for other conditions of support, are functions of the degree of support, which will vary from member to member. Actually, the plate strength of a fabricated member is a characteristic of the whole cross section, not of an individual plate. The existing coefficients for b/t ratios for truss members are the product of theory tempered by experience and allowances for many nonideal characteristics of plates in members. Therefore, the procedure described below has been used in developing b/t requirements.

The service load width-thickness provisions in the AASHTO specifications can be written as

$$b/t = N_{SL}/\sqrt{0.55\ \sigma_{cr}/1.25} \tag{16}$$

or

$$\sigma_{cr} = (1.25\ N_{SL}^2/0.55)(t/b)^2 \tag{17}$$

For LFD with a maximum compressive stress of 0.85 σ_{cr},

$$0.85\ \sigma_{cr} = N_{LF}^2\ (t/b)^2 = [(1.25 \times 0.85)/0.55]\ (t/b)^2\ N_{SL}^2 \tag{18}$$

Therefore,

$$N_{LF} = \sqrt{[(0.85 \times 1.25)/0.55]}\ N_{SL} \tag{19}$$

$$b/t = N_{LF}/\sqrt{\sigma_{cr}} \tag{20}$$

The resulting values are given in the second column of Table 1 along with K-values recommended by the American Institute of Steel Construction, the resulting coefficient, a recommended coefficient (N_{LF}), and a maximum b/t ratio. The recommended b/t coefficients (N_{LF}) were selected to agree, where possible, with the coefficients in the AASHTO load factor provisions for solid rib arches.

The existing AASHTO provisions for stiffened plates contained in the load factor provisions for composite box girders (adjusted for 85 percent of maximum stress) or, preferably, the load factor provisions for solid rib arches are applicable to stiffened plates in truss members.

Fatigue Design

Fatigue design proceeds exactly as in conventional service load design.

Connection Design

The load factor allowable stresses are taken from Sections 1.7.71(A) and 1.7.72(C) of the AASHTO specifications except as modified below. The overload provision, Section 1.7.72(C), will control the design of friction joints. The corresponding design capacities determined by using the proposed load factors are as follows:

Group I:	1.5 [D + 4/3(L + I)] = 1.5 [1 + R/3] [F_v] [m] [a]
Group II:	1.46 [D + W] = 1.46 F_v [m] [a]
Group IIA:	1.60W = 1.60 F_v [m] [a]
Group III:	1.46 [D + L + I + 0.3W + WL + LF] = 1.46 F_v [m] [a]
Group XI:	1.14 [D + HW] = 1.14 F_v [m] [a]

The value of F_v is obtained from Tables 1.7.41C1 and 1.7.41C2 in the AASHTO specifications, m is the number of bolts, a is the area per bolt, and R is the ratio of live load and impact force to total force. A procedure could also be developed based on allowing friction bolts to slip into bearing at factored loads.

The design of welds is based on AASHTO Section 1.7.71(2). No modification of stated design allowables is envisioned at this time.

Eyebar Pins

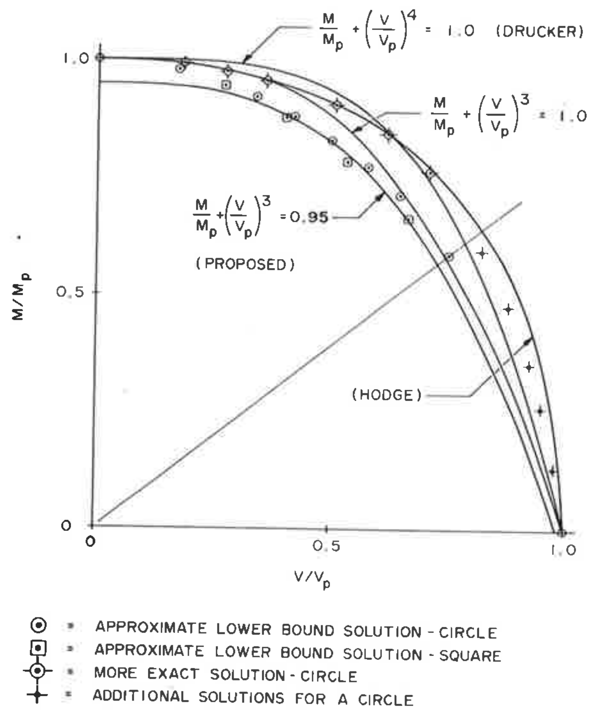
The proposed allowable bearing stress on pins not subject to rotation is 1.35 F_y. This value is based on the Ontario Highway Bridge Design Code (3), which uses a value of 1.50, in which 0 = 0.9 for steel.

Table 1. Determination of service load width-thickness ratios.

Type of Plate	Coefficient	K-Value[a]	Coefficient	N_{LF}	Max b/t Ratio
Main	5560	4	5660	5700	45
Cover	6950	5	6333	6750	50
Perforation					
Basic plate	8340	6.97	7477	8000	55
Edge of perforation	2260	0.7	2370	2200	12/16[b]

[a]Recommended by AISC. [b]Main members/secondary members.

Figure 2. Interaction curves for pins subjected to shear and moment.



The problem of pin capacity in combined shear and bending is best approached with an interaction curve. Figure 2 shows the results of three approximations to determine a suitable interaction curve. The following considerations are noted:

1. Points marked ⊙ represent lower-bound solutions obtained for a circular shape by assuming some portion of the cross section to be yielded in bending and to carry no shear. The remainder of the cross section was assumed to be elastic in bending and shear. The maximum shear stress was equal to the shear yield stress. When plotted as a normalized interaction curve, lower-bound points computed as described above plot in the same location regardless of the choice of yield criterion. The magnitude of the shear force is, of course, a function of the yield criterion.
2. Points marked ⊡ represent lower-bound solutions, obtained as described above, for a square cross section. A square section was also analyzed because available published solutions applied to rectangular cross sections. A comparison of lower-bound results for both shapes provides a basis for evaluating previously proposed interaction curves for use in the design of eyebar pins.
3. Points marked -⊙- were obtained by a computer program that analyzed a circular cross section, broken into 20 layers, by tracing the spread of plastification through the cross section corresponding to increasing, but proportional, moment and shear. The progression of yield for each of these points was from the top and bottom toward the middle in order of layer position. The von Mises yield criterion was used in these computations because it is incorporated into the existing AASHTO load factor provisions for girder design. The calculations were repeated with the Tresca yield criterion and little difference was observed. The von Mises yield criterion is $\sigma^2 + 3\tau^2 = \sigma_y{}^2$; the Tresca criterion is $\sigma^2 + 4\tau^2 = \sigma_y{}^2$.
4. Points marked + were obtained for a circular shape by using the same computer program for ratios of shear and moment, which caused the progression of plasticity to proceed either from the middle layer out to both edges in order of layer position or to start at the middle and then proceed to total plastification in an order that did not bear any relation to the order of the layer position. This implies a discontinuous strain field, a phenomenon that can exist in plastic flow. These points are regarded as informative but less reliable than the points marked -⊙- because not all implications of the discontinuous strain field on the type of bound (i.e., upper or lower) have been evaluated.
5. The radial line in Figure 2 represents the division between ratios of shear and moment for which first yield occurs in shear or bending. Above the radial line, first yield results from bending; below it, first yield results from shear.
6. Plasticity theory indicates that the failure criterion must be convex. Therefore, the shape of the interaction curve from the lowest point marked -⊙- to the horizontal axis at $V/V_p = 1.0$ is at least a straight line; i.e., it cannot curve inward.
7. Two interaction curves for rectangular cross sections published by Drucker (4) and Hodge (5) are also shown in Figure 2. The equation by Drucker is an empirical expression developed after consideration of a number of upper and lower bounds. The curve marked "Hodge" has been scaled from the paper by Hodge (5). The analytic expression was considerably more complex than Drucker's simple expression. Both curves were developed from analyses based on the Tresca yield criterion, although Hodge noted that the von Mises criterion could also have been used. Neither paper contained experimental verification.
8. An interaction curve that is similar to Drucker's but uses an exponent of 3 instead of 4 is also shown. That curve is somewhat more conservative than either Drucker's or Hodge's curve and is suggested as the basis for eyebar pin design, modified as indicated below.

The discussion above indicates that it would be reasonable to apply Drucker's interaction curve to the design of eyebar pins. However, since neither Drucker nor Hodge published experimental verification in their respective papers, and considering the importance of eyebar pins, the more conservative interaction curve marked "proposed" in Figure 2 is suggested for use in the design of Greater New Orleans Bridge No. 2.

In summary, shear and bending in eyebar pins

Table 2. Comparative chord designs.

Chord	Member No.	Member Type[a]	Type of Steel	Dead Load Percentage of Total Service Load Design	LFD Area	Without 10 Percent Overstress: Service Load Design Area	Without 10 Percent Overstress: Service Load Area/Load Factor Area	With 10 Percent Overstress: Service Load Design Area	With 10 Percent Overstress: Service Load Area/Load Factor Area
Top	NU9-NU7	T	A572	68	287.85	326.61	1.13	NA	NA
	NU7-NU5	T	A572	76	402.60	464.19	1.15	421.99	1.05
	NU5-NU4	T	A572	81	498.60	578.35	1.16	525.78	1.05
	NU4-NU2	T	A572	81	512.10	588.81	1.15	535.28	1.05
	CU2-CU4	T	A572	86	418.35	490.45	1.17	445.87	1.07
	CU4-CU5	T	A572	86	404.85	481.65	1.19	437.86	1.08
	CU5-CU7	T	A572	86	247.98	299.81	1.21	272.55	1.10
	SU3-SU5	C	A588	85	335.06	386.95	1.15	352.36	1.05
	SU5-SU7	C	A588	85	412.06	469.88	1.14	429.41	1.04
	AU2-AU4	T	A572	82	499.35	581.19	1.16	528.36	1.06
	AU4-AU5	T	A572	82	485.85	569.35	1.17	517.60	1.07
	AU5-AU7	T	A572	79	376.23	438.39	1.17	398.53	1.06
Bottom	NL8-NL6	C	A514	72	346.59[b]	357.06[b]	1.03	NA	NA
	NL6-NL4	C	A514	78	398.22	436.94	1.10	398.69	1.00
	NL4-NL2	C	A514	84	462.75	523.69	1.13	475.66	1.03
	NL2-NL0	C	A514	84	462.75	523.69	1.13	475.66	1.03
	NL0-CL2	C	A514	86	436.94	497.81	1.14	449.84	1.03
	CL2-CL4	C	A514	86	436.94	497.81	1.14	449.84	1.03
	CL4-CL6	C	A514	86	333.69[b]	357.06[b]	1.07	354.66[b]	1.06
	SL4-SL6	T	A572	85	251.85	297.45	1.18	270.41	1.07
	SL6-SL7	T	A572	85	275.85	324.58	1.18	295.07	1.07
	AL0-AL2	C	A514	84	462.75	523.69	1.13	475.66	1.03
	AL2-AL4	C	A514	84	462.75	523.69	1.13	475.66	1.03
	AL4-AL6	C	A514	80	376.38	424.03	1.13	382.81	1.02
	AL6-AL8	C	A514	76	333.69[b]	344.19[b]	1.03	333.69[b]	1.00

Note: T = tension and C = compression.

[a]If tension member, net area is given; if compression member, gross area is given.
[b]Member design limited by b/t requirements.

should be evaluated by using the following equations:

$$M_p = (D^3/6)(F_y) \quad (21)$$

$$V_p = (\pi D^2/4)[(F_y)/\sqrt{3}] \quad (22)$$

$$(M/M_p) + (V/V_p)^3 < 0.95 \quad (23)$$

CONCLUSIONS

Table 2 compares designs of 25 chord members and illustrates the savings possible with strength design. The members shown were generally controlled by strength requirements rather than fatigue or minimum plate sizes; exceptions are noted. In the latter two cases, both design methods would result in the same design. Comparison of the results in which the 10 percent overstress service load criterion was not invoked shows that the ratio of service-load-design area to LFD area ranges from 1.03 to 1.21 and averages 1.14. Inclusion of the 10 percent overstress criterion results in a range from 1.00 to 1.10 and an average of 1.05.

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Publication of this paper sponsored by Committee on Steel Bridges.