Load Factor Design for Steel Highway Bridges

W. C. HANSELL AND I. M. VIEST

In 1965 an advisory committee was formed by American Iron and Steel Institute to review current bridge design practices and to develop design recommendations for a more consistent and efficient use of steel in highway bridges. That committee, drawn from highway bridge engineers, university professors, and steel industry representatives, initiated a study that resulted in the Tentative Criteria for Load Factor Design of Steel Highway Bridges.

The Tentative Criteria were presented to the AASHO Committee on Bridges and Structures in 1968 and were published in 1969 as AISI Bulletin 15.[1] The Criteria in the bulletin were supplemented by a commentary that explained the origins of various provisions, provided supporting evidence, and cited basic references. After a year of study and certain modifications, the Criteria were adopted by the AASHO Committee on Bridges and Structures in 1970 as an alternate design method and were published in the 1971 AASHO Interim Specifications.[2]

ACKNOWLEDGMENTS

To fill the need for broad bridge design background and perspective in this study, the late George S. Vincent was retained to prepare the Tentative Criteria in co-operation with leading structural engineering authorities from four universities. The advisory committee was composed of Messrs. A. L. Collin, A. L. Elliott, R. S. Fountain, T. V. Galambos, C. A. Marmelstein, W. H. Munse and I. M. Viest (Chairman). The list of consultants included Professors J. W. Baldwin, G. C. Driscoll, J. W. Fisher, T. V. Galambos, K. H. Lenzen, A. Ostapenko and B. T. Yen.

An essential step in the development of the Load Factor Design (LFD) Criteria was to compare its design output with bridges built under past and present working stress designs. For this purpose the firm of Richardson, Gordon and Associates was retained to redesign a number of existing bridges using the Load Factor method. The results of this comparative design study are reported in AISI Bulletin 15 and are discussed later in this paper.

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Table 1. The Basic Approach of LFD

	Performance Requirement	Design Load Values	Structural Response Criteria	Design Parameters
Serviceability	Service loads	$D + (L + I)$	Fatigue, LL deflection	LL stress range, Stiffness
Serviceability	Overloads	$\alpha D + \beta(L + I)$	Permanent deformation	$\frac{\text{Max. Stress}}{F_y}$ Slip for friction joints
Safety	Maximum design loads	$\gamma[\alpha D + \beta(L + I)]$	ϕ(Max. strength)	Bending, shear and axial load capacity

BASIC APPROACH

The LFD method is outlined in Table 1. Basically, the method associates load categories with the structural response criteria for proportioning steel bridge members. The underlying philosophy is to ensure safe and serviceable performance while providing a consistent live load carrying capacity for all bridges on the highway system.

Load Factor Design recognizes three load categories: service loads, overloads, and maximum design loads.

Service loads are ordinary vehicles that may operate on a highway legally without special load permit. For design purposes, service loads are represented by the dead (D) and live plus impact ($L + I$) loads given in the AASHO Specifications.[3] The structural response criteria for service loads are concerned with fatigue — governed primarily by live load stress range, and live load deflections — governed by stiffness.

Overloads are heavy special permit vehicles that can be allowed on a structure on infrequent occasions without causing permanent damage. For design purposes, the total loading at overloads is $\alpha D + \beta(L + I)$ where α is a load factor that allows for possible increases in the dead load and β is a load factor equal to the ratio of the overload to the service live load. The structural response criteria for special permit overloads are concerned with permanent deformations, caused by yielding or by slip in friction joints, that would impair serviceability.

Maximum design loads are hypothetical vehicles that establish the required maximum strength of the structure. Maximum design loads are expressed as $\gamma[\alpha D + \beta(L + I)]$, where load factor γ provides for all sources of uncertainty with regard to the load analysis and other overall effects.

The maximum strength of each bridge element must equal or exceed forces on that element caused by the maximum design load:

$$\phi \text{ (Max. Strength)} \geqq \gamma[\alpha D + \beta(L + I)]$$

where strength factor ϕ accounts for uncertainties with regard to maximum strength.

ORGANIZATION AND SCOPE

The Tentative Criteria in AISI Bulletin 15 are organized into three major parts dealing with General Provisions, Maximum Strength, and Service Behavior, as outlined in Table 2. The provisions of Division 1 — Design in the basic AASHO Specifications[3] are used in LFD except as specifically modified by the Interim LFD Specifications. This means that items such as: general features of design, loadings, traffic lanes, reduction in load intensity, impact, and distribution of loads are unchanged in LFD. One exception is the separate LFD overload provision.

Table 2. Organization of LFD Provisions

1969 AISI Tentative Criteria Section	Topics	1971 AASHO Interim Specification Articles
1	General provisions: Scope, definition, loads, design theory, assumptions, design strength for steel, maximum design loads	1.7.118 to 1.7.124
2	Computation of maximum strength: Homogeneous or composite or hybrid beams and girders, composite box girders, compression members, splices, connections and details	1.7.125 to 1.7.136
3	Service behavior: Overload, fatigue, deflection	1.7.137 to 1.7.139

The LFD provisions can be categorized into three groups when compared to the AASHO Working Stress Design (WSD). The first group includes provisions, such as fatigue and deflections, that are virtually the same for LFD and WSD. The second group concerns WSD equations that are simply converted to a maximum strength formulation for use in LFD. The third group includes LFD provisions that stem from recent research not yet reflected in WSD but documented in the LFD Commentary.

Maximum strength equations are given for proportioning most common types of bridge members, including: homogeneous and hybrid composite and noncomposite beams and girders, composite box girders, compression members, and connection details.

The scope clause of the Interim Specifications defines Load Factor Design as "a method of proportioning structural members for multiples of the design loads." The method applies to simple and continuous beam and girder bridges of moderate length. For the present, the limitation of 500-ft spans cited in the Introduction to the AASHO Specifications[3] is a reasonable LFD guideline.

The design strength for steel is the specified minimum yield point or yield strength F_y. Within the elastic range, steel stress is assumed proportional to strain, while in the yield plateau steel stressses are taken equal to the yield strength.

For maximum strength evaluation of composite beams, the concrete stress is taken as $0.85 f_c'$ in compression and zero in tension.

Structural analysis for moments, shears, and other forces is based on elastic behavior although a limited redistribution of elastic moments is permitted under certain conditions.

LOAD FACTORS α, β

Article 1.2.2 of AASHO Specifications[3] provides directly for future increases in dead load. Therefore, $\alpha = 1.0$ was chosen for the LFD Criteria.

In agreement with earlier developed provisions for ultimate strength of concrete bridges,[4,5] the value of load factor β was chosen as $5/3$. Larger overload vehicles can usually be accommodated through elimination of concurrent traffic and restrictions on transverse positioning and speed of the overload vehicle.

For all loadings less than H20, LFD includes a provision for an infrequent heavy overload occupying a single lane without concurrent loading. In this case, $\beta = 2.2$ is specified. For an H15 bridge, this corresponds approximately to $5/3$ times an H20 truck in a single lane.

STRENGTH FACTOR ϕ

The strength factor ϕ represents several sources of uncertainty. For example, the maximum strength determination is influenced by variations in materials and section size, and by spread in test data as the result of uncontrolled and random variables. Other design uncertainties include: test conditions that differ from those in the actual structure; construction workmanship; and approximations made in strength calculations.

There is also the practical consideration of different consequences of failure for different elements of the structural system. For instance, columns (which generally are not redundant elements with alternate load paths) are regarded as requiring a relatively larger margin for structural integrity than the elements that they support. It is also accepted practice to proportion a structural system so that its strength is limited by the main members rather than by the connections between them.

The following conclusions were reached from a careful review of the sources of uncertainty with regard to strength:

First, there is no factual basis for using different ϕ values for bending and shear strength of a steel flexural member. For these principal design elements, a uniform base ϕ value is used. This is reasonable because the corresponding maximum strength equations approximate the low strength range of the test data. Second, larger variations in the strength of columns, connectors, and fasteners suggest the need for different strength factors.

The LFD Criteria do not specify separate values of ϕ and γ for bending and shear. Instead, they adopt a uniform value of γ/ϕ that applies to all design elements and additional *relative* ϕ factors that apply to columns, connectors, and fasteners.

The relative strength factors ϕ from the AASHO Interim Specifications are listed in Table 3. They depend on the type of member or fastener and in some cases on the type of load. These factors are included in the expressions for maximum strength.

Table 3. Relative Strength Factors

Type of Member or Fastener	Type of Load	Relative ϕ Factor
Column	Axial compression	0.85
Shear connector	Static shear strength	0.85
Groove weld	All	1.00
Fillet weld	Shear	0.64
Bolts and rivets	Shear (bearing-type)	0.75
Low carbon A307 bolts	Tension	0.67
High strength A325 bolts	Tension	0.75

The value $\phi = 0.85$ is applied to the maximum strength of compression members, in recognition of factors like initial bow and unknown eccentricity, and the fact that many columns are key nonredundant structural elements. The static strength of shear connectors for composite beams in both LFD and WSD is reduced by $\phi = 0.85$.

Relative ϕ factors are introduced into the design of connectors and fasteners in LFD to assure that the maximum strength of the bridge is limited by the strength of main members rather than by connection details and to account for larger experimental scatter in the strength of fasteners. Fastener strength is taken for design as the product of the relative ϕ factor and the experimental maximum strength.

With the exception of groove welds where maximum static strength depends on the capacity of the connected parts, relative ϕ factors of 0.75 or less are used for design of rivets, bolts, and welds. Based on experimental mean strength, this automatically incorporates an additional $\frac{1}{3}$ strength margin into the design of fasteners.

RATIO γ/ϕ

When the uniform ϕ for bending and shear is shifted to the load side of the LFD equation, the maximum strength is expressed as

$$\text{Max. Strength} = \frac{\gamma}{\phi}\,[\alpha D + \beta(L + I)]$$

In this form, all sources of uncertainty, other than the allowances for future increases of dead load and for overloads, are represented by the term γ/ϕ. This term, together with load factors α and β, establishes the basic margin of safety in Load Factor Design.

To incorporate the vital element of past experience into the LFD method, the margin of safety in current WSD practice was used as a guide in establishing the γ/ϕ value. The safety of the WSD approach has been proven adequate. However, the live load margin in WSD shows a substantial variation with span, implying that the level of safety for live load also varies widely.

Consider for example a braced noncomposite bridge stringer proportioned for WSD moment requirements. The allowable moment capacity $0.55F_yS$ must at least equal the service load moment represented by $D + (L + I)$ or

$$F_yS = \frac{1}{0.55}\,[D + (L + I)]$$

The WSD "safety factor" $1/0.55 = 1.82$ must account for all sources of uncertainty as to loads and strength. Taking the maximum moment capacity $M_u = F_yS$ for a braced noncompact section and deducting the dead load moment gives the maximum moment capacity available for live load. The WSD live load margin is then defined as the ratio

$$\frac{M_u - D}{L + I} = \left(\frac{1}{0.55} - 1\right)\frac{D}{L + I} + \frac{1}{0.55}$$

Based on standard plans for simple-span steel bridges prepared by the Bureau of Public Roads in 1960, the ratio $D/(L + I)$ may be approximated by:

$$\frac{D}{L + I} \simeq 0.0132 \times \text{span length in feet}$$

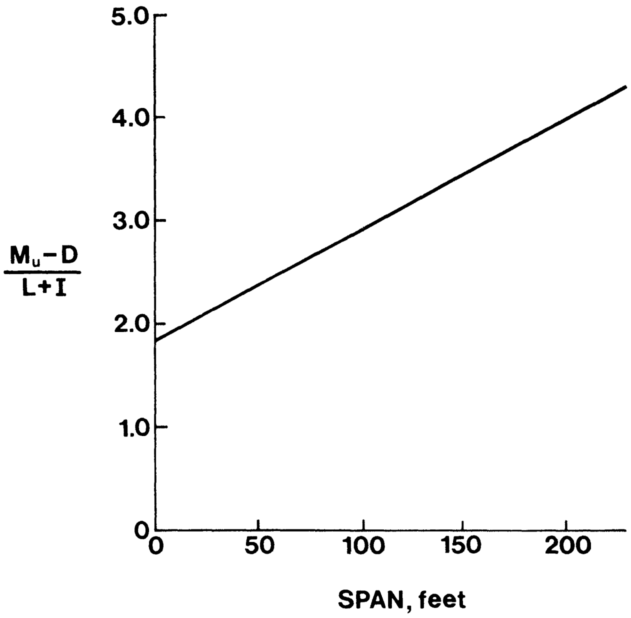


Fig. 1. Live load margin in WSD

The WSD live load margin is plotted in Fig. 1 as a function of the span length. The minimum short span value is more than doubled for long spans. It is evident that in WSD the minimum level of safety is associated with the short spans. However, as the highway system includes both short and long spans — often even in the same bridge — nothing is gained by increasing the level of safety with the span length. Thus the level of safety associated with short spans will be satisfactory also for long spans.

The objective in selecting the value of γ/ϕ for LFD was to provide *safe* and *economical* designs. The level of safety was established by selecting γ/ϕ so as to give the same braced noncompact steel section by LFD and WSD for noncomposite simple spans of about 40 ft.

The section modulus required by LFD for a noncompact noncomposite section is obtained by equating the maximum moment capacity F_yS to the factored moments at maximum loads:

$$F_yS = \frac{\gamma}{\phi}\left[\alpha D + \beta(L + I)\right]$$

If R_s is defined as the ratio of S for LFD to S for WSD, then for braced noncompact steel sections

$$R_s = 0.55\,\frac{\gamma}{\phi}\left(\frac{\alpha R + \beta}{R + 1}\right)$$

where

$$R = \frac{D}{L + I} \simeq 0.0132 \times \text{span length in feet}$$

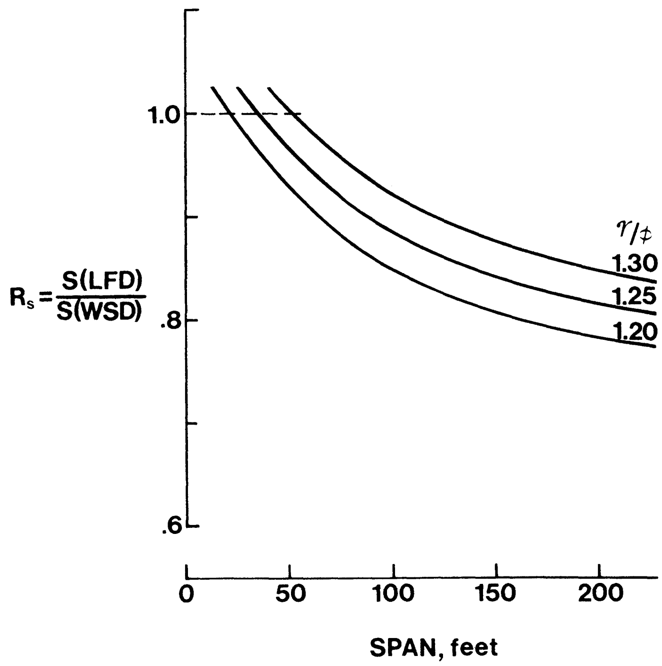


Fig. 2. LFD/WSD section modulus ratio

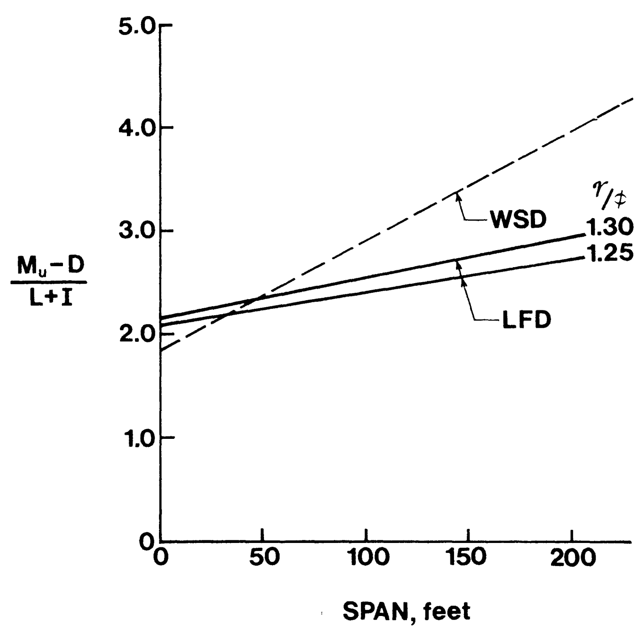


Fig. 3. Live load margin in LFD

Curves for the section modulus ratio R_s are shown in. Fig. 2 for $\alpha = 1$, $\beta = 5/3$, and selected values of γ/ϕ. When $R_s = 1$, LFD and WSD require the same steel section. This occurs for simple spans from 25 to 50 ft as γ/ϕ ranges from 1.20 to 1.30.

In the Tentative Criteria,[1] $\gamma/\phi = 1.25$ was proposed. The value adopted for the Interim Specification[2] is $\gamma/\phi = 1.30$.

Figure 2 suggests how the economic objective of LFD is achieved. As the span length increases, LFD requires a smaller section than WSD. Using noncomposite noncompact sections and $\gamma/\phi = 1.30$, the potential reduction in required section modulus is from 10 to 19 percent for spans ranging from 120 to 300 ft. The corresponding LFD live load margin, given by

$$\frac{M_u - D}{L + I} = \left(\frac{\gamma}{\phi} - 1\right)\frac{D}{L + I} + \frac{5}{3}\frac{\gamma}{\phi}$$

is more nearly constant in LFD than in WSD, as shown in Fig. 3.

It is important to note that in the preceding discussion only maximum loads were considered. To be complete, comparisons between LFD and WSD must also consider overloads and service loads. This was accomplished in the comparative design study discussed later in this paper.

LOAD COMBINATIONS

The Interim LFD Specifications[2] set forth the load combinations listed in Table 4 for determining maximum design loads. All loads in all groups are subject to the same overall load factor, 1.30. Multi-lane live loads, including the load reductions in AASHO Article 1.2.9, are used for all groups except IA.

Table 4. Load Combinations

Group	Combinations for Max. Design Load
I	$1.30\left[D + \frac{5}{3}(L + I)\right]$
IA	$1.30\ [D + 2.2\ (L + I)]$
II*	$1.30\ [D + W + F + SF + B + S + T]$
III	$1.30\ [D + L + I + CF + 0.3W + WL + F + LF]$

* Replace W with EQ, and SF with ICE, when appropriate.

Group I is the basic combination for dead, live, and impact loads with $\alpha = 1.0$ and $\beta = 5/3$.

When the loading is less than H20, the LFD approach uses Group IA loading as an additional provision for infrequent heavy loads. In this case $L + I$ represents single lane service loads.

The intent of Group II is to provide for load combinations that may affect the structure when live load is absent. Wind bracing systems may be controlled by this group. When appropriate, wind (W) is replaced by earthquake forces (EQ) and stream flow (SF) by ice loads.

Group III includes those effects that may combine with full live plus impact load.

The symbols in Table 4 represent moments, shears, or forces caused by the loads and effects defined in Article 1.2.22 of the AASHO Specifications.[3]

SERVICE BEHAVIOR

Section 3 of the Tentative Criteria[1] and Articles 1.7.137 to 139 in the AASHO Interim Specifications[2] contain the serviceability requirements for load factor design, including overloads, fatigue, and deflections. The criteria for fatigue and deflections refer to service loads, and are the same as for WSD except that the basic allowable stresses do not apply for LFD.

The overload provisions are concerned with maximum bending stresses and margins against nominal yield for noncomposite and composite members and with maximum shear on high strength bolts designed as friction connectors. These provisions guard against permanent deformations under overload that would impair serviceability.

The bridge experiments at the AASHO Road Test[6] in Illinois have demonstrated rather forcefully that, under some circumstances, permanent deformations can take place in steel beams at stresses lower than the nominal yield point of the steel. The regular test traffic intentionally caused yielding in all steel beams at test stresses approaching $0.8F_y$ to $0.9F_y$. The total accumulated permanent sets measured at the end of the test traffic are plotted in Fig. 4 for all six steel bridges that survived more than 390,000 vehicle passages. This is roughly analogous to 20 overload crossings every day for more than 50 years. Since this number of maximum over-

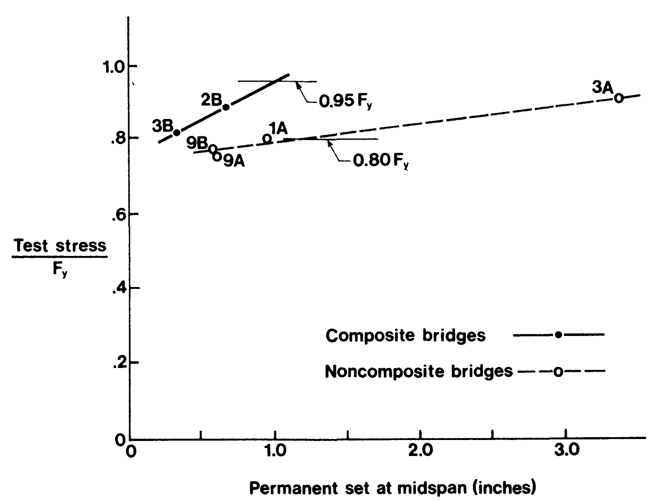
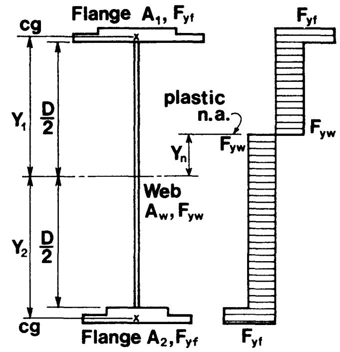


Fig. 4. Permanent set of AASHO road test bridges

Table 5. Properties of Compact Sections

PROPERTY	SYMMETRICAL SECTIONS	UNSYMMETRICAL SECTIONS
Plastic Neutral Axis	$Y_n = 0$	$\frac{Y_n}{D} = \frac{A_1 - A_2}{2A_w \rho}$
Plastic Modulus	$Z = 2A_1Y_1 + \frac{D}{4} A_w \rho$	$Z = A_1Y_1 + A_2Y_2 + \frac{D}{4} A_w \rho \left[1 - 4\left(\frac{Y_n}{D}\right)^2\right]$
Maximum Moment	$M_u = F_{yf} Z$	$M_u = F_{yf} Z$



For hybrid section

$$\rho = \frac{F_{yw}}{F_{yf}}$$

For homogeneous section

$$\rho = 1$$

loads is considerably greater than can reasonably be expected on the highway system, the permanent sets shown in Fig. 4 would not be realized in practice.

Two of these bridges were composite and four were noncomposite; each dot represents one bridge. All bridges had a 50 ft span. The difference between the magnitude of permanent set for composite and noncomposite bridges is apparent. At stresses approaching 90 percent of the yield point, the permanent set was relatively low in the composite bridge 2B, but very large in the noncomposite bridge 3A.

The WSD overload provision in AASHO Article 1.2.4 permits a 50 percent overstress, corresponding to an $0.825F_y$ maximum bending stress, under single lane overloads.

On the basis of the Road Test data, as well as theoretical considerations, the LFD Criteria specify the maximum permissible stress under overloads as $0.8F_y$ for noncomposite bridges, and $0.95F_y$ for composite bridges. The test data indicates permanent sets of comparable magnitudes at these two limiting conditions.

MAXIMUM STRENGTH

Expressions for computation of maximum strength are given in Sect. 2 of the Tentative Criteria and in Articles 1.7.125 to 1.7.136 of the Interim Specifications. Detail discussion of the formulas and limitations may be found in the Commentary to the Tentative Criteria.[1]

Bending—In general, the provisions for the design of beams and girders distinguish between three types of sections: compact, braced noncompact, and unbraced.

Compact sections can develop a fully plastic stress distribution. At maximum strength all elements of the steel section are stressed to the yield point F_y of the element in tension or compression, as indicated in Table 5. The maximum moment capacity is $M_u = F_yZ$, where the section property Z is the plastic section modulus. Values of Z for rolled shapes are listed in Ref. 7. Formulas for noncomposite unsymmetrical and hybrid sections are provided in Table 5. Within the concrete compressive stress block of a composite beam, the stress at maximum strength is taken as $0.85 f_c'$.

Braced noncompact sections are capable of developing at least the yield stress in the extreme fibers of the cross section. This is the limit assumed for WSD bending provisions. The maximum moment capacity is $M_u = F_yS$ where the section property S is the (elastic or conventional) section modulus.

For *unbraced* sections, the maximum strength is limited by lateral buckling. It depends on the ratio of the distance L_b between laterally braced points on the compression flange and the projecting width b' of the compression flange:

$$M_u = F_yS\left[1 - \frac{3F_y}{4\pi^2E}\left(\frac{L_b}{b'}\right)^2\right]$$

A 20 percent increase in M_u is permitted when the smaller end moment is less than 70 percent of the larger end moment.

The limits between the three types of sections are specified in terms of the width-thickness ratio of the compression flange, the depth-thickness ratio of the web, the requirements for lateral bracing, and maximum compression and shear forces. Furthermore, compact sections are limited to steels exhibiting a yield plateau and strain hardening.

For continuous beams using compact sections, it is permitted to take advantage of the redistribution of elastic moments to the extent of 10 percent of the maximum moment. This takes into account the demonstrated ability of compact sections to form plastic hinges.

For positive moment sections of composite beams, the slab is assumed to provide the necessary lateral bracing. The maximum strength of compact composite sections is independent of the method of construction. On the other hand, the presence or absence of temporary supports must be considered in computing the maximum strength of noncompact composite sections.

Shear—The limiting slenderness for unstiffened webs is essentially the same as in AASHO provisons for WSD. When the shear force and the web slenderness exceed certain limits, the girder web must be provided with transverse stiffeners. The shear strength of such girders is given by Basler's equation

$$V_u = V_p \left[C + \frac{0.87\ (1 - C)}{\sqrt{1 + (d_0/D)^2}} \right]$$

with the first term in the bracket representing the elastic shear buckling strength and the second term representing the post-buckling shear strength.

The upper limits on the slenderness of webs with transverse stiffeners and on webs with one longitudinal stiffener combined with transverse stiffeners are slightly higher than those currently specified for WSD. The new limits are based on extensive fatigue and static tests of plate girders. However, it is not always economical to use the maximum allowable slenderness of the web.

Compression Members—The equations for the maximum strength of compression members are those recommended by the Column Research Council,[8] except that the Interim Specifications include the reduction factor $\phi = 0.85$. The maximum strength of a concentrically loaded column is given by

$$P_u = 0.85 A_s F_{cr}$$

The combined maximum axial force P and the maximum bending moment M acting on a member must satisfy interaction equations representing a transition from the above equation for concentrically loaded columns to the pure bending strength of a beam.

Connections—The maximum strength is given in the LFD Criteria for welds, bolts, and rivets. The rules for proportioning are concerned both with individual connectors and with overall connections.

GOVERNING LFD CRITERIA

Table 6 summarizes the controlling LFD design criterion that can be anticipated for various types of members and details, based on design studies completed to date. With the exception of compact sections, maximum strength provisions usually control the designs of main members. Beams that qualify as compact are governed by the overload provision. This means that the maximum bending stress under overload should not exceed $0.8F_y$ for noncomposite beams or $0.95F_y$ for composite sections. The overload condition also controls the design of high strength bolts in friction-type connections.

Fatigue requirements under service loads govern the shear connectors for a composite beam. Service load fatigue considerations will also affect the location of cover plate cutoffs and splice design in the same manner as in WSD.

Thus, except for the factored load calculations, a similar design procedure and number of steps can be used in LFD as in WSD.

Table 6. Results of Design Studies

Type of Member or Detail	Design Criterion
Noncomposite beams in bending	
Compact	Overload ($0.8F_y$)
Noncompact	Max. strength
Composite beams in bending	
Compact	Overload ($0.95F_y$)
Noncompact	Max. strength
Webs in shear, stiffeners	Max. strength
Welded and bearing connections	Max. strength
Friction connections	Overload
Shear connectors	Fatigue

AASHO MODIFICATIONS

This section compares the Tentative Criteria published as AISI Bulletin 15[1] with the 1971 AASHO Interim Specifications.[2] The two documents differ in the extent of included material, in format, in editorial treatment, and in four substantive items.

The Interim Specifications include only the design provisions. The Preface, Commentary, and Comparative Design Study are available in AISI Bulletin 15.

The format of the Interim Specifications conforms to that of the Tenth Edition of AASHO Specifications.[3] Table 2 compares the formats of the two documents.

Editorial treatment of the Interim Specifications included principally minor changes in nomenclature, changes in wording necessitated by the changes in format, and deletion of those provisions in the Tentative Criteria covered elsewhere in AASHO Specifications.

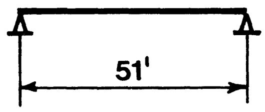
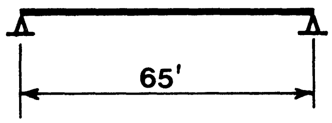
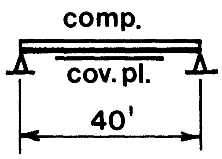
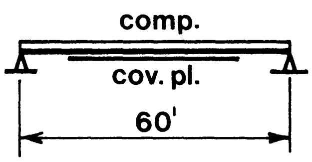
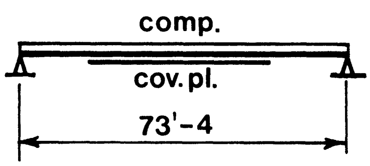
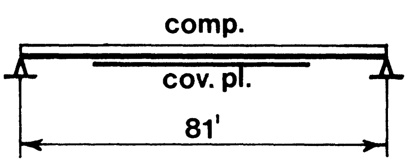
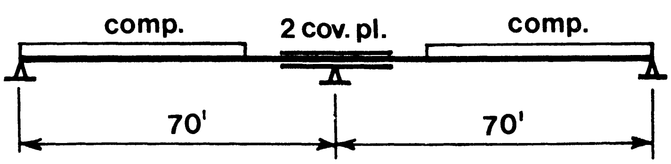
The overall factor in the load factor equation is specified as 1.30 in the Interim Specifications (Articles 1.7.124 and 1.7.128), while the Tentative Criteria include the value of 1.25 (Sects. 1.7 and 2.2.2).

For compression, the Interim Specifications specify a reduction factor of 0.85 (Article 1.7.135), while the Tentative Criteria include no reduction (Eqs. 2.4.1-1 and 2.4.2-1).

In keeping with current working stress provisions, the Interim Specifications do not include A490 high strength bolts, while they are included in the Tentative Criteria (Sect. 2.5).

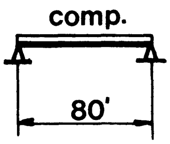
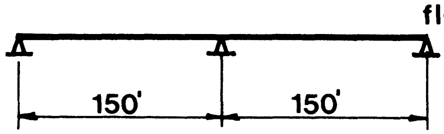
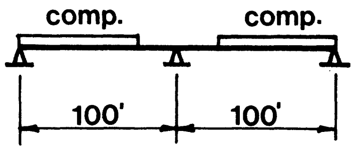
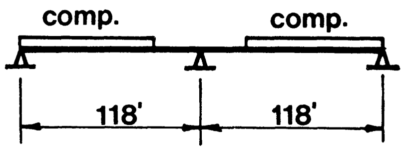
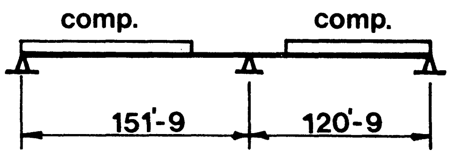
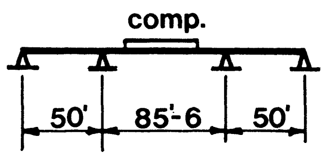
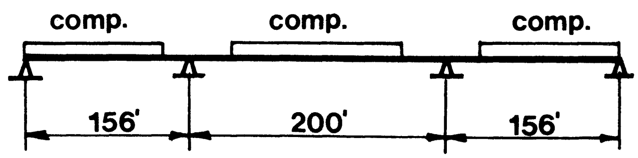
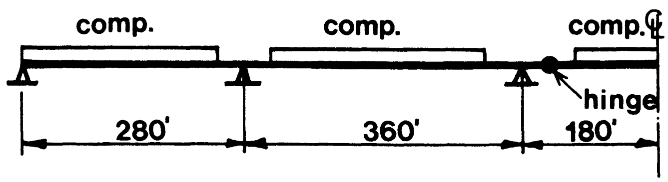
Finally, the Interim Specifications are a supplement to the Tenth Edition of the AASHO Specifications, while the Tentative Criteria, developed before 1969, refer to the Ninth Edition.

Table 7. Rolled Beam Comparative Designs

DESIGN NO.	DESCRIPTION	LOADING	STRINGER SPACING	STEEL GRADES	PERCENT MATERIAL SAVING
2		H15	7'-4	A36	3.7
3		H15	7'-4	A36	6.2 int. 0 ext.
1		HS20	8'-4	A36	14.7
6		H15	7'-4	A36	21.5
13		HS20	7'-7$\frac{1}{2}$	A36	13.8
4		H15	8'-0	A36	22.1 int. 14.2 ext.
7		HS20	8'-4	A36	5.0

Note: int. and ext. indicate interior and exterior stringer

Table 8. Welded Plate Girder Comparative Designs

DESIGN NO.	DESCRIPTION	LOADING	STRINGER SPACING	STEEL GRADES	PERCENT MATERIAL SAVING
5		HS20	8'-4	A36	7.0
9	 floor beam, stringer girder	HS20 truck HS15 lane	23'-0	A441	10.6
10		HS20	8'-4	A36	9.4
14		HS20	9'-3	A36	13.0
8		HS20	7'-0	A36 A441	14.4
15		HS20	7'-5	A36	5.5
11		HS20	8'-4	A36	13.5
12		HS20	20'-0	A36 A441 A514	11.7

COMPARATIVE DESIGN STUDY

In 1968, during the final stages of the development of the Tentative Criteria, a study was made to compare bridges designed according to the Tentative Criteria for Load Factor Design with bridges designed using the 1965 AASHO *Standard Specifications for Highway Bridges*. The comparative study involved 15 representative bridges of the beam and girder type. The original design to AASHO WSD requirements was available for each bridge.

The 15 structures included both simple and continuous designs with span lengths varying from 40 to 360 ft. The longest structure was a five-span continuous composite hybrid plate girder bridge with hinges in the center span. The bridges were designed for H15, HS15, or HS20 loadings using A36, A441, and A514 steels. Both rolled beam and welded plate girder designs were included, as summarized in Tables 7 and 8. A majority of the designs utilized composite action in positive moment regions, although three bridges were entirely noncomposite.

The comparative design study used the factors

$$\begin{aligned} \alpha &= 1.0 \text{ on dead load} \\ \beta &= 5/3 \text{ or } 2.2 \text{ on live load} \\ \gamma/\phi &= 1.25 \text{ on } [\alpha D + \beta(L + I)] \end{aligned}$$

as provided by the Tentative Criteria. Based on these load factors, the comparison between LFD and WSD main material requirements yielded the percent material savings shown in the right-hand column of Tables 7 and 8. Material requirements considered the weight of steel for one stringer including stiffeners and cover plates, but excluding diaphragms, bracing, and other details. To provide realistic comparisons, all details of the original WSD bridges were retained to the maximum possible extent.

Tables 7 and 8 illustrate a reasonably consistent trend of increasing material savings with increasing span. One exception is Design No. 7 in Table 7. This two-span continuous rolled beam design was controlled by fatigue requirements at the ends of the negative moment cover plates. This was the only case in which the LFD service load criteria governed the basic steel section.

Reduced material requirements are one source of economy in the LFD approach. In some cases, fabrication operations can also be reduced. For example, the LFD for Design No. 12, a major highway structure, reduced the number of intermediate transverse stiffeners by about one-third. In another case, two positive moment cover plates were eliminated for LFD Design No. 7.

The bridges in the comparative design study represent four groups denoted by:

RC for rolled W shape, composite
RN for rolled W shape, noncomposite
GC for welded girder, composite
GN for welded girder, noncomposite

To better illustrate major trends, the ratio of steel weight for the load factor designs to steel weight for the AASHO working stress designs can be plotted as a function of the $D/(L + I)$ ratio for each bridge. Since continuous spans have different $D/(L + I)$ ratios for positive and negative moment, weighted ratios are plotted.

The seven bridges using rolled W shapes are compared in Fig. 5. All of these designs were compact for positive moment. The governing LFD Criterion was the maximum bending stress under overload:

for the RN group,

$$0.8F_y \geq \frac{D + \beta(L + I)}{S}$$

and for the RC unshored composite group,

$$0.95F_y \geq \frac{D_s}{S_s} + \frac{D_c}{S_{cd}} + \frac{\beta(L + I)}{S_{cl}}$$

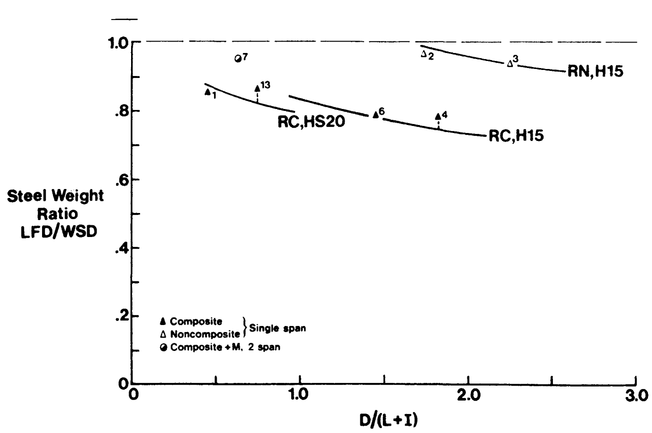


Fig. 5. Rolled beam comparative designs

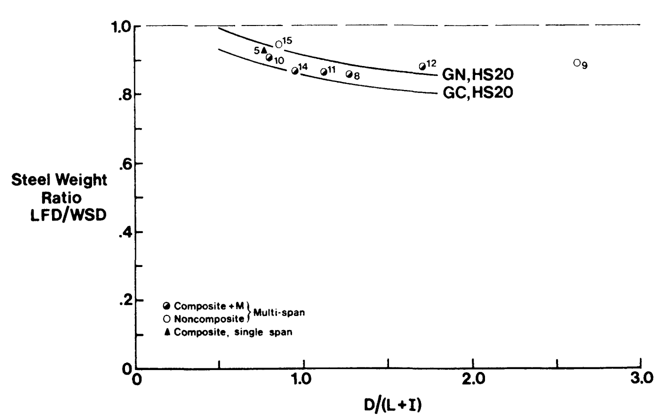


Fig. 6. Welded girder comparative designs

The six simple span comparative designs in Fig. 5 are grouped around the curves labeled RN and RC, derived from these LFD overload requirements.

The eight welded girder bridges are compared in Fig. 6. All of these structures were noncompact and the maximum strength at first yield governed their design. For the positive moment composite sections, designed as unshored, the group GC design criterion was:

$$F_y \geq 1.25 \left[\frac{D_s}{S_s} + \frac{D_c}{S_{cd}} + \frac{\beta(L+I)}{S_{cl}} \right]$$

For the noncomposite sections, the group GN designs satisfied the condition

$$F_y S \geq 1.25 \; [D + \beta(L+I)]$$

with negative moment strength reductions for unbraced length of the compression flange or for shear-moment interaction applied where necessary.

All of the comparative designs for HS20 live load in Fig. 6 are grouped around the GC and GN curves, derived from these LFD maximum strength criteria. The composite GC and noncomposite GN curves bracket all but one of the designs that were composite in positive moment regions and noncomposite for negative moment. The exception was Design No. 12 where LFD material requirements were influenced by shear-moment interaction and by the WSD bracing spacing which was not modified. The approach used for shear-moment interaction was to combine maximum shear for one live load position with maximum moment for a second live load position. This approach is conservative whenever maximum shear and maximum moment are derived from different live load patterns.

The overall load factor γ/ϕ was changed from 1.25 in the Tentative Criteria to 1.30 in the AASHO Interim Specifications. The effect of this change in load factors is to increase maximum strength requirements for the noncompact welded girder designs in Fig. 6 by a factor of $1.30/1.25 = 1.04$.

The compact rolled beam designs in Fig. 5 are not affected by this load factor modification. The LFD overload requirements that governed these designs are included without modification in the Interim AASHO Specifications.

Conclusions—Several conclusions were reached from this comparative design study:

First, the trends in Figs. 5 and 6 verify a basic LFD premise: the same noncomposite steel section requirements for WSD and LFD in the short span bridge range. This means that the level of safety for LFD bridges is consistent with the minimum safety inherent in current highway bridge designs.

Second, the LFD/WSD steel weight ratio decreases with increasing span and dead to live load ratio. Main member steel requirements were reduced by 10 percent or more for spans exceeding 100 ft.

Third, the LFD approach gives designs that utilize more of the inherent strength of composite construction than is permitted by current WSD procedures. The level of safety for the composite LFD bridges is consistent with that provided by noncomposite bridge members.

Fourth, fatigue requirements under service loads controlled the design of shear connectors and the location of cover plate cutoffs and some splices. Otherwise, fatigue requirements rarely influenced main member proportions in the LFD bridges.

Fifth, all welded girder bridges were noncompact. Their design was based on the maximum strength at first yield. All rolled beam designs were compact for positive moment. In this case, design was based on a margin against yielding under overloads.

Finally, the results of the comparative design study suggest that load factor design offers the potential of reducing the weight of steel beams and girders by up to 15 percent, with no increase in fabrication labor and no decrease in the load-carrying capabilities of the total highway system.

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