SECTION 11
DESIGN CRITERIA FOR BRIDGES

PART 1

APPLICATION OF CRITERIA FOR COST-EFFECTIVE HIGHWAY BRIDGE DESIGN

Robert L. Nickerson,* P.E.
President, NBE, Ltd.,
Hampstead, Maryland

Dennis Mertz,* P.E.
Assoc. Professor of Civil Engineering,
University of Delaware,
Newark, Delaware

The purpose of this section is to provide guidance to highway bridge designers for application of standard design specifications to the more common types of bridges and to provide rules of thumb to assist in obtaining cost-effective and safe structures. Because of the complexity of modern specifications for bridge design and construction and the large number of standards and guides with which designers must be familiar to ensure adequate designs, this section does not provide comprehensive treatment of all types of bridges. Because specifications are continually being revised, readers are cautioned to use the latest edition, including interims, in practical applications.

11.1 STANDARD SPECIFICATIONS

Designs for most highway bridges in the United States are governed by the ''Standard Specifications for Highway Bridges'' or the ''LRFD Bridge Design Specifications'' of the American Association of State Highway and Transportation Officials (AASHTO), 444 N. Capitol St., NW, Washington, DC 20001. AASHTO updates these specifications annually. Necessary revisions are published as ''Interim Specifications.'' A new edition of the Standard Specifications has been published about every fourth year, incorporating intervening ''Interim Spec-

*Revised Sec. 10, originally written by Frank D. Sears, Bridge Division, Federal Highway Administration, Washington, D.C. Material on ASD and LFD design was updated by Roger L. Brockenbrough.

ifications.'' The design criteria for highway bridges in this section are based on the 16th (1996) edition of the Standard Specifications, with 1997 and 1998 Interims, and the 2nd (1998) edition of the LRFD Specifications. Current plans of AASHTO are to discontinue maintenance of the Standard Specifications and to emphasize the LRFD Specifications. A complete design example for a two-span continuous I-girder bridge is included as an Appendix to this section to illustrate application of the LRFD Specifications.

For complex design-related items or modifications involving new technology, AASHTO issues tentative ''Guide Specifications,'' to allow further assessment and refinement of the new criteria. AASHTO may adopt a ''Guide Specification,'' after a trial period of use, as part of the Standard Specifications.

State highway departments usually adopt the AASHTO bridge specifications as their minimum standards for highway bridge design. Because conditions vary from state to state, however, many bridge owners modify the standard specifications to meet specific needs. For example, California has specific requirements for earthquake resistance that may not be appropriate for many east-coast structures.

To ensure safe, cost-effective, and durable structures, designers should meet the requirements of the latest specifications and guides available. For unusual types of structures, or for bridges with spans longer than about 500 ft, designers should make a more detailed application of theory and performance than is possible with standard criteria or the practices described in this section. Use of much of the standard specifications, however, is appropriate for unusual structures, inasmuch as these generally are composed of components to which the specifications are applicable.

11.2 DESIGN METHODS

AASHTO ''Standard Specifications for Highway Bridges'' present two design methods for steel bridges: service-load, or allowable-stress, design (ASD) and strength, or load-factor, design (LFD). Both are being replaced by load-and-resistance-factor design (LRFD). The LRFD Specifications utilize factors based on the theory of reliability and statistical knowledge of load and material characteristics. (See also Sec. 6.) It identifies methods of modeling and analysis. It incorporates many of the existing AASHTO ''Guide Specifications.'' Also, it includes features that are equally applicable to ASD and LFD that are not in the Standard Specifications. For example, the LRFD specifications include serviceability requirements for durability of bridge materials, inspectability of bridge components, maintenance that includes deck-replacement considerations in adverse environments, constructability, ridability, economy, and esthetics. Although procedures for ASD are presented in many of the following articles, LFD or LRFD may often yield more economical results. A structure designed by LRFD methods will be better proportioned, with all parts of the structure theoretically designed for the same degree of reliability.

Curved girders are not fully covered by the LRFD Specifications, and were not a part of the calibration data base. The LRFD Specification does allow girders with slight curvatures to be designed as if they are straight. Specifically, it is permitted for ''torsionally stiff closed sections whose central angle subtended by a curved span . . . is less than 12.0°.'' and for ''open sections whose radius is such that the central angle subtended by each span is less than the value given in'' Table 11.1. For the design of bridges with greater curvatures, refer to the AASHTO ''*Guide Specifications for Horizontally Curved Highway Bridges,*'' including the latest Interim Specifications. Also see Arts. 12.6 and 12.7. Current research may substantially modify these criteria in the future.

11.3 PRIMARY DESIGN CONSIDERATIONS

The primary purpose of a highway bridge is to safely carry (geometrically and structurally) the necessary traffic volumes and loads. Normally, traffic volumes, present and future, de-

TABLE 11.1 Maximum Central Angle for Neglecting Curvature in Determining Primary Bending Moments

Number of beams	Angle for one span	Angle for two or more spans
2	2°	3°
3 or 4	3°	4°
5 or more	4°	5°

termine the number and width of traffic lanes, establish the need for, and width of, shoulders, and the minimum design truck weight. These requirements are usually established by the owner's planning and highway design section using the roadway design criteria contained in "A Policy on Geometric Design of Highways and Streets," American Association of State Highway and Transportation Officials. If lane widths, shoulders, and other pertinent dimensions are not established by the owner, this AASHTO Policy should be used for guidance. Ideally, bridge designers will be part of the highway design team to ensure that unduly complex bridge geometric requirements, or excessive bridge lengths are not generated during the highway-location approval process.

Traffic considerations for bridges are not necessarily limited to overland vehicles. In many cases, ships and construction equipment must be considered. Requirements for safe passage of extraordinary traffic over *and* under the structure may impose additional restrictions on the design that could be quite severe.

Past AASHTO "Standard Specifications for Highway Bridges" did not contain requirements for a specified design service life for bridges. It has been assumed that, if the design provisions are followed, proper materials are specified, a quality assurance procedure is in place during construction, and adequate maintenance is performed, an acceptable service life will be achieved. An examination of the existing inventory of steel bridges throughout the United States indicates this to be generally true, although there are examples where service life is not acceptable. The predominant causes for reduced service life are geometric deficiencies because of increases in traffic that exceed the original design-traffic capacity. The LRFD specification addresses service life by requiring design and material considerations that will achieve a 75-year design life.

11.3.1 Deflection Limitations

In general, highway bridges consisting of simple or continuous spans should be designed so that deflection due to live load plus impact should not exceed $1/800$ the span. For bridges available to pedestrians in urban areas, this deflection should be limited to $1/1000$ the span. For cantilevers, the deflection should generally not exceed $1/300$ the cantilever arm, or $1/375$ where pedestrian traffic may be carried. (See also Art. 11.21.) In LRFD, these limits are optional.

Live-load deflection computations for beams and girders should be based on gross moment of inertia of cross section, or of transformed section for composite girders. For a truss, deflection computations should be based on gross area of each member, except for sections with perforated cover plates. For such sections, the effective area (net volume divided by length center to center of perforations) should be used.

11.3.2 Stringers and Floorbeams

Stringers are beams generally placed parallel to the longitudinal axis of the bridge, or direction of traffic, in highway bridges, such as truss bridges. Usually. they should be framed into floorbeams. But if they are supported on the top flanges of the floorbeams, it is desirable that the stringers he continuous over two or more panels. In bridges with wood floors, intermediate cross frames or diaphragms should be placed between stringers more than 20 ft long.

In skew bridges without end floorbeams, the stringers, at the end bearings, should be held in correct position by end struts also connected to the main trusses or girders. Lateral bracing in the end panels should be connected to the end struts and main trusses or girders.

Floorbeams preferably should be perpendicular to main trusses or girders. Also, connections to those members should be positioned to permit attachment of lateral bracing, if required, to both floorbeam and main truss or girder.

Main material of floorbeam hangers should not be coped or notched. Built-up hangers should have solid or perforated web plates or lacing.

11.4 HIGHWAY DESIGN LOADINGS

The AASHTO "Standard Specifications for Highway Bridges" require bridges to be designed to carry dead and live loads and impact, or the dynamic effect of the live load. Structures should also be capable of sustaining other loads to which they may be subjected, such as longitudinal, centrifugal, thermal, seismic, and erection forces. Various combinations of these loads must be considered as designated in groups I through X. (See Art. 11.5.1.)

The LRFD Specification separates loads into two categories: permanent and transient. The following are the loads to be considered and their designation (load combinations are discussed in Art. 11.5.4):

Permanent Loads

DD = downdrag
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = accumulated locked-in force effects resulting from construction
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill

Transient Loads

BR = vehicular braking force
CE = vehicular centrifugal force
CR = creep
CT = vehicular collision force
CV = vessel collision force
EQ = earthquake
FR = friction
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load

LS = live load surcharge
PL = pedestrian live load
SE = settlement
SH = shrinkage
TG = temperature gradient
TU = uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

Certain loads applicable to the design of superstructures of steel beam/girder-slab bridges are discussed in detail below.

Dead Loads. Designers should use the actual dead weights of materials specified for the structure. For the more commonly used materials, the AASHTO Specifications provide the weights to be used. For other materials, designers must determine the proper design loads. It is important that the dead loads used in design be noted on the contract plans for analysis purposes during possible future rehabilitations.

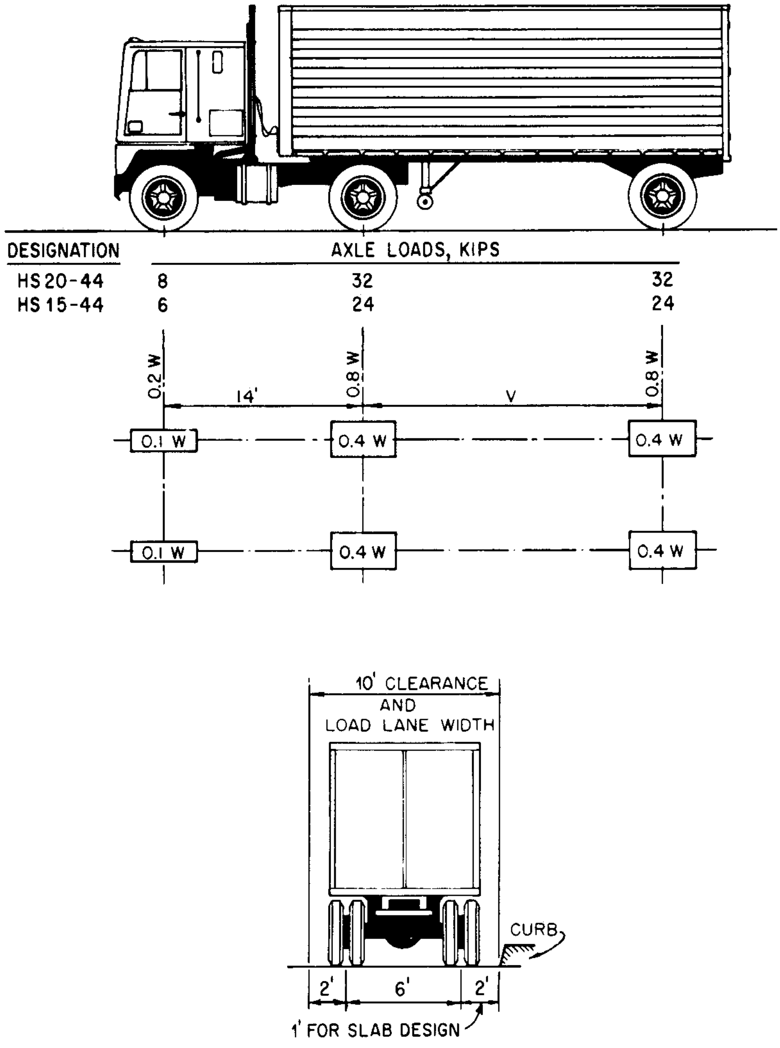
Live Loads. There are four standard classes of highway vehicle loadings included in the Standard Specifications: H15, H20, HS15, and HS20. The AASHTO "Geometric Guide" states that the minimum design loading for new bridges should be HS20 (Fig. 11.l) for all functional classes (local roads through freeways) of highways. Therefore, most bridge owners require design for HS20 truck loadings or greater. AASHTO also specifies an alternative tandem loading of two 25-kip axles spaced 4 ft c to c.

The difference in truck gross weights is a direct ratio of the HS number; e.g., HS15 is 75% of HS20. (The difference between the H and HS trucks is the use of a third axle on an HS truck.) Many bridge owners, recognizing the trucking industries' use of heavier vehicles, are specifying design loadings greater than HS20.

For longer-span bridges, lane loadings are used to simulate multiple vehicles in a given lane. For example, for HS20 loading on a simple span, the lane load is 0.64 kips per ft plus an 18-kip concentrated load for moment or a 26-kip load for shear. A simple-span girder bridge with a span longer than about 140 ft would be subjected to a greater live-load design moment for the lane loading than for the truck loading (Table 11.7). (For end shear and reaction, the breakpoint is about 120 ft). Truck and lane loadings are not applied concurrently for ASD or LFD.

In ASD and LFD, if maximum stresses are induced in a member by loading of more than two lanes, the live load for three lanes should be reduced by 10%, and for four or more lanes, 25%. For LRFD, a reduction or increase depends on the method for live-load distribution.

For LRFD, the design vehicle design load is a combination of truck (or tandem) and lane loads and differs for positive and negative moment. Figure 11.2 shows the governing live loads for LRFD to produce maximum moment in a beam. The vehicular design live loading is one of the major differences in the LRFD Specification. Through statistical analysis of existing highway loadings, and their effect on highway bridges, a combination of the design truck, or design tandem (intended primarily for short spans), and the design lane load, constitutes the HL-93 design live load for LRFD. As in previous specifications, this loading occupies a 10 ft width of a design lane. Depending upon the number of design lanes on the bridge, the possibility of more than one truck being on the bridge must be considered. The effects of the HL-93 loading should be factored by the multiple presence factor (see Table



STANDARD HS TRUCKS

FIGURE 11.1 Standard HS loadings for design of highway bridges. Truck loading for ASD and LFD. *W* is the combined weight of the first two axles. *V* is the spacing of the axles, between 14 and 30 ft, inclusive, that produces maximum stresses.

11.2). However, the multiple presence factor should not to be applied for fatigue calculations, or when the subsequently discussed approximate live load distribution factors are used.

Impact. A factor is applied to vehicular live loads to represent increases in loading due to impact caused by a rough roadway surface or other disturbance. In the AASHTO Standard Specifications, the impact factor *I* is a function of span and is determined from

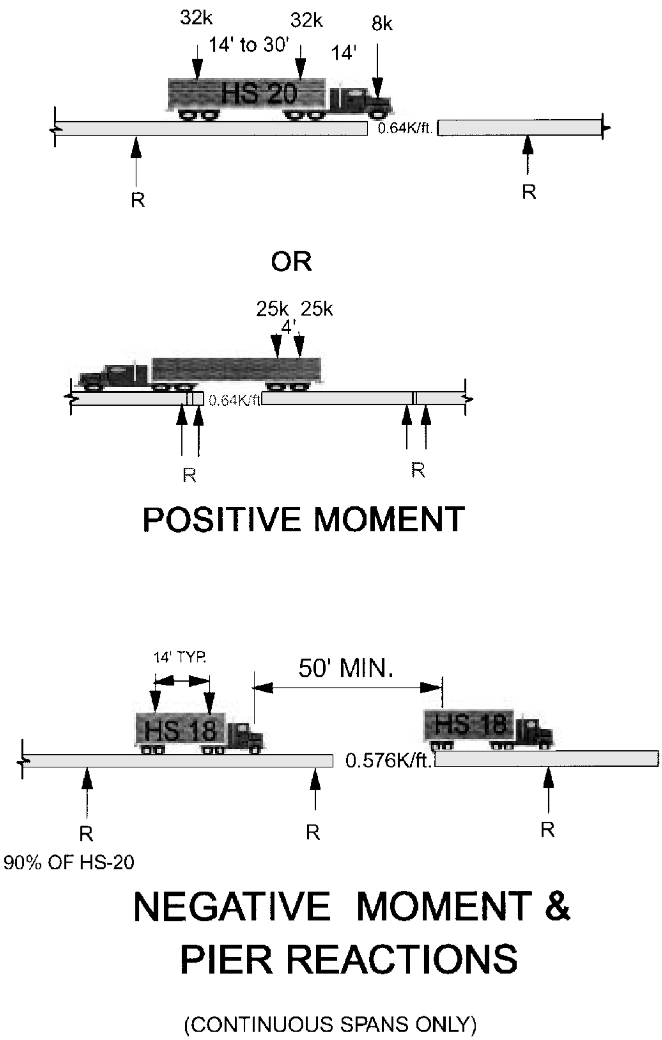


FIGURE 11.2 Loadings for maximum moment and reaction for LRFD design of highway bridges.

TABLE 11.2 Multiple Presence Factors

Number of loaded lanes	Multiple presence factor, *m*
1	1.20
2	1.00
3	0.85
>3	0.65

$$I = \frac{50}{L + 125} \leq 0.30 \tag{11.1}$$

In this formula, L, ft, should be taken as follows:

	For moment	For shear
For simple spans.	L = design span length for roadway decks, floorbeams, and longitudinal stringers	L = length of loaded portion from point of consideration to reaction
For cantilevers.	L = length from point of consideration to farthermost axle	Use $I = 0.30$
For continuous spans	L = design length of span under consideration for positive moment; average of two adjacent loaded spans for negative moment	L = length as for simple spans

For LRFD, the impact factor is modified in recognition of the concept that the factor should be based on the type of bridge component, rather than the span. Termed "dynamic load allowance," values are given in Table 11.3. It is applied only to the truck portion of the live load.

Live Loads on Bridge Railings. Beginning in the 1960s, AASHTO specifications increased minimum design loadings for railings to a 10-kip load applied horizontally, intended to simulate the force of a 4000-lb automobile traveling at 60 mph and impacting the rail at a 25° angle. In 1989, AASHTO published AASHTO "Guide Specifications for Bridge Railings" with requirements more representative of current vehicle impact loads and dependent on the class of highway. Since the effect of impact-type loadings are difficult to predict, the AASHTO Guide requires that railings be subjected to full-scale impact tests to a performance level *PL* that is a function of the highway type, design speed, percent of trucks in traffic, and bridge-rail offset. Generally, only low-volume, rural roads may utilize a rail tested to the PL-1 level, and high-volume interstate routes require a PL-3 rail. The full-scale tests apply the forces that must be resisted by the rail and its attachment details to the bridge deck.

PL-1 represents the forces delivered by an 1800-lb automobile traveling at 50 mph, or a 5400-lb pickup truck at 45 mph, and impacting the rail system at an angle of 20°. PL-2 represents the forces delivered from an automobile or pickup as in PL-1, but traveling at a speed of 60 mph, in addition to an 18,000-lb truck at 50 mph at an angle of 15°. PL-3

TABLE 11.3 Dynamic Load Allowance, IM, for Highway Bridges for LRFD

Component	Limit state	Dynamic load allowance, %
Deck joints	All	75
All other components	Fatigue and fracture	15
	All	33

represents forces from an automobile or pickup as in PL-2, in addition to a 50,000-lb van-type tractor-trailer traveling at 50 mph and impacting at an angle of 15°.

The performance criteria require not only resistance to the vehicle loads but also acceptable performance of the vehicle after the impact. The vehicle may not penetrate or hurdle the railing, must remain upright during and after the collision. and be smoothly redirected by the railing. Thus, a rail system that can withstand the impact of a tractor-trailer truck, may not be acceptable if redirection of a small automobile is not satisfactory.

The LRFD Specifications have included the above criteria, updated to include strong preference for use of rail systems that have been subjected to full scale impact testing, because the force effects of impact type loadings are difficult to predict. Test parameters for rail system impact testing are included in NCHRP Report 350 "Recommended Procedures for the Safety Performance Evaluation of Highway Features." These full-scale tests provide the forces that the rail-to-bridge deck attachment details must resist.

Because of the time and expense involved in full-scale testing, it is advantageous to specify previously tested and approved rails. State highway departments may provide these designs on request.

Earthquake Loads. Seismic design is governed by the AASHTO "Standard Specifications for Seismic Design of Highway Bridges." Engineers should be familiar with the total content of these complex specifications to design adequate earthquake-resistant structures. These specifications are also the basis for the earthquake "extreme-event" limit state of the LRFD specifications, where the intent is to allow the structure to suffer damage but have a low probability of collapse during seismically induced ground shaking. Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage. (See Art. 11.11.)

The purpose of the seismic design specifications is to ". . . establish design and construction provisions for bridges to minimize their susceptibility to damage from earthquakes." Each structure is assigned to a seismic performance category (SPC), which is a function of location relative to anticipated design ground accelerations and to the importance classification of the highway routing. The SPC assigned, in conjunction with factors based on the site soil profile and response modification factor for the type of structure, establishes the minimum design parameters that must be satisfied.

Steel superstructures for beam/girder bridges are rarely governed by earthquake criteria. Also, because a steel superstructure is generally lighter in weight than a concrete superstructure, lower seismic forces are transmitted to the substructure elements.

Vessel Impact Loads. A loading that should be considered by designers for bridges that cross navigable waters is that induced by impact of large ships. Guidance for consideration of vessel impacts on a bridge is included in the AASHTO "Guide Specification and Commentary for Vessel Collision Design of Highway Bridges." This Guide Specification is based on probabilistic theories, accounting for differences in size and frequency of ships that will be using a waterway. The Guide is also the basis for the LRFD extreme-event limit state for vessel collision.

Thermal Loads. Provisions must be included in bridge design for stresses *and* movements resulting from temperature variations to which the structure will be subjected. For steel structures, anticipated temperature extremes are as follows:

Moderate climate: 0 to 120°F
Cold climate: −30°F to +120°F

With a coefficient of expansion of 65×10^{-7} in/in/°F, the resulting change in length of a 100-ft-long bridge member is

Moderate climate: $120 \times 65 \times 10^{-7} \times 100 \times 12 = 0.936$ in

Cold climate: $150 \times 65 \times 10^{-7} \times 100 \times 12 = 1.170$ in

If a bridge is erected at the average of high and low temperatures, the resulting change in length will be one-half of the above.

For complex structures such as trusses and arches, length changes of individual members may induce secondary stresses that must be taken into account.

Longitudinal Forces. Roadway decks are subjected to braking forces, which they transmit to supporting members. AASHTO Standard Specifications specify a longitudinal design force of 5% of the live load in all lanes carrying traffic in the same direction, without impact. The force should be assumed to act 6 ft above the deck.

For LRFD, braking forces should be taken as 25% of the axle weights of the design truck or tandem per lane, placed in all design lanes that are considered to be loaded and which are carrying traffic headed in the same direction. These forces are applied 6.0 ft above the deck in either longitudinal direction to cause extreme force effects.

Centrifugal Force on Highway Bridges. Curved structures will be subjected to centrifugal forces by the live load. The force CF, as a percentage of the live load, without impact, should be applied 6 ft above the roadway surface, measured at centerline of the roadway.

$$CF = \frac{6.68S^2}{R} = 0.00117S^2D \tag{11.2a}$$

where S = design speed, mph
D = degree of curve = $5{,}729.65/R$
R = radius of curve, ft

For LRFD, the coefficient C is multiplied by the design truck or tandem:

$$C = \frac{4v^2}{3gR} \tag{11.2b}$$

where v = highway design speed, ft/s
g = gravitational acceleration, 32.2 f/s^2
R = radius of curvature, ft

Sidewalk Loadings. In the interest of safety, many highway structures in non-urban areas are designed so that the full shoulder width of the approach roadway is carried across the structure. Thus, the practical necessity for a sidewalk or a refuge walk is eliminated. There is no practical necessity that refuge walks on highway structures exceed 2 ft in width. Consequently, no live load need be applied. Current safety standards eliminate refuge walks on full-shoulder-width structures.

In urban areas, however, structures should conform to the configuration of the approach roadways. Consequently, bridges normally require curbs or sidewalks, or both. In these instances, AASHTO Standard Specifications indicate that sidewalks and supporting members should be designed for a live load of 85 psf. Girders and trusses should be designed for the following sidewalk live loads, lb per sq ft of sidewalk area:

Spans 0 to 25 ft 85
Spans 26 to 100 ft 60
Spans over 100 ft .... $P = \left(30 + \frac{3{,}000}{L}\right)\frac{55 - W}{50} \le 60$
where L = loaded length, ft and W = sidewalk width, ft.

For LRFD a load of 75 psf is applied to all sidewalks wider than 2 ft.

Structures designed for exclusive use of pedestrians should be designed for 85 psf under either AASHTO specification.

Curb Loading. For ASD or LFD, curbs should be designed to resist a lateral force of at least 0.50 kip per lin ft of curb. This force should be applied at the top of the curb or 10 in above the bridge deck if the curb is higher than 10 in. For LRFD, curbs are limited to no more than 8 in high.

Where sidewalk, curb, and traffic rail form an integral system, the traffic railing loading applies. Stresses in curbs should be computed accordingly.

Wind Loading on Highway Bridges. The wind forces prescribed below, based on the AASHTO Standard Specifications, Group II and Group V loadings, are considered a uniformly distributed, moving live load. They act on the exposed vertical surfaces of all members, including the floor system and railing as seen in elevation, at an angle of 90° with the longitudinal axis of the structure. These forces are presumed for a wind velocity of 100 mph. They may be modified in proportion to the square of the wind velocity if conditions warrant change.

Superstructure. For trusses and arches: 75 psf but not less than 0.30 kip per lin ft in the plane of loaded chord, nor 0.15 kip per lin ft in the plane of unloaded chord.

For girders and beams: 50 psf but not less than 0.30 kip per lin ft on girder spans.

Wind on Live Load. A force of 0.10 kip per lin ft should be applied to the live load, acting 6 ft above the roadway deck.

Substructure. To allow for the effect of varying angles of wind in design of the substructure, the following longitudinal and lateral wind loads for the skew angles indicated should be assumed acting on the superstructure at the center of gravity of the exposed area.

When acting in combination with live load, the wind forces given in Table 11.4 may be reduced 70%. But they should be combined with the wind load on the live load, as given in Table 11.5.

For usual girder and slab bridges with spans not exceeding about 125 ft, the following wind loads on the superstructure may be used for substructure design in lieu of the more elaborate loading specified in Tables 11.4 and 11.5:

Wind on structure
 50 psf transverse
 12 psf longitudinal

Wind on live load

TABLE 11.4 Skewed Superstructure Wind Forces for Substructure Design*

	Trusses		Girders	
Skew angle of wind, deg	Lateral load, psf	Longitudinal load, psf	Lateral load, psf	Longitudinal load, psf
0	75	0	50	0
15	70	12	44	6
30	65	28	41	12
45	47	41	33	16
60	25	50	17	19

* "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

TABLE 11.5 Wind Forces on Live Loads for Substructure Design*

Skew angle of wind, deg	Lateral load, lb per lin ft	Longitudinal load, lb per lin ft
0	100	0
15	88	12
30	82	24
45	66	32
60	34	38

* "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

100 psf transverse
40 psf longitudinal

Transverse and longitudinal loads should be applied simultaneously.

Wind forces applied directly to the substructure should be assumed at 40 psf for 100-mph wind velocity. For wind directions skewed to the substructure, this force may be resolved into components perpendicular to end and side elevations, acting at the center of gravity of the exposed areas. This wind force may be reduced 70% when acting in combination with live load.

Overturning Forces. In conjunction with forces tending to overturn the structure, there should be added an upward wind force, applied at the windward quarter point of the transverse superstructure width, of 20 psf, assumed acting on the deck and sidewalk plan area. For this load also, a 70% reduction may be applied when it acts in conjunction with live load.

For LRFD wind load calculations, see Art. 13.8.2.

Uplift on Highway Bridges. Provision should be made to resist uplift by adequately attaching the superstructure to the substructure. AASHTO Standard Specifications recommend engaging a mass of masonry equal to:

1. 100% of the calculated uplift caused by any loading or combination of loading in which the live-plus-impact loading is increased 100%.
2. 150% of the calculated uplift at working-load level.

Anchor bolts under the above conditions should be designed at 150% of the basic allowable stress.

AASHTO LRFD Specifications require designing for calculated uplift forces due to buoyancy, etc., and specifically requires hold down devices in seismic zones 2, 3, and 4.

Forces of Stream Current, Ice, and Drift on Highway Bridges. All piers and other portions of structures should be designed to resist the maximum stresses induced by the forces of flowing water, floating ice, or drift.

For ASD or LFD, the longitudinal pressure P, psf, of flowing water on piers should be calculated from

$$P = KV^2 \qquad (11.3a)$$

where V = velocity of water, fps, and K = constant. In the AASHTO Standard Specifications, $K = 1.4$ for all piers subject to drift build-up and for square-ended piers, 0.7 for circular piers, and 0.5 for angle-ended piers where the angle is 30° or less.

In the ASSHTO LRFD Specifications, the pressure P, ksf, is calculated from

$$P = \frac{C_D V^2}{1000} \tag{11.3b}$$

where V = velocity of water, fps, for design flood and appropriate limit state, and CD is a drag coefficient (0.7 for semi-circular nosed pier, 1.4 for square ended pier, 1.4 for debris launched against pier, and 0.8 for wedge nosed pier with nose angle 90° or less).

For ice and drift loads, see AASHTO specifications.

Buoyancy should be taken into account in the design of substructures, including piling, and of superstructures, where necessary.

11.5 LOAD COMBINATIONS AND EFFECTS

11.5.1 Overview

The following groups represent various combinations of service loads and forces to which a structure may be subjected. Every component of substructure and superstructure should be proportioned to resist all combinations of forces applicable to the type of bridge and its site.

For working-stress design, allowable unit stresses depend on the loading group, as indicated in Table 11.6. These stresses, however, do not govern for members subject to repeated stresses when allowable fatigue stresses are smaller. Note that no increase is permitted in allowable stresses for members carrying only wind loads. When the section required for each loading combination has been determined, the largest should be selected for the member being designed.

The ''Standard Specifications for Highway Bridges'' of the American Association of State Highway and Transportation Officials specifies for LFD, factors to be applied to the various types of loads in loading combinations. These load factors are based on statistical analysis of loading histories. In addition, in LRFD, reduction factors are applied to the nominal resistance of materials in members and to compensate for various uncertainties in behavior.

To compare the effects of the design philosophies of ASD, LFD, and LRFD, the group loading requirements of the three methods will be examined. For simplification, only D, L, and I of Group I loading will be considered. Although not stated, all three methods can be considered to use the same general equation for determining the effects of the combination of loads:

$$N\Sigma(F \times \text{load}) \leq \text{RF} \times \text{nominal resistance} \tag{11.4}$$

where N = design factor used in LRFD for ductility, redundancy, and operational importance of the bridge
= 1.0 for ASD and LFD

$\Sigma(F \times \text{load})$ = sum of the factored loads for a combination of loads

F = load factor that is applied to a specific load
= 1.0 for ASD; D, L, and I

load = one or more service loads that must be considered in the design

RF = resistance factor (safety factor for ASD) that is applied to the nominal resistance

Nominal resistance = the strength of a member based on the type of loading; e.g., tension, compression, or shear

For a non-compact flexural member subjected to bending by dead load, live load, and impact forces, let D, L, I represent the maximum tensile stress in the extreme surface due to dead load, live load, and impact, respectively. Then, for each of the design methods, the following must be satisfied:

TABLE 11.6 Loading Combinations for Allowable-Stress Design

Group	loading combination	Percentage of basic unit stress
I	$D + L + I + CF + E + B + SF$	100
I*A*	$D + 2(L + I)$	150
I*B*	$D + (L + I)^* + CF + E + B + SF$	†
II	$D + E + B + SF + W$	125
III	$D + L + I + CF + E + B + SF + 0.3W + WL + LF$	125
IV	$D + L + I + E + B + SF + T$	125
V	$D + E + B + SF + W + T$	140
VI	$D + I + CF + E + B + SF + 0.3W + WL + LF + T$	140
VII	$D + E + B + SF + EQ$	133
VIII	$D + L + I + CF + E + B + SF + ICE$	140
IX	$D + E + B + SF + W + ICE$	150
X‡	$D + L + I + E$	100

where D = dead load
L = live load
I = live-load impact
E = earth pressure (factored for some types of loadings)
B = buoyancy
W = wind load on structure
WL = wind load on live load of 0.10 kip per lin ft
LF = longitudinal force from live load
CF = centrifugal force
T = temperature
EQ = earthquake
SF = stream-flow pressure
ICE = ice pressure

* For overload live load plus impact as specified by the operating agency.

† Percentage $= \dfrac{\text{maximum unit stress (operating rating)}}{\text{allowable basic unit stress}} \times 100$

‡ For culverts.

ASD: $$D + L + I \leq 0.55F_y \tag{11.5}$$

LFD: $$1.3D + 2.17(L + I) \leq F_y \tag{11.6}$$

For strength limit state I, assuming D is for components and attachments

LRFD: $$1.25D + 1.75(L + I) \leq F_y \tag{11.7}$$

For LFD and LRFD, if the section is compact, the full plastic moment can be developed. Otherwise, the capacity is limited to the yield stress in the extreme surface.

The effect of the applied loads appears to be less for LRFD, but many other factors apply to LRFD designs that are not applicable to the other design methods. One of these is a difference in the design live-load model. Another major difference is that the LRFD specifications require checking of connections *and* components for minimum and maximum loadings. (Dead loads of components and attachments are to be varied by using a load factor of 0.9 to 1.25.) LRFD also requires checking for five different *strength* limit states, three *service* limit states, a *fatigue-and-fracture* limit state, and two *extreme-event* limit states. Although each structure may not have to be checked for all these limit states, the basic philosophy of the LRFD specifications is to assure serviceability over the design service life, safety of the bridge through redundancy and ductility of all components and connections, and survival (prevention of collapse) of the bridge when subjected to an extreme event; e.g., a 500-year flood. (See Art. 11.5.4.)

11.5.2 Simplified Example of Methods

To compare the results of a design by ASD. LFD, and LRFD, a 100-ft, simple-span girder bridge is selected as a simple example. It has an 8-in-thick, noncomposite concrete deck, and longitudinal girders, made of grade 50 steel, spaced 12 ft c to c. It will carry HS20 live load. The section modulus S, in^3, will be determined for a laterally braced interior girder with a live-load distribution factor of 1.0.

The bending moment due to dead loads is estimated to be about 2,200 ft-kips. The maximum moment due to the HS20 truck loading is 1,524 ft-kips (Table 11.7).

$$\text{LRFD Lane-load live-load moment} = \frac{wL^2}{8} = \frac{0.64(100)^2}{8} = 800 \text{ ft-kips}$$

For both ASD and LFD, the impact factor (Eq. 11.1) is

$$I = \frac{50}{100 + 125} = 0.22$$

For LRFD, $IM = 0.33$, Table 11.3.

Allowable-Stress Design. The required section modulus S for the girder for allowable-stress design is computed as follows: The design moment is

$$M = M_D + (1 + I)M_L = 2{,}200 + 1.22 \times 1{,}524 = 4{,}059 \text{ ft-kips}$$

For $F_y = 50$ ksi, the allowable stress is $F_b = 0.55 \times 50 = 27$ ksi. The section modulus required is then

$$S = \frac{M}{F_b} = \frac{4{,}059 \times 12}{27} = 1{,}804 \text{ in}^3$$

The section in Fig. 11.3, weighing 280.5 lb per ft, supplies a section modulus within 1% of required S—O.K.

Load-Factor Design. The design moment for LFD is

$$\begin{aligned} M_u &= 1.3M_D + 2.17(1 + I)M_L \\ &= 1.3 \times 2{,}200 + 2.17 \times 1.22 \times 1{,}524 = 6{,}895 \text{ ft-kips} \end{aligned}$$

For $F_y = 50$ ksi, the section modulus required for LFD is

$$S = \frac{M_u}{F_y} = \frac{6{,}895 \times 12}{50} = 1{,}655 \text{ in}^3$$

If a noncompact section is chosen, this value of S is the required elastic section modulus. For a compact section, it is the plastic section modulus Z. Figure 11.4 shows a noncompact section supplying the required section modulus, with a ⅜-in-thick web and 1⅝-in-thick flanges. For a compact section, a ⅝-in-thick web is required and 1¼-in-thick flanges are satisfactory. In this case, the noncompact girder is selected and will weigh 265 lb per ft.

TABLE 11.7 Maximum Moments, Shears, and Reactions for Truck or Lane Loads on One Lane, Simple Spans*

	H15		H20		HS15		HS20	
Span, ft	Moment†	End shear and end reaction‡	Moment†	End shear and end reaction‡	Moment†	End shear and end reaction‡	Moment†	End shear and end reaction‡
10	60.0§	24.0§	80.0§	32.0§	60.0§	24.0§	80.0§	32.0§
20	120.0§	25.8§	160.0§	34.4§	120.0§	31.2§	160.0§	41.6§
30	185.0§	27.2§	246.6§	36.3§	211.6§	37.2§	282.1§	49.6§
40	259.5§	29.1	346.0§	38.8	337.4§	41.4§	449.8§	55.2§
50	334.2§	31.5	445.6§	42.0	470.9§	43.9§	627.9§	58.5§
60	418.5	33.9	558.0	45.2	604.9§	45.6§	806.5§	60.8§
70	530.3	36.3	707.0	48.4	739.2§	46.8§	985.6§	62.4§
80	654.0	38.7	872.0	51.6	873.7§	47.7§	1,164.9§	63.6§
90	789.8	41.1	1,053.0	54.8	1,008.3§	48.4§	1,344.4§	64.5§
100	937.5	43.5	1,250.0	58.0	1,143.0§	49.0§	1,524.0§	65.3§
110	1,097.3	45.9	1,463.0	61.2	1,277.7§	49.4§	1,703.6§	65.9§
120	1,269.0	48.3	1,692.0	64.4	1,412.5§	49.8§	1,883.3§	66.4§
130	1,452.8	50.7	1,937.0	67.6	1,547.3§	50.7	2,063.1§	67.6
140	1,648.5	53.1	2,198.0	70.8	1,682.1§	53.1	2,242.8§	70.8
150	1,856.3	55.5	2,475.0	74.0	1,856.3	55.5	2,475.1	74.0
160	2,075.0	57.9	2.768.0	77.2	2,076.0	57.9	2,768.0	77.2
170	2,307.8	60.3	3,077.0	80.4	2,307.8	60.3	3,077.1	80.4
180	2,551.5	62.7	3,402.0	83.6	2,551.5	62.7	3,402.1	83.6
190	2,807.3	65.1	3,743.0	86.8	2,807.3	65.1	3,743.1	86.8
200	3,075.0	67.5	4,100.0	90.0	3,075.0	67.5	4,100.0	90.0
220	3,646.5	72.3	4,862.0	96.4	3,646.5	72.3	4,862.0	96.4
240	4,266.0	77.1	5,688.0	102.8	4,266.0	77.1	5,688.0	102.8
260	4,933.5	81.9	6,578.0	109.2	4,933.5	81.9	6,578.0	109.2
280	5,649.0	86.7	7,532.0	115.6	5,649.0	86.7	7,532.0	115.6
300	6,412.5	91.5	8,550.0	122.0	6,412.5	91.5	8,550.0	122.0

* Based on "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials. Impact not included.

† Moments in thousands of ft-lb (ft-kips).

‡ Shear and reaction in kips. Concentrated load is considered placed at the support. Loads used are those stipulated for shear.

§ Maximum value determined by standard truck loading. Otherwise, standard lane loading governs.

Load-and-Resistance-Factor Design. The live-load moment M_L is produced by a combination of truck and lane loads, with impact applied only to the truck moment:

$$M_L = 1.33 \times 1524 + 800 = 2827 \text{ ft-kips}$$

The load factor N is a combination of factors applied to the loadings. Assume that the bridge has ductility (0.95), redundancy (0.95), and is of operational importance (1.05). Thus, $N = 0.95 \times 0.95 \times 1.05 = 0.95$. The design moment for limit state I is

$$M_u = N[\mathrm{F_D}M_D + F_L\mathrm{M_L}]$$

$$= 0.95[1.25 \times 2200 + 1.75 \times 2827] = 7312 \text{ ft-kips}$$

Hence, since the resistance factor for flexure is 1.0, the section modulus required for LRFD is

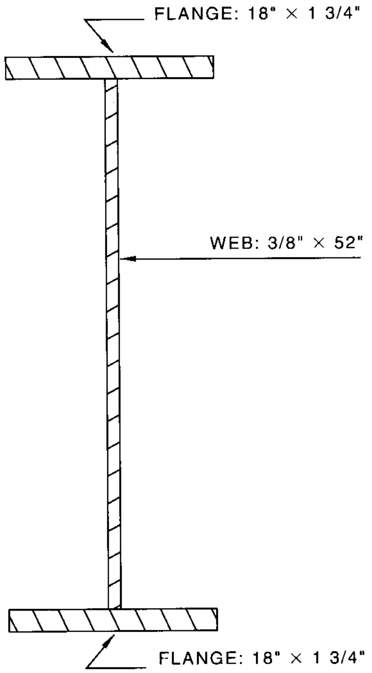


FIGURE 11.3 Girder with transverse stiffeners determined by ASD and LRFD for a 100-ft span: $S = 1799\ \text{in}^3$; $w = 280.5$ lb per ft.

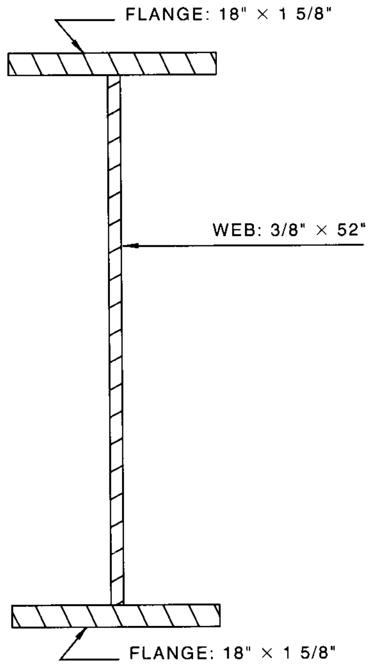


FIGURE 11.4 Girder with transverse stiffeners determined by load-factor design for a 100-ft span: $S = 1681\ \text{in}^3$; $w = 265$ lb per ft.

$$S = \frac{7312 \times 12}{50} = 1755\ \text{in}^3$$

The section selected for ASD (Fig. 11.3) is satisfactory for LRFD.

For this example, the weight of the girder for LFD is 94% of that required for ASD and 90% of that needed for LRFD. The heavier girder required for LRFD is primarily due to the larger live load specified. For both LFD and LRFD, a compact section is advantageous, because it reduces the need for transverse stiffeners for the same basic weight of girder.

11.5.3 LRFD Limit States

The LRFD Specifications requires bridges "to be designed for specified **limit states** to achieve the objectives of constructibility, safety and serviceability, with due regard to issues of inspectability, economy and aesthetics". Each component and connection must satisfy Eq. 11.8 for each limit state. All limit states are considered of equal importance. The basic relationship requires that the effect of the sum of the factored loads, Q, must be less than or equal to the factored resistance, R, of the bridge component being evaluated for *each* limit state. This is expressed as

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r \tag{11.8}$$

where η_i = a factor combining the effects of ductility, η_D, redundancy, η_R, and importance, η_I. For a non-fracture critical steel member on a typical bridge, η_i will be 1.0.

γ_i = statistically based factor to be applied to the various load effects

Q_i = effect of each individual load as included in Art. 11.5.4. This could be a moment, shear, stress, etc.
ϕ = statistically based resistance factor to be applied to the material property, as discussed in Art. 11.6.
R_n = nominal resistance of the material being evaluated based on the stress, deformation or strength of the material.
R_r = factored resistance, $R_n \times \phi$.

There are four limit states to be satisfied: Service; Fatigue and Fracture; Strength; and, Extreme Event. The Service Limit State has three different combinations of load factors, which place restrictions on stress, deformation and crack width under regular service conditions. Service I and III apply to control of prestressed members. Service II, intended to control yielding of steel structures and slip of slip-critical connections, corresponds to what was previously known as the "overload" check.

The Fatigue and Fracture Limit State checks the dynamic effect on the bridge components of a single truck known as the fatigue truck. Restrictions are placed on the range of stress induced by passage of trucks on the bridge. This limit is intended to prevent initiation of fatigue cracking during the design life of the bridge. Article 11.10 provides additional discussion of the Fatigue Limit State.

Fracture is controlled by the requirement for minimum material toughness values included in the LRFD Specification and the AASHTO or ASTM material specifications, and depends upon where the bridge is located. (See Art. 1.1.5.) Section 11.9 provides additional discussion of the Fracture Limit State.

The Strength Limit State has five different combinations of load factors to be satisfied. This limit state assures the component and/or connection has sufficient strength to withstand the designated combinations of the different permanent and transient loadings that could statistically happen during the life of the structure. This is the most important limit state since it checks the basic strength requirements. Strength I is the basic check for normal usage of the bridge. Strength II is the check for owner specified permit vehicles. Strength III checks for the effects of high winds (>55 mph) with no live load on the bridge, since trucks would not be able to travel safely under this condition. Strength IV checks strength under a possible high dead to live load force-effect ratio, such as for very long spans. This condition governs when the ratio exceeds 7.0. Strength V checks the strength when live load is on the bridge and a 55 mph wind is blowing.

Extreme Event Limit State is intended "to ensure the structural survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle or ice flow possibly under a scoured condition." This design requirement recognizes that structural damage is acceptable under extreme events, but collapse should be prevented.

For the design example included in the Appendix, page 11.78, the engineers provided a summary to illustrate the relative influence for all the LRFD requirements on the design. The results for each limit state are expressed in terms of a performance ratio, defined as the ratio of a calculated value to the corresponding allowable value. This summary, Table A1, indicates that the *Fatigue and Fracture Limit State, Base metal at connection plate weld to bottom flange* (*at 0.41L*) is the governing criteria. In fact, it is slightly overstressed, in that the ratio between actual and allowable value is 1.008. However, this very small excess was accepted. It is recommended that designers develop performance ratios for all designs.

11.5.4 LRFD Load Combinations

The effects of each of the loads discussed in Art. 11.4, appropriately factored, must be evaluated in various combinations for LRFD as indicated in Tables 11.8 and 11.9. These combinations are statistically based determinations for structure design. Only those applicable to steel bridge superstructure designs are listed. See the LRFD Specification for a complete

TABLE 11.8 Partial Load Combinations and Load Factors for LRFD

	Factors for indicated load combinations*				
Limit state	*DC*, *DD*, *DW*, *EH*, *EV*, *ES*	*LL*, *IM*, *CE*, *BR*, *PL*, *LS*	*WA*	*WS*	*WL*
Strength I	γ_p	1.75	1.00	—	—
Strength II	γ_p	1.35	1.00	—	—
Strength V	γ_p	1.35	1.00	0.40	1.00
Service II	1.00	1.30	1.00	—	—
Fatigue (*LL*, *IM* & *CE* only)	—	0.75	—	—	—

* See Table 11.9 for γ_p values. See Art. 11.4 for load descriptions.

listing. See the example in the Appendix for a listing of design factors and illustration of application of load combinations and load factors.

11.6 NOMINAL RESISTANCE FOR LRFD

The nominal resistance of the various bridge components, such as flexural members, webs in shear, and fasteners (bolts or welds), is given by equations in the LRFD Specification. Each nominal resistance must be multiplied by a resistance factor, ϕ, which is a statistically based number that accounts for differences between calculated strength and actual strength. The ϕ factor, Table 11.10, provides for inaccuracies in theory and variations in material properties and dimensions. Expressions for the nominal resistance of many types of members are given in other sections of this Handbook. The nominal resistance of slip-critical bolts is considered in the following.

Field connections in beams and girders are almost always made using high-strength bolts. Bolts conforming to AASHTO M164 (ASTM A325) are the most used types. AASHTO M253 (ASTM A490) are another type, but are rarely used. The LRFD Specification requires that bolted connections ''subject to stress reversal, heavy impact loads, severe vibration or where stress and strain due to joint slippage would be detrimental to the serviceability of the structure'' be designed as slip-critical. Slip-critical connections must be proportioned at Service II Limit State load combinations as specified in Table 11.8. The nominal slip resistance, R_n, of each bolt is

TABLE 11.9 LRFD Load Factors for Permanent Loads, γ_p

	Load factor	
Type of load	Maximum	Minimum
DC: component & attachments	1.25	0.90
DW: wearing surface & utilities	1.50	0.65

TABLE 11.10 Resistance Factors, ϕ, for Strength Limit State for LRFD

Flexure	$\phi_f = 1.00$
Shear	$\phi_v = 1.00$
Axial compression, steel only	$\phi_c = 0.90$
Axial compression, composite	$\phi_c = 0.90$
Tension, fracture in net section	$\phi_u = 0.80$
Tension, yielding in gross section	$\phi_y = 0.95$
Bearing on pins, in reamed, drilled or bolted holes and milled surfaces	$\phi_b = 1.00$
Bolts bearing on material	$\phi_{bb} = 0.80$
Shear connectors	$\phi_{sc} = 0.85$
A325 and A490 bolts in tension	$\phi_t = 0.80$
A307 bolts in tension	$\phi_t = 0.80$
A307 bolts in shear	$\phi_s = 0.65$
A325 and A490 bolts in shear	$\phi_s = 0.80$
Block shear	$\phi_{bs} = 0.80$
Weld metal in complete penetration welds:	
Shear on effective area	$\phi_{e1} = 0.85$
Tension or compression normal to effective area	ϕ = base metal ϕ
Tension or compression parallel to axis of weld	ϕ = base metal ϕ
Weld metal in partial penetration welds:	
Shear parallel to axis of weld	$\phi_{e2} = 0.80$
Tension or compression parallel to axis of weld	ϕ = base metal ϕ
Compression normal to the effective area	ϕ = base metal ϕ
Tension normal to the effective area	$\phi_{e1} = 0.80$
Weld metal in fillet welds:	
Tension or compression parallel to axis of the weld	ϕ = base metal
Shear in throat of weld metal	$\phi_{e2} = 0.80$

Note: All resistance factors for the extreme event limit state, except for bolts, are taken as 1.0.

$$R_n = K_h K_S N_S P_t \tag{11.9}$$

where N_s = number of slip planes per bolt
P_t = minimum required bolt tension (see Table 11.11)
K_h = hole size factor (see Table 11.12)
K_s = surface condition factor (see Table 11.13)

11.7 DISTRIBUTION OF LOADS THROUGH DECKS

Specifications of the American Association of State Highway and Transportation Officials (AASHTO) require that the width of a bridge roadway between curbs be divided into design traffic lanes 12 ft wide and loads located to produce maximum stress in supporting members.

TABLE 11.11 Minimum Required Bolt Tension

Bolt diameter, in	Required tension, P_t, kips M164 (A325)	M253 (A490)
5⁄8	19	27
3⁄4	28	40
7⁄8	39	55
1	51	73
1 1⁄8	56	92
1 1⁄4	72	116
1 3⁄8	85	139
1 1⁄2	104	169

(Fractional parts of design lanes are not used.) Roadway widths from 20 to 24 ft, however, should have two design lanes, each equal to one-half the roadway width. Truck and lane loadings are assumed to occupy a width of 10 ft placed anywhere within the design lane to produce maximum effect.

If curbs, railings, and wearing surfaces are placed after the concrete deck has gained sufficient strength, their weight may be distributed equally to all stringers or beams. Otherwise, the dead load on the outside stringer or beam is the portion of the slab it carries.

The strength and stiffness of the deck determine, to some extent, the distribution of the live load to the supporting framing.

Shear. For determining end shears and reactions, the deck may be assumed to act as a simple span between beams for lateral distribution of the wheel load. For shear elsewhere, the wheel load should be distributed by the method required for bending moment.

Moments in Longitudinal Beams. For ASD and LRFD, the fraction of a wheel load listed in Table 11.14 should be applied to each interior longitudinal beam for computation of live-load bending moments.

For an outer longitudinal beam, the live-load bending moments should be determined with the reaction of the wheel load when the deck is assumed to act as a simple span between beams. When four or more longitudinal beams carry a concrete deck, the fraction of a wheel load carried by an outer beam should be at least $S/5.5$ when the distance between that beam and the adjacent interior beam S, ft, is 6 or less. For $6 < S < 14$, the fraction should be at least $S/(4 + 0.25S)$. For $S > 14$, no minimum need be observed.

TABLE 11.12 Values of K_h

Standard size holes	1.0
Oversize and short-slotted holes	0.85
Long-slotted holes with slot perpendicular to direction of force	0.70
Long-slotted holes with slot parallel to direction of force	0.60

TABLE 11.13 Values of K_s

Class A surface conditions	0.33
Class B surface conditions	0.50
Class C surface conditions	0.33

Note:
Class A surfaces are with unpainted clean mill scale, or blast cleaned surfaces with a Class A coating.
Class B surfaces are unpainted and blast cleaned, or painted with a Class B coating.
Class C surfaces are hot-dipped galvanized, and roughened by wire brushing.

Moments in Transverse Beams. When a deck is supported directly on floorbeams, without stringers, each beam should receive the fraction of a wheel load listed in Table 11.15, as a concentrated load, for computation of live-load bending moments.

Distribution for LRFD. Research has led to recommendations for changes in the distribution factors DF in Tables 11.14 and 11.15. AASHTO has adopted these recommendations as the basis for an approximate method in the LRFD Specifications, when a bridge meets specified requirements. As an alternative, a more refined method such as finite-element analysis is permitted.

TABLE 11.14 Fraction of Wheel Load DF Distributed to Longitudinal Beams for ASD and LRFD*

Deck	Bridge with one traffic lane	Bridge with two or more traffic lanes
Concrete:		
On I-shaped steel beams	$S/7$, $S \leq 10$†	$S/5.5$, $S \leq 14$†
On steel box girders	$W_L = 0.1 + 1.7R + 0.85/N_w$‡	
Steel grid:		
Less than 4 in thick	$S/4.5$	$S/4$
4 in or more thick	$S/6$, $S \leq 6$†	$S/5$, $S \leq 10.5$†
Timber:		
Plank	$S/4$	$S/3.75$
Strip 4 in thick or multiple-layer floors over 5 in thick	$S/4.5$	$S/4$
Strip 6 in or more thick	$S/5$, $S \leq 5$†	$S/4.25$, $S \leq 6.5$†

*Based on "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.
†For larger values of S, average beam spacing, ft, the load on each beam should be the reaction of the wheel loads with the deck assumed to act as a simple span between beams.
‡Provisions for reduction of live load do not apply to design of steel box girders with W_L, fraction of a wheel (both front and rear).

R = number of design traffic lanes N_w divided by number of box girders ($0.5 \leq R \leq 1.5$)
$N_w = W_c/12$, reduced to nearest whole number
W_c = roadway width, ft, between curbs or barriers if curbs are not used.

TABLE 11.15 Fraction of Wheel Load Distributed to Transverse Beams*

Deck	Fraction per beam
Concrete	$S/6$†
Steel grid:	
Less than 4 in thick	$S/4.5$
4 in or more thick	$S/6$†
Timber:	
Plank	$S/4$
Strip 4 in thick, wood block on 4-in plank subfloor, or multiple-layer floors more than 5 in thick	$S/4.5$
Strip 6 in or more thick	$S/5$†

*Based on "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

†When the spacing of beams S, ft, exceeds the denominator, the load on the beam should be the reaction of the wheel loads when the deck is assumed to act as a simple span between beams.

The LRFD Specification gives the following equations as the approximate method for determining the distribution factor for moment for steel girders. They are in terms of the LRFD design truck load per lane, and their application is illustrated in the design example in the Appendix. For one lane loaded

$$\mathrm{DF} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \tag{11.10}$$

For two lanes loaded

$$\mathrm{DF} = 0.075 + (S/9.5)^{0.6}(S/L)^{0.2}(K_g/12Lt_s^3)^{0.1} \tag{11.11}$$

where S = beam spacing, ft
L = span, ft
t_s = thickness of concrete slab, in
$K_g = n(I + Ae_g^2)$
n = modular ratio = ratio of steel modulus of elasticity E_s to the modulus of elasticity E_c of the concrete slab
I = moment of inertia, in^4, of the beam
A = area, in^2, of the beam
e_g = distance, in, from neutral axis of beam to center of gravity of concrete slab

Eq. 11.10 and 11.11 apply only for spans from 20 ft to 240 ft with 4-½ to 12 in thick concrete decks (or concrete filled, or partially filled, steel grid decks), on four or more steel girders spaced between 3.5 ft and 16.0 ft. The multiple presence factors, m, in Table 11.2 are **not** to be used when this approximate method of load distribution is used. For girder spacing outside the above limits, the live load on each beam is determined by the lever rule (summing moments about one support to find the reaction at another support by assuming the supported component is hinged at interior supports). When more refined methods of analysis are used, the LRFD Specification states that "a table of live load distribution coefficients for extreme force effects in each span shall be provided in the contract documents to aid in permit issuance and rating of the bridge."

11.8 BASIC ALLOWABLE STRESSES FOR BRIDGES

Table 11.16 lists the basic allowable stresses for highway bridges recommended in AASHTO "Standard Specifications for Highway Bridges" for ASD. The stresses are related to the minimum yield strength F_y, ksi, or minimum tensile strength F_u, ksi, of the material in all cases except those for which stresses are independent of the grade of steel being used.

The basic stresses may be increased for loading combinations (Art. 11.5). They may be superseded by allowable fatigue stresses (Art. 11.10).

Allowable Stresses in Welds. Standard specifications require that weld metal used in bridges conform to the "Bridge Welding Code," ANSI/AASHTO/AWS D1.5, American Welding Society.

Yield and tensile strengths of weld metal usually are specified to be equal to or greater than the corresponding strengths of the base metal. The allowable stresses for welds in bridges generally are as follows:

Groove welds are permitted the same stress as the base metal joined. When base metals of different yield strengths are groove-welded, the lower yield strength governs.

Fillet welds are allowed a shear stress of $0.27F_u$, where F_u is the tensile strength of the electrode classification or the tensile strength of the connected part, whichever is less. When quenched and tempered steels are joined, an electrode classification with strength less than that of the base metal may be used for fillet welds, but this should be clearly specified in the design drawings.

Plug welds are permitted a shear stress of 12.4 ksi.

These stresses may be superseded by fatigue requirements (Art. 11.10). The basic stresses may be increased for loading combinations as noted in Art. 11.5.

Effective area of groove and fillet welds for computation of stresses equals the effective length times effective throat thickness. The effective shearing area of plug welds equals the nominal cross-sectional area of the hole in the plane of the faying surface.

Effective length of a groove weld is the width of the parts joined, perpendicular to the direction of stress. The effective length of a straight fillet weld is the overall length of the full-sized fillet, including end returns. For a curved fillet weld, the effective length is the length of line generated by the center point of the effective throat thickness. For a fillet weld in a hole or slot, if the weld area computed from this length is greater than the area of the hole in the plane of the faying surface, the latter area should be used as the effective area.

Effective throat thickness of a groove weld is the thickness of the thinner piece of base metal joined. (No increase is permitted for weld reinforcement. It should be removed by grinding to improve fatigue strength.) The effective throat thickness of a fillet weld is the shortest distance from the root to the face, computed as the length of the altitude on the hypotenuse of a right triangle. For a combination partial-penetration groove weld and a fillet weld, the effective throat is the shortest distance from the root to the face minus ⅛ in for any groove with an included angle less than 60° at the root of the groove.

In some cases, strength may not govern the design. Standard specifications set maximum and minimum limits on size and spacing of welds. These are discussed in Art. 5.19.

Rollers and Expansion Rockers. The maximum compressive load, P_m, kips, should not exceed the following:

for cylindrical surfaces,

$$P_m \leq 8 \left(\frac{WD_1}{1 - D_1/D_2} \right) \frac{F_y^2}{E_s} \tag{11.12}$$

for spherical surfaces,

TABLE 11.16 Basic Allowable Stresses, ksi, for Allowable Stress Design of Highway Bridges[a]

Loading condition	Allowable stress, ksi
Tension:	
Axial, gross section without bolt holes	$0.55F_y$
Axial, net section	$0.55F_y$[b]
Bending, extreme fiber of rolled shapes, girders, and built-up sections, gross section[c]	$0.55F_y$
Compression:	
Axial, gross section in:	
Stiffeners of plate girders	$0.55F_y$
Splice material	$0.55F_y$
Compression members;[d]	
$KL/r \leq C_c$	$\dfrac{F_y}{F.S.}\left[1 - \dfrac{(KL/r)^2 F_y}{4\pi^2 E}\right]$
$KL/r \geq C_c$	$\dfrac{\pi^2 E}{F.S.(KL/r)^2}$
Bending, extreme fiber of:	
Rolled shapes, girders, and built-up sections with:	
Compression flange continuously supported	$0.55F_y$
Compression flange intermittently supported[g]	$\dfrac{50 \times 10^6 C_b}{S_{xc}}\left(\dfrac{I_{yc}}{L}\right) \times \sqrt{0.772\dfrac{J}{I_{yc}} + 9.87\left(\dfrac{d}{L}\right)^2}$
Pins	$0.80F_y$
Shear:	
Webs of rolled beams and plate girders, gross section	$0.33F_y$
Pins	$0.40F_y$
Bearing:	
Milled stiffeners and other steel parts in contact (rivets and bolts excluded)	$0.80F_y$
Pins:	
Not subject to rotation[h]	$0.80F_y$
Subject to rotation (in rockers and hinges)	$0.40F_y$

[a] F_y = minimum yield strength, ksi, and F_u = minimum tensile strength, ksi. Modulus of elasticity E = 29,000 ksi.

[b] Use 0.46 F_u for ASTM A709, Grades 100/100W (M270) steels. Use net section if member has holes more than 1¼ in in diameter.

[c] When the area of holes deducted for high-strength bolts or rivets is more than 15% of the gross area, that area in excess of 15% should be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than 1¼ in diameter should be deducted. For ASTM A709 Grades 100/100W (M270) steels, use $0.46F_u$ on net section instead of $0.55F_y$ on gross section. For other steels, limit stress on net section to $0.50F_u$ and stress on gross section to $0.55F_y$.

[d] K = effective length factor. See Art. 6.16.2.
$C_c = \sqrt{2\pi^2 E/F_y}$
E = modulus of elasticity of steel, ksi
r = governing radius of gyration, in
L = actual unbraced length, in
$F.S.$ = factor of safety = 2.12

TABLE 11.16 Basic Allowable Stresses, ksi, for Allowable Stress Design of Highway Bridges[a] (*Continued*)

[g] Not to exceed $0.55F_y$.

L = length, in, of unsupported flange between lateral connections, knee braces, or other points of support

I_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web, in^4

d = depth of girder, in

$J = \dfrac{[(bt^3)_c + (bt^3)_t + Dt_w^3}{3}$, where b and t are the flange width and thickness, in, of the compression and tension flange, respectively, and t_w and D are the web thickness and depth, in, respectively

S_{xc} = section modulus with respect to compression flange, in^3

$C_b = 1.75 + 1.05\ (M_1/M_2) + 0.3\ (M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 the larger end moment in the unbraced segment of the beam; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

For the use of larger C_b values, see Structural Stability Research Council *Guide to Stability Design Criteria for Metal Structures.* If cover plates are used, the allowable static stress at the point of theoretical cutoff should be determined by the formula.

[h] Applicable to pins used primarily in axially loaded members, such as truss members and cable adjusting links, and not applicable to pins used in members subject to rotation by expansion or deflection.

$$P_m \leq 40 \left(\frac{D_1}{1 - D_1/D_2} \right)^2 \frac{F_y^3}{E_s^2} \tag{11.13}$$

where D_1 = diameter of rocker or roller surface, in

D_2 = diameter of mating surface, in. D_2 should be taken as positive if the curvatures have the same sign, infinite if the mating surface is flat.

F_y = specified minimum yield strength of the least strong steel at the contact surface, ksi

E_s = modulus of elasticity of steel, ksi

W = width of the bearing, in

Allowable Stresses for Bolts. Bolted shear connections are classified as either bearing-type or slip-critical. The latter are required for connections subject to stress reversal, heavy impact, large vibrations, or where joint slippage would be detrimental to the serviceability of the bridge. These connections are discussed in Sec. 5. Bolted bearing-type connections are restricted to members in compression and secondary members.

Fasteners for bearing-type connections may be ASTM A307 carbon-steel bolts or A325 or A490 high-strength bolts. High-strength bolts are required for slip-critical connections and where fasteners are subjected to tension or combined tension and shear.

Bolts for highway bridges are generally ¾ or ⅞ in in diameter. Holes for high-strength bolts may be standard, oversize, short-slotted, or long-slotted. Standard holes may be up to 1⁄16 in larger in diameter than the nominal diameters of the bolts. Oversize holes may have a maximum diameter of 15⁄16 in for ¾-in bolts and 1 1⁄16 in for ⅞-in bolts. Minimum diameter of a slotted hole is the same as that of a standard hole. For ¾-in and ⅞-in bolts, short-slotted holes may be up to 1 in and 1⅛ in long, respectively, and long-slotted holes, a maximum of 1⅞ and 2 3⁄16 in long, respectively.

In the computation of allowable loads for shear or tension on bolts, the cross-sectional area should be based on the nominal diameter of the bolts. For bearing, the area should be taken as the product of the nominal diameter of the bolt and the thickness of the metal on which it bears.

Allowable stresses for bolts specified in ‘‘Standard Specifications for Highway Bridges’’ of the American Association of State Highway and Transportation Officials (AASHTO) are summarized in Tables 11.17 and 11.18. The percentages of stress increase specified for load combinations in Art. 11.5 also apply to high-strength bolts in slip-critical joints, but the percentage may not exceed 133%.

TABLE 11.17 Allowable Stresses, ksi, on Bolts in Highway Bridges—ASD

		Allowable shear, F_v				
		Slip-critical connections				
				Long-slotted holes		
ASTM designation	Allowable tension, F_t	Standard-size holes	Oversize and short-slotted holes	Transverse load	Parallel load	Bearing-type joints
A307	18					11
A325	38[a]					19
		15*	13*	11*	9*	
		23†	19†	16†	14†	
		15‡	13‡	11‡	9‡	
A490	47[a]					25
		19*	16*	13*	11*	
		29†	24†	20†	17†	
		19‡	16‡	13‡	11‡	

* **Class A:** When contact surfaces have a slip coefficient of 0.33, such as clean mill scale and blast-cleaned surfaces, with Class A coating.

† **Class B:** When contact surfaces have a slip coefficient of 0.50, such as blast-cleaned surfaces and such surfaces with Class B coating.

‡**Class C:** When contact surfaces have a slip coefficient of 0.40, such as hot-dipped galvanized and roughened surfaces.

Class A and B coatings include those with a mean slip coefficient of as least 0.33 or 0.50, respectively. See Appendix A, "Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Structural Connections of the Engineering Foundation.

TABLE 11.18 Allowable Bearing Stresses, ksi, on Bolted Joints in Highway Bridges—ASD

Conditions for connection material	A307 bolts	A325 bolts	A490 bolts
Threads permitted in shear planes	20		
Single bolt in line of force in a standard or short-slotted hole		$0.9F_u$*†	$0.9F_u$*†
Two or more bolts in line of force in standard or short-slotted holes		$1.1F_u$*†	$1.1F_u$*
Bolts in long-slotted holes		$0.9F_u$*†	$0.9F_u$*

* F_u = specified minimum tensile strength of connected parts. Connections with bolts in oversize holes or in slotted holes with the load applied less than about 80° or more than about 100° to the axis of the slot should be designed for a slip resistance less than that computed from Eq. 11.14.

† Not applicable when the distance, parallel to the load, from the center of a bolt to the edge of the connected part is less than 1½d, where d is the nominal diameter of the bolt, or the distance to an adjacent bolt is less than 3d.

In addition to satisfying these allowable-stress requirements, connections with high-strength bolts should also meet the requirements for combined tension and shear and for fatigue resistance.

Furthermore, the load P_S, kips, on a slip-critical connection should be less than

$$P_s = F_s A_b N_b N_s \tag{11.14}$$

where F_s = allowable stress, ksi, given in Table 11.17 for a high-strength bolt in a slip-critical joint
A_b = area, in^2, based on the nominal bolt diameter
N_b = number of bolts in the connection
N_s = number of slip planes in the connection

Surfaces in slip-critical joints should be Class A, B, or C, as described in Table 11.17, but coatings providing a slip coefficient less than 0.33 may be used if the mean slip coefficient is determined by test. In that case, F_s for use in Eq. (11.14) should be taken as for Class A coatings but reduced in the ratio of the actual slip coefficient to 0.33.

Tension on high-strength bolts may result in prying action on the connected parts. See Art. 5.25.3.

Combined shear and tension on a slip-critical joint with high-strength bolts is limited by the interaction formulas in Eqs. (11.15) and (11.16). The shear f_v, ksi (slip load per unit area of bolt), for A325 bolts may not exceed

$$f_v = F_s(1 - 1.88 f_t / F_u) \tag{11.15}$$

where f_t = computed tensile stress in the bolt due to applied loads including any stress due to prying action, ksi
F_s = nominal slip resistance per unit of bolt area from Table 11.17
F_u = 120 ksi for A 325 bolts up to 1-in diameter
= 105 ksi for A 325 bolts over 1-in diameter
= 150 ksi for A 490 bolts.

Where high-strength bolts are subject to both shear and tension, the tensile stress may not exceed the value obtained from the following equations:

for $f_v / F_v \leq 0.33$

$$F_t' = F_t \tag{11.16a}$$

for $f_v / F_v > 0.33$

$$F_t' = F_t \sqrt{1 - (f_v / F_v)^2} \tag{11.16b}$$

where f_v = computed bolt shear stress in shear, ksi
F_v = allowable shear stress on bolt from Table 11.17, ksi
F_t = allowable tensile stress or bolt from Table 11.17, ksi
F_t' = reduced allowable tensile stress on bolt due to the applied shear stress, ksi.

Combined shear and tension in a bearing-type connection is limited by the interaction equation.

$$f_v^2 + 0.36 f_t^2 = F_v^2 \tag{11.17}$$

where f_v = computed shear stress ksi, in bolt, and F_v = allowable shear, ksi, in bolt (Table 11.17). Equation (11.17) is based on the assumption that bolt threads are excluded from the shear plane.

Fatigue may control design of a bolted connection. To limit fatigue, service-load tensile stress on the area of a bolt based on the nominal diameter, including the effects of prying action, may not exceed the stress in Table 11.19. The prying force may not exceed 80% of the load.

11.9 FRACTURE CONTROL

Fracture-critical members are treated in the AASHTO LRFD Specifications and in the AASHTO "Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members." A fracture-critical member (FCM) or member component is a tension member or component whose failure is expected to result in collapse of the bridge or the inability of the bridge to perform its function. Although the definition is limited to tension members, failure of any member or component due to any type of stress or strain can also result in catastrophic failure. This concept applies to members of any material.

The AASHTO "Standard Specifications for Highway Bridges" contains provisions for structural integrity. These recommend that, for new bridges, designers specify designs and details that employ continuity and redundancy to provide one or more alternate load paths. Also, external systems should be provided to minimize effects of probable severe loads.

The AASHTO LRFD specification, in particular, requires that multi-load-path structures be used unless "there are compelling reasons to the contrary." Also, main tension members and components whose failure may cause collapse of the bridge must be designated as FCM and the structural system must be designated nonredundant. Furthermore, the LRFD specification includes fracture control in the fatigue and fracture limit state.

Design of structures can be modified to eliminate the need for special measures to prevent catastrophe from a fracture, and when this is cost-effective, it should be done. Where use of an FCM is unavoidable, for example, the tie of a tied arch, as much redundancy as possible should be provided via continuity, internal redundancy through use of multiple plates, and similar measures.

Steels used in FCM must have supplemental impact properties as listed in Table 1.2. FCM should be so designated on the plans with the appropriate temperature zone (Table 1.2) based on the anticipated minimum service temperature. Fabrication requirements for FCM are outlined in ANSI/AASHTO/AWS D1.5.

High Performance Steels (HPS), as discussed in Art. 1.5 provide an opportunity to significantly increase reliability of steel bridges. With impact properties for this steel usually exceeding 100 ft-lb at −10°F, it easily meets the requirements for fracture critical material. For example, the HPS70W material requirement for welded, 4-in thick plates, in FCMs in a temperature zone 3 application is 35 ft-lb at −30°F (see Table 1.2).

TABLE 11.19 Allowable Tensile Fatigue Stresses for Bolts in Highway Bridges*—ASD

Number of cycles	A325 bolts	A490 bolts
20,000 or less	39.5	48.5
20,000 to 500,000	35.5	44.0
More than 500,000	27.5	34.0

*As specified in "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

11.10 REPETITIVE LOADINGS

Most structural damage to steel bridges is the result of repetitive loading from trucks or wind. Often, the damage is caused by secondary effects, for example, when live loads are distributed transversely through cross frames and induce large out-of-plane distortions that were not taken into account in design of the structure. Such strains may initiate small fatigue cracks. Under repetitive loads, the cracks grow. Unless the cracks are discovered early and remedial action taken, they may create instability under a combination of stress, loading rate, and temperature, and brittle fracture could occur. Proper detailing of steel bridges can prevent such fatigue crack initiation.

To reduce the probability of fracture, the structural steels included in the AASHTO specifications for M270 steels, and ASTM A709 steels when ''supplemental requirements'' are ordered,* are required to have minimum impact properties (Art. 1.1.5). The higher the impact resistance of the steel, the larger a crack has to be before it is susceptible to unstable growth. With the minimum impact properties required for bridge steels, the crack should be large enough to allow discovery during the biannual bridge inspection before fracture occurs. The M270 specification requires average energy in a Charpy V-notch test of 15 ft-lb for grade 36 steels and ranging up to 35 ft-lb for grade 100 steels, at specified test temperatures. More conservative values are specified for FCM members (Art. 11.9). Toughness values depend on the lowest ambient service temperature (LAST) to which the structure may be subjected. Test temperatures are 70°F higher than the LAST to take into account the difference between the loading rate as applied by highway trucks and the Charpy V-notch impact tests.

Allowable Fatigue Stresses for ASD and LFD Design. Members, connections, welds, and fasteners should be designed so that maximum stresses do not exceed the basic allowable stresses (Art. 11.8) and the range in stress due to loads does not exceed the allowable fatigue stress range. Table 11.20A lists allowable fatigue stress ranges in accordance with the number of cycles to which a member or component will be subjected and several stress categories for structural details. The details described in Table 6.27 for structural steel for buildings are generally applicable also to highway bridges. The diagrams are provided as illustrative examples and are not intended to exclude other similar construction. (See also Art. 6.26.) The allowable stresses apply to load combinations that include live loads and wind. For dead plus wind loads, use the stress range for 100,000 cycles. Table 11.20B lists the number of cycles to be used for design.

Stress range is the algebraic difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebraic sign from compression stress.

Table 11.20A (a) is applicable to redundant load-path structures. These provide multiple load paths so that a single fracture in a member or component cannot cause the bridge to collapse. The AASHTO standard specifications list as examples a simply supported, single-span bridge with several longitudinal beams and a multi-element eye bar in a truss. Table 11.20A (b) is applicable to non-redundant load-path structures. The AASHTO specifications give as examples flange and web plates in bridges with only one or two longitudinal girders, one-element main members in trusses, hanger plates, and caps of single- or two-column bents.

Improved ASD and LFD Provisions for Fatigue Design. AASHTO has published ''Guide Specifications for Fatigue Design of Steel Bridges.'' These indicate that the fatigue provisions in the ''Standard Specifications for Highway Bridges'' do not accurately reflect the actual

---

*ASTM A709 steels thus specified are equivalent to AASHTO material specification M270 steels and grade designations are similar.

TABLE 11.20A Allowable Stress Range, ksi, for Repeated Loads on Highway Bridges[a]—ASD and LFD Design

	Number of loading cycles			
Stress category	100,000[b]	500,000[c]	2,000,000[d]	More than 2,000,000[d]
(a) For redundant load-path structures				
A	63 (49)[e]	37 (29)[e]	24 (18)[e]	24 (16)[e]
B	49	29	18	16
B′	39	23	14.5	12
C	35.5	21	13	10
				12[g]
D	28	16	10	7
E	22	13	8	4.5
E′	16	9.2	5.8	2.6
F	15	12	9	8
(b) For non-redundant load-path structures				
A	50 (39)[e]	29 (23)[e]	24 (16)[e]	24 (16)[e]
B	39	23	16	16
B′	31	18	11	11
C	28	16	10	9
			12[f]	11[f]
D	22	13	8	5
E[g]	17	10	6	2.3
E′	12	7	4	1.3
F	12	9	7	6

[a] Based on data in the "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.
[b] Equivalent to about 10 applications every day for 25 years.
[c] Equivalent to about 50 applications every day for 25 years.
[d] Equivalent to about 200 applications every day for 25 years.
[e] Values in parentheses apply to unpainted weathering steel A709, all grades, when used in conformance with Federal Highway Administration "Technical Advisory on Uncoated Weathering Steel in Structures," Oct. 3, 1989.
[f] For welds of transverse stiffeners to webs or flanges of girders.
[g] AASHTO prohibits use of partial-length welded cover plates on flanges more than 0.8 in thick in non-redundant load-path structures.

fatigue conditions in such bridges; instead, they combine an artificially high stress range with an artificially low number of cycles to get a reasonable result. The actual effective stress ranges rarely exceed 5 ksi, whereas the number of truck passages in the design life of a bridge can exceed many million.

For this reason, these guide specifications give alternative fatigue-design procedures to those in the standard specifications. They are based on a more realistic loading, equal to 75% of a single HS20 (or HS15) truck with a fixed rear axle spacing of 30 ft. The procedures accurately reflect the actual conditions in bridges subjected to traffic loadings and provide the following additional advantages: (1) They permit more flexibility in accounting for differing traffic conditions at various sites. (2) They permit design for any desired design life. (3) They provide reasonable and consistent levels of safety over a broad range of design conditions. (4) They are based on extensive research and can be conveniently modified in

TABLE 11.20B Design Stress Cycles for Main Load-Carrying Members for ASD

Type of road	Case	ADTT[a]	Truck loading	Lane loading[b]
Freeways, expressways, major highways, and streets	I	2,500 or more	2,000,000[c]	500,000
Freeways, expressways, major highways, and streets	II	Less than 2,500	500,000	100,000
Other highways and streets not included in Case I or II	III		100,000	100,000

[a] Average Daily Truck Traffic (one direction).
[b] Longitudinal members should also be checked for truck loading.
[c] Members must also be investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge.

the future if needed to reflect new research results. (5) They are consistent with fatigue-evaluation procedures for existing bridges.

The guide specifications use the same detail categories and corresponding fatigue strength data as the standard specifications. They also use methods of calculating stress ranges that are similar to those used with the standard specifications.

Thus, it is important that designers possess both the standard specifications and the guide specifications to design fatigue-resistant details properly. However, there is a prevailing misconception in the interpretation of the term "fatigue life." For example, the guide specifications state, "The safe fatigue life of each detail shall exceed the desired design life of the bridge." The implication is that the initiation of a fatigue crack is the end of the service life of the structure. In fact, the initiation of a fatigue crack does *not* mean the end of the life of an existing bridge, or even of the particular member, as documented by the many bridges that have experienced fatigue cracking and even full-depth fracture of main load-carrying members. These cracks and fractures have been successfully repaired by welding, drilling a hole at the crack tip, or placing bolted cover plates over a fracture. These bridges continue to function without reduction in load-carrying capacity or remaining service life.

Fatigue Provisions for LRFD. The AASHTO load-and-resistance factor design specifications can be best understood by considering a schematic log-log fatigue-resistance curve where stress range is plotted against number of cycles, Fig. 11.5. The curve represents the locus of points of equal fatigue damage. Along the sloping portion, for a given stress range, a corresponding finite life is anticipated. The constant-amplitude fatigue threshold represented by the dashed horizontal line defines the infinite-life fatigue resistance. If all of the stress ranges experienced by a detail are less than the stress range defined by the fatigue threshold, it is anticipated that the detail will not crack.

The *LRFD Specifications* attempt to combine the best attributes of the *Guide Specification,* including the special fatigue loading described previously, and those of the *Standard Specifications,* including the detail category concept. The *LRFD Specifications* define the nominal fatigue resistance for each fatigue category as

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{1/3} \geq \frac{1}{2}(\Delta F)_{TH} \tag{11.18}$$

where $N = (365)(75)n(ADTT)_{SL}$
A = fatigue detail category constant, Table 11.21
n = number of stress range cycles per truck passage, Table 11.22
$(ADTT)_{SL}$ = single-lane $ADTT$ (average daily truck traffic)

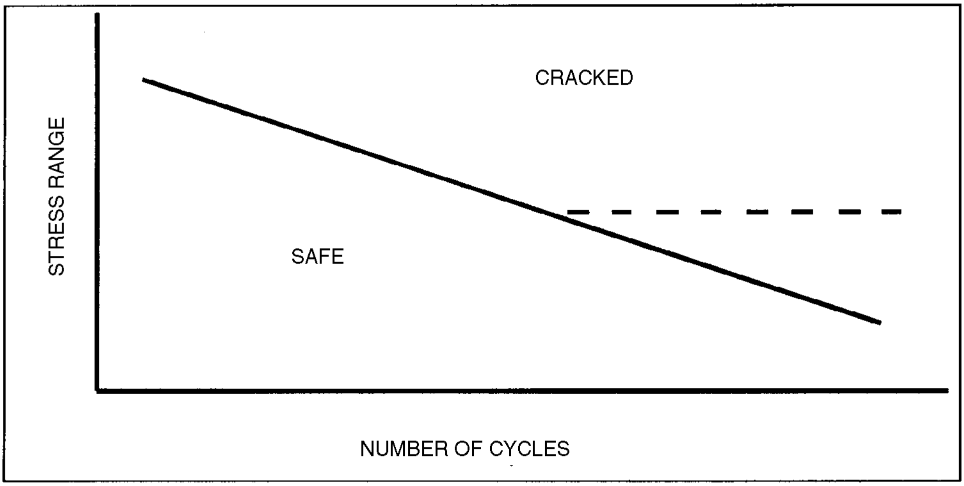


FIGURE 11.5 Schematic fatigue-resistance curve.

$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold, ksi, Table 11.23

However, the nominal fatigue resistance range for base metal at details connected with transversely loaded fillet welds, where a discontinuous plate is loaded, is taken as the lesser of $(\Delta F)^c_n$ and:

$$(\Delta F)_n = (\Delta F)^c_n \left(\frac{0.06 + 0.79 \dfrac{H}{t_p}}{1.1 t_p^{1/6}} \right) \tag{11.19}$$

where $(\Delta F)^c_n$ = the nominal fatigue resistance for detail category C, ksi
H = effective throat of fillet weld, in
t_p = thickness of loaded plate, in

TABLE 11.21 Detail Category* Constant, A

Detail category	Constant, A
A	250.0×10^{-8}
B	120.0×10^{-8}
B′	61.0×10^{-8}
C	44.0×10^{-8}
C′	44.0×10^{-8}
D	22.0×10^{-8}
E	11.0×10^{-8}
E′	3.9×10^{-8}
M164 (A325) bolts in axial tension	17.1×10^{-8}
M253 (A490) bolts in axial tension	31.5×10^{-8}

*Detail categories are similar to those presented in Art. 6.22. See AASHTO LRFD Specification for complete details.

TABLE 11.22 Cycles per Truck Passage, n

(*a*) Longitudinal members

Member type	Span length >40.0 ft	Span length ≤40.0 ft
Simple-span girders	1.0	2.0
Continuous girders		
1) Near interior support	1.5	2.0
2) Elsewhere	1.0	2.0
Cantilever girders	5.0	
Trusses	1.0	

(*b*) Transverse members

Spacing >20.0 ft	Spacing ≤20.0 ft
1.0	2.0

The term $(A/N)^{1/3}$ in Eq. 11.18 represents the sloping line in Fig. 11.5, and $(\Delta F)_{TH}$ the horizontal line. The multiplier of ½ represents the ratio of the factored fatigue load to the maximum load. In other words, if the stress range due to the factored fatigue truck is less than ½ of the constant-amplitude fatigue threshold, the detail should experience infinite life. The load factor for fatigue is 0.75, Table 11.8. The truck loading for fatigue is shown in Fig. 11.6.

The fatigue resistance defined in LRFD is similar to that in earlier specifications, although the format is different. Complete LRFD design examples, including fatigue designs of typical girder details, have demonstrated that design in accord with the *LRFD Specifications* is basically equivalent to design in accordance with the provisions for redundant structures in

TABLE 11.23 Constant Amplitude Fatigue Threshold, $(\Delta F)_{TH}$

Detail category	Threshold, ksi
A	24.0
B	16.0
B′	12.0
C	10.0
C′	12.0
D	7.0
E	4.5
E′	2.6
M164 (A325) bolts in axial tension	31.0
M253 (A490) bolts in axial tension	38.0

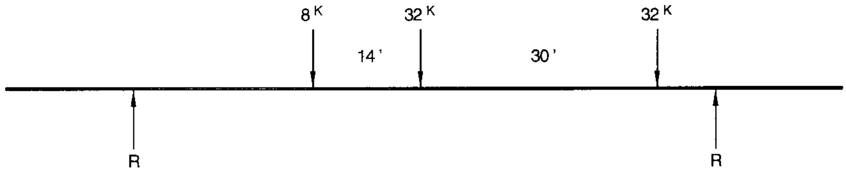


FIGURE 11.6 Design truck for calculation of fatigue stresses. Impact is taken as 15% of live load.

the *Standard Specifications.* In developing the LRFD provisions, it was determined that because of the greater fracture toughness specified for non-redundant structures, a reduction in allowable stress range for such structures was unnecessary.

An understanding of the fatigue susceptibility of various details is important for the design of reliable structures. Numerous references are available to maintain familiarity with the state of the art, including:

Fisher, J. W., Frank, K. H., Hirt, M. A., and McNamee, B. M. (1970). *Effect of Weldments on the Fatigue Strength of Steel Beams,* NCHRP Report 102. Highway Research Board, Washington, DC.

Fisher, J. W., Albrecht, P. A., Yen, B. T., Klingerman, D. J., and McNamee, B. M. (1974). *Fatigue Strength of Steel Beams with Transverse Stiffeners and Attachments.* NCHRP Report 147. Highway Research Board, Washington, DC.

Fisher, J. W., Hausammann, H., Sullivan, M. D., and Pense, A. W. (1979). *Detection and Repair of Fatigue Damage in Welded Highway Bridges.* NCHRP Report 206. Transportation Research Board, Washington, DC.

Fisher, J. W., Barthelemy, B. M., Mertz, D. R., and Edinger, J. A. (1980). *Fatigue Behavior of Full-Scale Welded Bridge Attachments.* NCHRP Report 227. Transportation Research Board, Washington, DC.

Fisher, J. W. (1974). *Guide to 1974 AASHTO Fatigue Specifications,* American Institute of Steel Construction, Chicago, Ill.

Keating, P. B. and Fisher, J. W. (1986). *Evaluation of Fatigue Test Data and Design Criteria.* NCHRP Report 299, Transportation Research Board, Washington, DC.

11.11 DETAILING FOR EARTHQUAKES

Bridges must be designed so that catastrophic collapse cannot occur from seismic forces. Damage to a structure, even to the extent that it becomes unusable, may be acceptable, but collapse is not!

The "Standard Specifications for Seismic Design of Highway Bridges" of the American Association of State Highway and Transportation Officials contain standards for seismic design that are comprehensive in nature and embody several concepts that are significant departures from previous design provisions. They are based on the observed performance of bridges during past earthquakes and on recent research. The specifications include an extensive commentary that documents the basis for the standards and an example illustrating their use. LRFD specifications include seismic design as part of the Extreme Event Limit State.

Although the specifications establish design seismic-force guidelines, of equal importance is the emphasis placed on proper detailing of bridge components. For instance, one of the leading causes of collapse when bridges are subjected to earthquakes is the displacement that occurs at bridge seats. If beam seats are not properly sized, the superstructure will fall off the substructure during an earthquake. Minimum support lengths to be provided at beam ends, based on seismic performance category, is a part of the specifications. Thus, to ensure earthquake-resistant structures, both displacements and loads must be taken into account in bridge design.

Retrofitting existing structures to provide earthquake resistance is also an important consideration for critical bridges. Guidance is provided in ''Seismic Retrofitting Guidelines for Highway Bridges,'' Federal Highway Administration (FHWA) Report No. RD-83/007, and ''Seismic Design and Retrofit Manual for Highway Bridges,'' FHWA Report No. IP-87-6, Federal Highway Administration, McLean, VA 22101.

11.12 DETAILING FOR BUCKLING

Prevention of buckling is important in bridge design, because of the potential for collapse. Three forms of buckling must be considered in bridge design.

11.12.1 Types of Buckling

The first, and most serious, is primary buckling of an axially loaded compression member. Such column buckling may include Euler-type elastic buckling and inelastic buckling. This is a rare occurrence with highway bridges, attesting to the adequacy of the current design provisions.

A second form of buckling is local plate buckling. This form of buckling usually manifests itself in the form of excessive distortion of plate elements. This may not be acceptable from a visual perspective, even though the member capacity may be sufficient. When very thin plates are specified, in the desire to achieve minimum weight and supposedly minimum cost, distortions due to welding may induce initial out-of-plane deformations that then develop into local buckling when the member is loaded. Proper welding techniques and use of transverse or longitudinal stiffeners, while maintaining recommended width-thickness limitations on plates and stiffeners, minimize the probability of local buckling.

The third, and perhaps the most likely form of buckling to occur in steel bridges, is lateral buckling. It develops when compression causes a flexural member to become unstable. Such buckling can be prevented by use of lateral bracing, members capable of preventing deformation normal to the direction of the compressive stress at the point of attachment.

Usually, lateral buckling is construction-related. For example, it can occur when a member is fabricated with very narrow compression flanges without adequate provision for transportation and erection stresses. It also can occur when adequate bracing is not provided during deck-placing sequences. Consequently, designers should ensure that compression flanges are proportioned to provide stability during all phases of the service life of bridges, including construction stages, when temporary lateral bracing may be required.

11.12.2 Maximum Slenderness Ratios of Bridge Members

Ratios of effective length to least radius of gyration of columns should not exceed the values listed in Table 11.24.

The length of top chords of half-through trusses should be taken as the distance between laterally supported panel points. The length of other truss members should be taken as the distance between panel-point intersections, or centers of braced points, or centers of end connections.

TABLE 11.24 Maximum Slenderness Ratios for Highway Bridge Members for ASD, LFD, and LRFD

Member	Highway
Main compression members	120
Wind and sway bracing in compression	140
Tension members	
Main	200
Main subject to stress reversal	140
Bracing	240

11.12.3 Plate-Buckling Criteria for Compression Elements

The "Standard Specifications for Highway Bridges" of the American Association of State Highway and Transportation Officials set a maximum width-thickness ratio b/t or D/t for compression members as given in Table 11.25.

11.12.4 Stiffening of Girder Webs (ASD)

Bending of girders tends to buckle thin webs. This buckling may be prevented by making the web sufficiently thick (Table 11.25) or by stiffening the web with plates attached normal to the web. The stiffeners may be set longitudinally or transversely (vertically), or both ways. (See Art 11.17.)

Bearing stiffeners are required for plate girders at concentrated loads, including all points of support. Rolled beams should have web stiffeners at bearings when the unit shear stress in the web exceeds 75% of the allowable shear. Bearing stiffeners should be placed in pairs, one stiffener on each side of the web. Plate stiffeners or the outstanding legs of angle stiffeners should extend as close as practicable to the outer edges of the flanges. The stiffeners should be ground to fit against the flange through which the concentrated load, or reaction, is transmitted, or they should be attached to that flange with full-penetration groove welds. They should be fillet welded to both flanges if they also serve as diaphragms connections. They should be designed for bearing over the area actually in contact with the flange. No allowance should be made for the portions of the stiffeners fitted to fillets of flange angles or flange-web welds. A typical practice is to clip plate stiffeners at 45° at upper and lower ends to clear such fillets or welds. Connections of bearing stiffeners to the web should be designed to transmit the concentrated load, or reaction, to the web.

Bearing stiffeners should be designed as columns. For ordinary welded girders, the column section consists of the plate stiffeners and a strip of web. (At interior supports of continuous hybrid girders, however, when the ratio of web yield strength to tension-flange yield strength is less than 0.7, no part of the web should be considered effective.) For stiffeners consisting of two plates, the effective portion of the web is a centrally located strip $18t$ wide, where t is the web thickness, in (Fig. 11.7*a*). For stiffeners consisting of four or more plates, the effective portion of the web is a centrally located strip included between the stiffeners and extending beyond them a total distance of $18t$ (Fig. 11.7*b*). The radius of gyration should be computed about the axis through the center of the web. The width-thickness ratio of a stiffener plate or the outstanding leg of a stiffener angle should not exceed

TABLE 11.25 Maximum Width-Thickness Ratios for Compression Elements of Highway Bridge Members for ASD

(*a*) Plates supported on only one side

Components	Limiting stress, ksi[a]	b/t for calculated stress less than the limiting stress[b]	b/t for calculated stress equal to the limiting stress[a]
Compression members[c]	$0.44F_y$	$51.4/\sqrt{f_a} \leq 12$	$75/\sqrt{F_y}$
Welded-girder flange[d]	$0.55F_y$	$103/\sqrt{f_b} \leq 24$	$140/\sqrt{F_y}$
Composite girder[d]		$122/\sqrt{f_{dl}}$	
Bolted-girder flange[e]	$0.55F_y$	$51.4/\sqrt{f_b} \leq 12$	$70/\sqrt{F_y}$
Composite girder[e]		$61/\sqrt{f_{dl}}$	

(*b*) Plates supported on two sides

Component	Limiting stress, ksi[a]	b/t for calculated stress less than the limiting stress[b]	b/t for calculated stress equal to the limiting stress[a]
Girder web without stiffeners[f]	F_v	$270/\sqrt{f_v} \leq 150$	$470/\sqrt{F_y}$
Girder web with transverse stiffeners[f]	F_b	$730/\sqrt{f_b} \leq 170$	$990/\sqrt{F_y}$
Girder web with longitudinal stiffeners[f,h]	F_b	$128\sqrt{k}/\sqrt{f_b} \leq 340$	
Girder web with transverse stiffeners and one longitudinal stiffener[f]	F_b		$1980/\sqrt{F_y}$
Box-shapes—main plates or webs[g]	$0.44F_y$	$126/\sqrt{f_a} \leq 45$	$190/\sqrt{F_y}$
Box or H shapes—solid cover plates or webs between main elements[g]	$0.44F_y$	$158/\sqrt{f_a} \leq 50$	$240/\sqrt{F_y}$
Box shapes—perforated cover plates[g]	$0.44F_y$	$190/\sqrt{f_a} \leq 55$	$285/\sqrt{F_y}$

[a] F_y = specified minimum yield strength of the steel, ksi
F_b = allowable bending stress, ksi
F_v = allowable shear stress, ksi
[b] f_a = computed compressive stress, ksi
f_b = computed compressive bending stress, ksi
f_v = computed shear stress, ksi
f_{dl} = top flange compressive stress due to noncomposite dead load.
[c] For outstanding plates, outstanding legs of angles, and perforated plates at the perforations. Width b is the distance from the edge of plate or edge of perforation to the point of support. t is the thickness.
[d] b is the width of the compression flange and t is the thickness.
[e] b is the width of flange angles in compression, except those reinforced by plates. t is the thickness.
[f] b represents the depth of the web D, clear unsupported distance between flanges.
[g] When used as compression members, b is the distance between points of support for the plate and between roots of flanges for webs of rolled elements. t is the thickness.
[h] Plate buckling coefficient k is defined as follows:

$$\text{for} \quad \frac{d_s}{D_c} \geq 0.4 \qquad k = 5.17\left(\frac{D}{d_s}\right)^2$$

$$\text{for} \quad \frac{d_s}{D_c} < 0.4 \qquad k = 11.64\left(\frac{D}{D_c - d_s}\right)^2$$

where d_s is the distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or the leg of the compression flange component, and D_c is the depth of the web in compression.

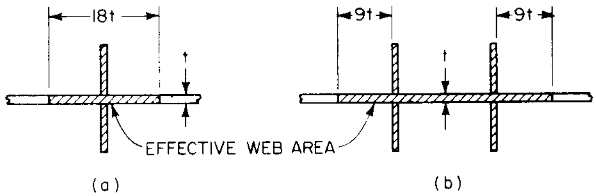


FIGURE 11.7 Effective column areas for design of stiffeners: (*a*) for one pair of stiffeners; (*b*) for two pairs.

$$\frac{b}{t} = \frac{69}{\sqrt{F_y}} \tag{11.20}$$

where F_y = yield strength, ksi, for stiffener steel.

For highway bridges, no stiffeners, other than bearing stiffeners, are required, in general, if the depth-thickness ratio of the web does not exceed the value for girder webs without stiffeners in Table 11.25. But stiffeners may be required for attachment of cross frames.

Transverse stiffeners should be used for highway girders where D/t exceeds the aforementioned values, where D is the depth of the web, the clear unsupported distance between flanges. When transverse stiffeners are used, the web depth-thickness ratio should not exceed the values given in Table 11.25 for webs without longitudinal stiffeners and with one longitudinal stiffener. Intermediate stiffeners may be A36 steel, whereas web and flanges may be a higher grade.

Where required, transverse stiffeners may be attached to the highway-girder web singly or in pairs. Where stiffeners are placed on opposite sides of the web, they should be fitted tightly against the compression flange. Where a stiffener is placed on only one side of the web, it must be in bearing against, but need not be attached to the compression flange. Intermediate stiffeners need not bear against the tension flange. However, the distance between the end of the stiffener weld and the near edge of the web-to-flange fillet welds must not be less than $4t$ or more than $6t$.

Transverse stiffeners may be used, where not otherwise required, to serve as connection plates for diaphragms or cross frames. In such cases, the stiffeners must be rigidly connected to both the tension and compression flanges to prevent web fatigue cracks due to out-of-plane movements. The stiffener may be welded to both flanges, or a special bolted detail may be used to connect to the tension flange. The appropriate fatigue category must be used for the tension flange to reflect the detail used (see Art. 11.10).

Transverse stiffeners should be proportioned so that

$$I \geq d_o t^3 J \tag{11.21}$$

$$J = 2.5\left(\frac{D}{d_o}\right)^2 - 2 \geq 0.5 \tag{11.22}$$

where I = moment of inertia, in^4, of transverse intermediate stiffener
J = ratio of rigidity of stiffener to web
d_o = actual distance, in, between transverse stiffeners
t = web thickness, in

For stiffener pairs, I should be taken about the center of the web. For single stiffeners, I should be taken about the web face in contact with the stiffeners. In either case, transverse stiffeners should project a distance, in, from the web of at least $b_f/4$, where b_f is the flange width, in, and at least $D'/30 + 2$, where D' is the girder depth, in. Thickness should be at least $\frac{1}{16}$ of this width.

Intermediate transverse stiffeners should have a gross cross-sectional area A, in^2, of at least

$$A = Y[0.15BDt_w(1 - C)(f_v/F_v) - 18t_w^2] \tag{11.23}$$

where Y = ratio of the yield strength of the web steel to the yield strength of the stiffener steel
t_w = web thickness, in
f_v = computed shear stress, ksi, in the web
F_v = allowable shear stress, ksi, in the web
B = 1.0 for pairs of stiffeners
= 1.8 for single angles
= 2.4 for single plates
C = ratio of buckling shear stress to yield shear stress
= 1.0 when $D/t_w < 190\sqrt{k/F_y}$ (11.24*a*)

$$= \frac{6000}{D/t_w}\sqrt{\frac{k}{F_y}} \qquad \text{when} \qquad 190\sqrt{k/F_y} \le D/t_w \le 237\sqrt{k/F_y} \tag{11.24b}$$

$$= \frac{45{,}000k}{(D/t_w)^2F_y} \qquad \text{when} \qquad D/t_w > 237\sqrt{k/F_y} \tag{11.24c}$$

$$k = 5[1 + (D/d_o)^2] \tag{11.24d}$$

When A computed from Eq. (11.23) is very small or negative, transverse stiffeners need only satisfy Eq. (11.21) and the width-thickness limitations given previously.

Intermediate transverse stiffeners, with or without longitudinal stiffeners, should be spaced close enough that the computed shear stress f'_v does not exceed

$$f'_v = F_v\left[C + \frac{0.87(1 - C)}{\sqrt{1 + (d_o/D)^2}}\right] \tag{11.25a}$$

where C is defined by Eqs. (11.24*a*) to (11.24*d*). Spacing is limited to a maximum of $3D$, or for panels without longitudinal stiffeners, to ensure efficient fabrication, handling, and erection of the girders, to $67{,}600D(t_w/D)^2$. At a simple support, the first intermediate stiffener should be close enough to the support that the shear stress in the end panel does not exceed

$$f'_v = CF_y/3 \le F_y/3 \tag{11.25b}$$

but not farther than $1.5D$.

If the shear stress is larger than $0.6F_v$ in a girder panel subjected to combined shear and bending moment, the bending stress F_s with live loads positioned for maximum moment at the section should not exceed

$$F_s = (0.754 - 0.34f_v/f'_v)F_y \tag{11.26}$$

Fabricators should be given leeway to vary stiffener spacing and web thickness to optimize costs. Girder webs often compose 40 to 50% of the girder weight but only about 10% of girder bending strength. Hence, least girder weight may be achieved with minimum web thickness and many stiffeners but not necessarily at the lowest cost. Thus, the contract drawings should allow fabricators the option of choosing stiffener spacing. The contract drawings should also note the thickness requirements for a web with a minimum number of stiffeners. (A stiffener is required at every cross frame.) This allows fabricators to choose the most economical fabrication process. If desired, flange thicknesses can be reduced slightly if the thicker-web option is selected. In some cases, the most economical results may be obtained with a stiffened web having a thickness 1⁄16 in less than that of an unstiffened web (Art. 11.17).

Preferably, the drawings should show the details for a range from unstiffened to fully stiffened webs. During the design stage, this is a relatively simple task. In contrast, after a construction contract has been awarded, the contractor cannot be expected to submit alternative girder designs, with or without value engineering, because it is often more trouble than the effort is worth. Contractors generally bid on what is shown on the plans, risking the possibility of losing the contract to a concrete alternative or to another contractor. On the other hand, by providing contract documents with sufficient flexibility, owners can profit from the fact that different fabricators have different methods of cost-effective fabrication that can be utilized on behalf of owners.

Longitudinal stiffeners should be used where D/t exceeds the values given in Table 11.25. They are required, even if the girder has transverse stiffeners, if the values of D/t for a web with transverse stiffeners is exceeded.

The optimum distance, d_s, of a plate longitudinal stiffener from the inner surface of the compression flange is $D/5$ for a symmetrical girder. The optimum distance, d_s, for an unsymmetrical composite girder in positive-moment regions may be determined from

$$\frac{d_s}{D_{cs}} = \frac{1}{1 + 1.5\sqrt{\dfrac{f_{DL+LL}}{f_{DL}}}} \tag{11.27}$$

where D_{cs} is the depth of the web in compression of the non-composite steel beam or girder, f_{DL} is the non-composite dead-load stress in the compression flange, and f_{DL+LL} is the total non-composite and composite dead-load plus the composite live-load stress in the compression flange at the most highly stressed section of the web. The optimum distance, d_s, of the stiffener in negative-moment regions of composite sections is $2D_c/5$, where D_c is the depth of the web in compression of the composite section at the most highly stressed section of the web. The stiffener should be proportioned so that

$$I \geq Dt^3 \left[2.4 \left(\frac{d_o}{D}\right)^2 - 0.13\right] \tag{11.28a}$$

where I = moment of inertia, in^4, of longitudinal stiffener about edge in contact with web and d_o = actual distance, in, between transverse stiffeners. Width-thickness ratio of the longitudinal stiffener should not exceed

$$\frac{b_s}{t_s} = \frac{95.94}{\sqrt{F_y}} \tag{11.28b}$$

Bending stress in the stiffener should not exceed the allowable for the stiffener steel. The stiffener may be placed on only one side of the web. Not required to be continuous, it may be interrupted at transverse stiffeners.

Spacing of transverse stiffeners used with longitudinal stiffeners should satisfy Eq. (11.25*a*) but should not exceed 1.5 times the subpanel depth in the panel adjacent to a simple support as well as in interior panels. The limit on stiffener spacing given previously to ensure efficient handling of girders does not apply when longitudinal stiffeners are used. Also, in computation of required moment of inertia and area of transverse stiffeners from Eqs. (11.21) to (11.23), the maximum sub-panel depth should be substituted for D.

Longitudinal stiffeners become economical for girder spans over 300 ft. Often, however, they are placed on fascia girders for esthetic reasons and may be used on portions of girders subject to tensile stresses or stress reversals. If this happens, designers should ensure that butt splices used by the fabricators for the longitudinal stiffeners are made with complete-penetration groove welds of top quality. (Plates of the sizes used for stiffeners are called *bar stock* and are available in limited lengths, which almost always make groove-welded splices necessary.) Many adverse in-service conditions have resulted from use of partial-penetration groove welds instead of complete-penetration.

11.12.5 Lateral Bracing

In highway girder bridges, AASHTO requires that the need for lateral bracing be investigated. The stresses induced in the flanges by the specified wind pressure must be within specified limits. In many cases lateral bracing will not be required, and a better structure can be achieved by eliminating fatigue prone details. Flanges attached to concrete decks or other decks of comparable rigidity will not require lateral bracing. When lateral bracing is required, it should be placed in the exterior bays between diaphragms or cross-frames, in or near the plane of the flange being braced.

Bracing consists of members capable of preventing rotation or lateral deformation of other members. This function may be served in some cases by main members, such as floorbeams where they frame into girders; in other cases by secondary members especially incorporated in the steel framing for the purpose; and in still other cases by other construction, such as a concrete deck. Preferably, bracing should transmit forces received to foundations or bearings, or to other members that will do so.

AASHTO specifications state that the smallest angle used in bracing should be 3 × 2½ in. Size of bracing often is governed by the maximum permissible slenderness ratio (Table 11.24) or width-thickness ratio of components (Table 11.25). Some designers prefer to design bracing for a percentage, often 2%, of the axial force in the member.

Through-truss, deck-truss, and spandrel-braced-arch highway bridges should have top and bottom lateral bracing (Fig. 11.8). For compression chords, lateral bracing preferably should be as deep as the chords and connected to top and bottom flanges.

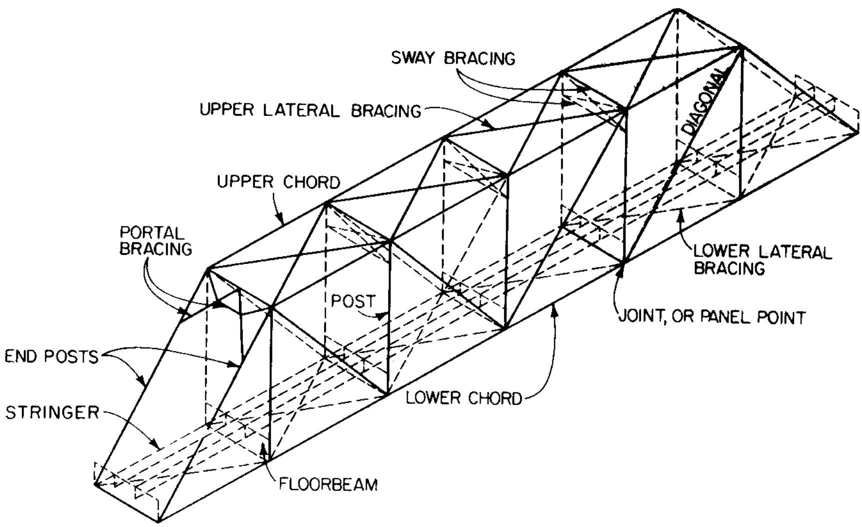


FIGURE 11.8 Components of a through-truss bridge.

If a double system of bracing is used (top and bottom laterals), both systems may be considered effective simultaneously if the members meet the requirements as both tension and compression members. The members should be connected at their intersections.

AASHTO ASD and LFD specifications require that a horizontal wind force of 50 lb/ft^2 on the area of the superstructure exposed in elevation be included in determining the need for, or in designing, bracing. Half of the force should be applied in the plane of each flange. The maximum induced stresses F, ksi, in the bottom flange from the lateral forces can be computed from

$$F = RF_{cb} \tag{11.29a}$$

where $R = (0.2272L - 11)/S_d^{3/2}$ without bottom lateral bracing
$= (0.059L - 0.640)/\sqrt{S_d}$ with bottom lateral bracing
L = span, ft
S_d = diaphragm or cross frame spacing, ft
$F_{cb} = 72M_{cb}/t_f b_f^2$
$M_{cb} = 0.08WS_d^2$
W = wind loading, kips per ft, along exterior flange
t_f = flange thickness, in
b_f = flange width, in

11.12.6 Cross Frames and Diaphragms for Deck Spans

In highway bridges, rolled beams and plate girders should be braced with cross frames or diaphragms at each end. Also, AASHTO specifications for ASD and LFD require that intermediate cross frames or diaphragms be spaced at intervals of 25 ft or less. They should be placed in all bays. Cross frames should be as deep as practicable. Diaphragms should be at least one-third and preferably one-half the girder depth. Cross frames and diaphragms should be designed for wind forces as described above for lateral bracing. The maximum horizontal force in the cross frames or diaphragms may be computed from

$$F_c = 1.14WS_d \tag{11.29b}$$

End cross frames or diaphragms should be designed to transmit all lateral forces to the bearings. Cross frames between horizontally curved girders should be designed as main members capable of transferring lateral forces from the girder flanges.

Although AASHTO specifications for ASD and LFD require cross frames or diaphragms at intervals of 25 ft or less, it is questionable whether spacing that close is necessary for bridges in service. Often, a three-dimensional finite-element analysis will show that few, if any, cross frames or diaphragms are necessary. Inasmuch as most fatigue-related damage to steel bridge is a direct result of out-of-plane forces induced through cross frames, the possibility of eliminating them should be investigated for all new bridges. However, although cross frames may not be needed for service loads, they may be necessary to ensure stability during girder erection and deck placement.

The AASHTO LRFD specifications do not require cross frames or diaphragms but specify that the need for diaphragms or cross frames should be investigated for all stages of assumed construction procedures and the final condition. Diaphragms or cross frames required for conditions other than the final condition may be specified to be temporary bracing. If permanent cross frames or diaphragms are included in the structural model used to determine force effects, they should be designed for all applicable limit states for the calculated member loads.

For plate girders, stiffeners used as cross-frame connection stiffeners should be connected to both flanges to prevent distortion-induced fatigue cracking. Although many designers believe welding stiffeners to the tension flange is worse than leaving the connection stiffener unattached, experience has proven otherwise. Virtually no cracks result from the attachment weld, but a proliferation of cracks develop when connection stiffeners are not connected to the tension flange. The LRFD specifications also recommend that, where cross frames are used, the attachment be designed for a transverse force of 20 kips (Fig. 11.9). This applies to straight, nonskewed bridges when better information is not available.

11.12.7 Portal and Sway Bracing

End panels of simply supported, through-truss bridges have compression chords that slope to meet the bottom chords just above the bearings. Bracing between corresponding sloping chords of a pair of main trusses is called portal bracing (Fig. 12.8). Bracing between corresponding vertical posts of a pair of main trusses is called sway bracing (Fig. 11.8).

All through-truss bridges should have portal bracing, made as deep as clearance permits. Portal bracing preferably should be of the two-plane or box type, rigidly connected to the flanges of the end posts (sloping chords). If single-plane portal bracing is used, it should be set in the central transverse plane of the end posts. Diaphragms then should be placed between the webs of the end posts, to distribute the portal stresses.

Portal bracing should be designed to carry the end reaction of the top lateral system. End posts should be designed to transfer this reaction to the truss bearings.

Through trusses should have sway bracing at least 5 ft deep in highway bridges at each intermediate panel point. Top lateral struts should be at least as deep as the top chord.

Deck trusses should have sway bracing between all corresponding panel points. This bracing should extend the full depth of the trusses below the floor system. End sway bracing should be designed to carry the top lateral forces to the supports through the truss end posts.

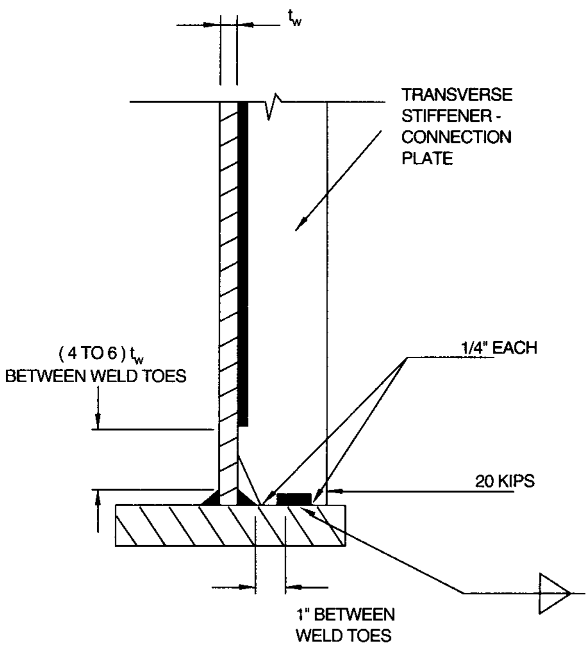


FIGURE 11.9 Girder connects to a cross frame through a transverse stiffener.

11.12.8 Bracing of Towers

Towers should be braced with double systems of diagonals and with horizontal struts at caps, bases, and intermediate panel points. Sections of members of longitudinal bracing in each panel should not be less than those of members in corresponding panels of the transverse bracing.

Column splices should be at or just above panel points. Bracing of a long column should fix the column about both axes at or near the same point.

Horizontal diagonal bracing should be placed, at alternate intermediate panel points, in all towers with more than two vertical panels. In double-track towers, horizontal bracing should be installed at the top to transmit horizontal forces.

Bottom struts of towers should be strong enough to slide the movable shoes with the structure unloaded, when the coefficient of friction is 0.25. Column bearings should be designed for expansion and contraction of the tower bracing.

11.13 CRITERIA FOR BUILT-UP TENSION MEMBERS

A tension member and all its components must be proportioned to meet the requirements for maximum slenderness ratio given in Table 11.24. The member also must be designed to ensure that the allowable tensile stress on the net section is not exceeded.

The **net section** of a high-strength-bolted tension member is the sum of the net sections of its components. The net section of a component is the product of its thickness and net width.

Net width is the minimum width normal to the stress minus an allowance for holes. The diameter of a hole for a fastener should be taken as ⅛ in greater than the nominal fastener diameter. The chain of holes that is critical is the one that requires the largest deduction for holes and may lie on a straight line or in a zigzag pattern. The deduction for any chain of holes equals the sum of the diameters of all the holes in the chain less, for each gage space in the chain, $s^2/4g$, where s is the pitch, in, of any two successive holes and g is the gage, in, of those holes.

For angles, the gross width should be taken as the sum of the widths of the legs less the thickness. The gage for holes in opposite legs is the sum of the gages from back of angle less the thickness. If a double angle or tee is connected with the angles or flanges back to back on opposite sides of a gusset plate, the full net section may be considered effective. But if double angles, or a single angle or tee, are connected on the same side of a gusset plate, the effective area should be taken as the net section of the connected leg or flange plus one-half the area of the outstanding leg. When angles connect to separate gusset plates, as in a double-webbed truss, and the angles are interconnected close to the gussets, for example, with stay plates, the full net area may be considered effective. Without such interconnection, only 80% of the net area may be taken as effective.

For built-up tension members with perforated plates, the net section of the plate through the perforation may be considered the effective area.

In pin-connected tension members other than eyebars, the net section across the pinhole should be at least 140%, and the net section back of the pinhole at least 100% of the required net section of the body of the member. The ratio of the net width, through the pinhole normal to the axis of the member, to thickness should be 8 or less. Flanges not bearing on the pin should not be considered in the net section across the pin.

To meet stress requirements, the section at pinholes may have to be reinforced with plates. These should be arranged to keep eccentricity to a minimum. One plate on each side should be as wide as the outstanding flanges will allow. At least one full-width plate on each segment should extend to the far side of the stay plate and the others at least 6 in beyond the near edge. These plates should be connected with fasteners or welds arranged to distribute the bearing pressure uniformly over the full section.

Eyebars should have constant thickness, no reinforcement at pinholes. Thickness should be between ½ and 2 in, but not less than ⅛ the width. The section across the center of the pinhole should be at least 135%, and the net section back of the pinhole at least 75% of the required net section of the body of the bar. The width of the body should not exceed the pin diameter divided by $3/4 + F_y/400$, where F_y is the steel yield strength, ksi. The radius of transition between head and body of eyebar should be equal to or greater than the width of the head through the center of the pinhole.

Eyebars of a set should be symmetrical about the central plane of the truss and as nearly parallel and close together as practicable. But adjacent bars in the same panel should be at least ½ in apart. The bars should be held against lateral movement.

Stitching. In built-up members, welds connecting plates in contact should be continuous. Spacing of fasteners should be the smaller of that required for sealing, to prevent penetration of moisture (Art. 5.11), or stitching, to ensure uniform action. The pitch of stitch fasteners on any single line in the direction of stress should not exceed $24t$, where t = thickness, in, of the thinner outside plate or shape. If there are two or more lines of fasteners with staggered pattern, and the gage g, in, between the line under consideration and the farther adjacent line is less than $24t$, the staggered pitch in the two lines, considered together, should not exceed $24t$ or $30t - 3g/4$. The gage between adjacent lines of stitch fasteners should not exceed $24t$.

Cover Plates. When main components of a tension member are tied together with cover plates, the shear normal to the member in the planes of the plates should be assumed equally divided between the parallel plates. The shearing force should include that due to the weight of the member plus other external forces.

When perforated cover plates are used, the openings should be ovaloid or elliptical (minimum radius of periphery 1½ in). Length of perforation should not exceed twice its width. Clear distance between perforations in the direction of stress should not be less than the distance l between the nearer lines of connections of the plate to the member. The clear distance between the end perforation and end of the cover plate should be at least $1.25l$. For plates groove-welded to the flange edge of rolled components, l may be taken as the distance between welds when the width-thickness ratio of the flange projection is less than 7; otherwise, the distance l should be taken between the roots of the flanges. Thickness of a perforated plate should be at least 1⁄50 of the distance between nearer lines of connection.

When stay plates are used to tie components together, the clear distance between them should be 3 ft or less. Length of end stay plates between end fasteners should be at least $1.25l$, and length of intermediate stay plates at least $0.563l$. Thickness of stay plates should not be less than $l/50$ in main members and $l/60$ in bracing. They should be connected by at least three fasteners on each side to the other components. If a continuous fillet weld is used, it should be at least 5⁄16 in.

Tension-member components also may be tied together with end stay plates and lacing bars like compression members. The last fastener in the stay plates preferably should also pass through the end of the adjacent bar.

11.14 CRITERIA FOR BUILT-UP COMPRESSION MEMBERS

Compression members should be designed so that main components are connected directly to gusset plates, pins, or other members. Stresses should not exceed the allowable for the gross section. The radius of gyration and the effective area of a member with perforated cover plates should be computed for a transverse section through the maximum width of perforation. When perforations are staggered in opposite cover plates, the effective area should be considered the same as for a section with perforations in the same transverse plane.

Solid-Rib Arches. A compression member and all its components must be proportioned to meet the requirements for maximum slenderness ratio in Table 11.24. The member also must satisfy width-thickness requirements (Table 11.25). In addition, for solid-rib arches, longitudinal stiffeners are required when the depth-thickness ratio of each web exceeds

$$\frac{D}{t} = \frac{158}{\sqrt{f_a}} \leq 60 \tag{11.30}$$

where D = unsupported distance, in, between flange components
t = web thickness, in
f_a = maximum compressive stress in web, ksi

If one longitudinal stiffener is used, it should have a moment of inertia I_s, in^4, of at least

$$I_s = 0.75Dt_w^3 \tag{11.31}$$

where D = clear unsupported depth of web, in, and t_w = web thickness, in. If the stiffener is placed at middepth of the web, the width-thickness ratio should not exceed

$$D/t_w = 237/\sqrt{f_a} \tag{11.32}$$

If two longitudinal stiffeners are used, each should have a moment of inertia of at least

$$I_s = 2.2Dt_w^3 \tag{11.33}$$

If the stiffeners are placed at the third points of the web depth, the width-thickness ratio should not exceed

$$D/t_w = 316/\sqrt{f_a} \tag{11.34}$$

Maximum width-thickness ratio for an outstanding element of a stiffener is given by

$$\frac{b'}{t_s} = \frac{51.4}{\sqrt{f_a + f_b/3}} \leq 12 \tag{11.35}$$

where b' = width of outstanding element, in
t_s = thickness of the element, in
f_b = maximum compressive bending stress, ksi

The preceding relationships for webs applies when

$$0.2 \leq f_b/(f_b + f_a) \leq 0.7 \tag{11.36}$$

For flange plates between the webs of a solid-rib arch, the width-thickness ratio should not exceed

$$\frac{b_f}{t_f} = \frac{134}{\sqrt{f_a + f_b}} \leq 47 \tag{11.37}$$

Maximum width-thickness ratio for the overhang of flange plates is given by

$$\frac{b'_f}{t_f} = \frac{51.4}{\sqrt{f_a + f_b}} \leq 12 \tag{11.38}$$

Stitching. In built-up members, welds connecting plates in contact should be continuous. Spacing of fasteners should be the smaller of that required for sealing, to prevent penetration of moisture (Art. 5.11), or stitching, to ensure uniform action and prevent local buckling. The pitch of stitch fasteners on any single line in the direction of stress should not exceed $12t$, where t = thickness, in, of the thinner outside plate or shape. If there are two or more lines of fasteners with staggered pattern, and the gage g, in, between the line under consideration and the farther adjacent line is less than $24t$, the staggered pitch in the two lines, considered together, should not exceed $12t$ or $15t - 3g/8$. The gage between adjacent lines of stitch fasteners should not exceed $24t$.

Fastener Pitch at Ends. Pitch of fasteners connecting components of a compression member over a length equal to 1.5 times the maximum width of member should not exceed 4 times the fastener diameter. The pitch should be increased gradually over an equal distance farther from the end.

Shear. On the open sides of compression members, components should be connected with perforated plates or by lacing bars and end stay plates. The shear normal to the member in the planes of the plates or bars should be assumed equally divided between the parallel planes. The shearing force should include that due to the weight of the member, other external forces, and a normal shearing force, kips, given by

$$V = \frac{P}{100}\left(\frac{100}{L/r + 10} + \frac{L/r}{3{,}300/F_y}\right) \tag{11.39}$$

where P = allowable compressive axial load on member, kips
L = length of member, in
r = radius of gyration, in, of section about axis normal to plane of lacing or perforated plate

Perforated Plates. When perforated cover plates are used, the openings should be ovaloid or elliptical (minimum radius of periphery 1½ in). Length of perforation should not exceed twice its width. Clear distance between perforations in the direction of stress should not be less than the distance l between the nearer lines of connections of the plate to the member. The clear distance between the end perforation and end of the cover plate should be at least $1.25l$. For plates groove-welded to the flange edge of rolled components, l may be taken as the distance between welds when the width-thickness ratio of the flange projection is less than 7; otherwise, the distance l should be taken between the roots of the flanges. Thickness should meet the requirements for perforated plates given in Table 11.25.

11.15 PLATE GIRDERS AND COVER-PLATED ROLLED BEAMS

Where longitudinal beams or girders support through bridges, the spans preferably should have two main members. They should be placed sufficiently far apart to prevent overturning by lateral forces.

Spans. For calculation of stresses, span is the distance between center of bearings or other points of support. For computing span-depth ratio for continuous beams, span should be taken as the distance between dead-load points of inflection.

Allowable-Stress Design. Beams and plate girders should be proportioned by the moment-of-inertia method; that is, for pure bending, to satisfy the flexure formula:

$$\frac{I}{c} \geq \frac{M}{F_b} \tag{11.40}$$

where I = moment of inertia, in^4, of gross section for compressive stress and of net section for tensile stress
c = distance, in, from neutral axis to outermost surface
M = bending moment at section, in kips
F_b = allowable bending stress, ksi

The neutral axis should be taken along the center of gravity of the gross section. For computing the moment of inertia of the net section, the area of holes for high-strength bolts in excess of 15% of the flange area should be deducted from the gross area.

Span-Depth Ratio. Depth of steel beams or girders for highway bridges should preferably be at least 1⁄25 of the span.

For bracing requirements, see Art. 11.14.

Cover-Plated Rolled Beams. Welds connecting a cover plate to a flange should be continuous and capable of transmitting the horizontal shear at any point. When the unit shear in the web of a rolled beam at a bearing exceeds 75% of the allowable shear for girder webs, bearing stiffeners should be provided to reinforce the web. They should be designed to satisfy the same requirements as bearing stiffeners for girders in Art. 11.12.

The theoretical end of a cover plate is the section at which the stress in the flange without that cover plate equals the allowable stress, exclusive of fatigue considerations. **Terminal distance,** or extension of cover plate beyond the theoretical end, is twice the nominal cover-plate width for plates not welded across their ends and 1.5 times the width for plates welded across their ends. Length of a cover plate should be at least twice the beam depth plus 3 ft. Thickness should not exceed twice the flange thickness.

Partial-length welded cover plates should extend beyond the theoretical end at least the terminal distance or a sufficient distance so that the stress range in the flange equals the allowable fatigue stress range for base metal at fillet welds, whichever is greater. Ends of tapered cover plates should be at least 3 in wide. Welds connecting a cover plate to a flange within the terminal distance should be of sufficient size to develop the computed stress in the cover plate at its theoretical end.

Because of their low fatigue strength, cover-plated beams are seldom cost-effective.

Girder Flanges. Width-thickness ratios of compression flanges of plate girders should meet the requirements given in Art. 11.12. For other girders, see Arts. 11.16, 11.18, and 11.19.

Each flange of a welded plate girder should consist of only one plate. To change size, plates of different thicknesses and widths may be joined end to end with complete-penetration groove welds and appropriate transitions (Art. 5.26).

Plate girders composed of flange angles, web plate, and cover plates attached with bolts or rivets are no longer used. In existing bolted girders, flange angles formed as large a part of the flange area as practicable. Side plates were used only where flange angles more than 7⁄8 in thick would otherwise be required. Except in composite design, the gross area of the compression flange could not be less than the gross area of the tension flange.

When cover plates were needed, at least one cover plate of the top flange extended full length of the girder unless the flange was covered with concrete. If more than one cover plate was desirable, the plates on each flange were made about the same thickness. When of unequal thickness, they were arranged so that they decreased in thickness from flange angles outward. No plate could be thicker than the flange angles. Fasteners connecting cover plates and flange were required to be adequate to transmit the horizontal shear at any point. Cover plates over 14 in wide should have four lines of fasteners.

Partial-length cover plates extended beyond the theoretical end far enough to develop the plate capacity or to reach a section where the stress in the remainder of the flange and cover plates equals the allowable fatigue stress range, whichever distance is greater.

Flange-to-Web Connections. Welds or fasteners for connecting the flange of a plate girder to the web should be adequate to transmit the horizontal shear at any point plus any load applied directly to the flange. AASHTO permits the web to be connected to each flange with a pair of fillet welds.

For flange splices, see Arts. 5.26 and 5.27.

Girder Web and Stiffeners. The web should be proportioned so that the average shear stress over the gross section does not exceed the allowable. In addition, depth-thickness ratio should meet the requirements of Art. 11.14. Also, stiffeners should be provided, where needed, in accordance with those requirements. For web splices, see Arts. 5.26, 5.27, and 5.30.

Camber. Girders should be cambered to compensate for dead-load deflection. Also, on vertical curves, camber preferably should be increased or decreased to keep the flanges parallel to the profile grade line.

See also Art. 11.17.

11.16 COMPOSITE CONSTRUCTION WITH I GIRDERS

With shear connectors welded to the top flange of a beam or girder, a concrete slab may be made to work with that member in carrying bending stresses. In effect, a portion of the slab, called the **effective width,** functions much like a steel cover plate. In fact, the effective slab area may be transformed into an equivalent steel area for computation of composite-girder stresses and deflection. This is done by dividing the effective concrete area by the modular ratio n, the ratio of modulus of elasticity of steel, 29,000 ksi, to modulus of elasticity of the concrete. The equivalent area is assumed to act at the center of gravity of the effective slab. The equivalent steel section is called the **transformed section.**

Allowable-Stress Design. Composite girders, in general, should meet the requirements of plate girders (Art. 11.15). Bending stresses in the steel girder alone and in the transformed section may be computed by the moment-of-inertia method, as indicated in Art. 11.15, or by load-factor design, and should not exceed the allowable for the material. The stress range at the shear connector must not exceed the allowable for a Category C detail.

The allowable concrete stress may be taken as $0.4f'_c$, where f'_c = unit ultimate compressive strength of concrete, psi, as determined by tests of 28-day-old cylinders. The allowable tensile stress of steel reinforcement for concrete should be taken as 20 ksi for A615 Grade 40 steel bars and 24 ksi for A615 Grade 60 steel bars. The modular ratio n may be assumed as follows:

f'_c	n
2,000–2,300	11
2,400–2,800	10
2,900–3,500	9
3,600–4,500	8
4,600–5,900	7
6,000 or more	6

To account for creep of the concrete under dead load, design of the composite section should include the larger of the dead-load stresses when the transformed section is determined with n or $3n$.

The neutral axis of the composite section preferably should lie below the top flange of the steel section. Concrete on the tension side should be ignored in stress computations.

Effective Slab Width. The assumed effective width of slab should be equal to or less than one-quarter the span, distance center to center of girders, and 12 times the least slab thickness (Fig. 11.10). For exterior girders, the effective width on the exterior side should not exceed the actual overhang. When an exterior girder has a slab on one side only, the assumed effective width should be equal to or less than one-twelfth the span, half the distance to the next girder, and 6 times the least slab thickness (Fig. 11.10).

Span-Depth Ratios. For composite highway girders, depth of steel girder alone should preferably be at least 1/30 of the span. Depth from top of concrete slab to bottom of bottom flange should preferably be at least 1/25 of the span. For continuous girders, spans for this purpose should be taken as the distance between dead-load inflection points.

Girder Web and Stiffeners. The steel web should be proportioned so that the average shear stress over the gross section does not exceed the allowable. The effects of the steel flanges and concrete slab should be ignored. In addition, depth-thickness ratio should meet the requirements of Art. 11.12. Also, stiffeners should be provided, where needed, in accordance with those requirements. For web splices, see Arts. 5.26, 5.27, and 5.30.

Bending Stresses. If, during erection, the steel girder is supported at intermediate points until the concrete slab has attained 75% of its required 28-day strength, the composite section may be assumed to carry the full dead load and all subsequent loads. When such shoring is not used, the steel girder alone must carry the steel and concrete dead loads. The composite section will support all loads subsequently applied. Thus, maximum bending stress in the steel of an unshored girder equals the sum of the dead-load stress in the girder alone plus stresses produced by loads on the composite section. Maximum bending stress in the concrete equals the stresses produced by those loads on the composite section at its top surface.

The positive-moment portion of continuous composite-girder spans should be designed in the same way as for simple spans. The negative-moment region need not be designed for composite action, in which case shear connectors need not be installed there. But additional connectors should be placed in the region of the dead-load inflection point as indicated later. If composite action is desired in the negative-moment portion, shear connectors should be

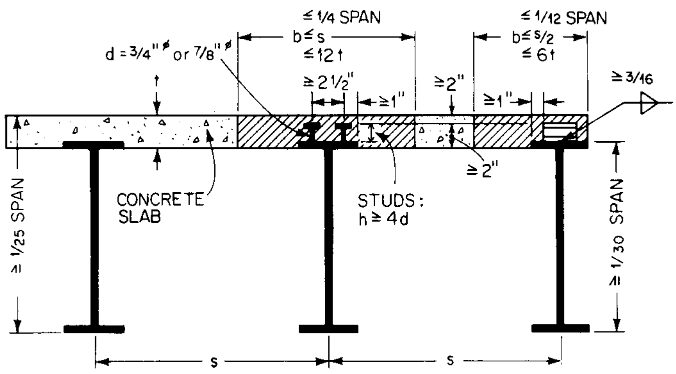


FIGURE 11.10 Effective width of concrete slab for composite construction.

installed. Then, longitudinal steel reinforcement in the concrete should be provided to carry the full tensile force. The concrete should be assumed to carry no tension.

Shear Connectors. To ensure composite action, shear connectors must be capable of resisting both horizontal and vertical movements between concrete and steel. They should permit thorough compaction of the concrete so that their entire surfaces are in contact with the concrete. Usually, headed steel studs or channels, welded to the top flange of the girder, are used.

Channels should be attached transverse to the girder axis, with fillet welds at least along heel and toe. Minimum weld size permitted for this purpose is 3⁄16 in.

Studs should be 3⁄4- or 7⁄8-in nominal diameter. Overall length after welding should be at least 4 times the diameter. Steel should be A108, Grades 1015, 1018, or 1020, either fully or semikilled. The studs should be end-welded to the flange with automatically timed equipment. If a 360° weld is not obtained, the interrupted area may be repaired with a 3⁄16-in fillet weld made by low-hydrogen electrodes in the shielded metal-arc process. Usually, two or more studs are installed at specific sections of a composite girder, at least four stud diameters c to c.

Clear depth of concrete cover over the tops of shear connectors should be at least 2 in. In addition, connectors should penetrate at least 2 in above the bottom of the slab. Clear distance between a flange edge and a shear-connector edge should not be less than 1 in in highway bridges, 1½ in in railroad bridges.

Pitch of Shear Connectors. In general, shear connectors should not be spaced more than 24 in c to c along the span. Over interior supports of continuous beams, however, wider spacing may be used to avoid installation of connectors at points of high tensile stress.

Pitch may be determined by fatigue shear stresses due to change in horizontal shear or by ultimate-strength requirements for resisting total horizontal shear, whichever requires the smaller spacing. (Also, see the following method for stress design.)

Fatigue. As live loads move across a bridge, the vertical shear at any point in a girder changes. For some position of the loading, vertical shear at the point due to live load plus impact reaches a maximum. For another position, shear there due to live load plus impact becomes a minimum, which may be opposite in sign to the maximum. The algebraic difference between maximum and minimum shear, kips, is the range of shear V_r.

The range of horizontal shear, kips per lin in, at the junction of a slab and girder at the point may be computed from

$$S_r = \frac{V_r Q}{I} \tag{11.41}$$

where Q = statical moment, in^3, about the neutral axis of the composite section, of the transformed compressive concrete area, or for negative bending moment, of the area of steel reinforcement in the concrete

I = moment of inertia, in^4, about the neutral axis, of the transformed composite girder in positive-moment regions, and in negative-moment regions, the moment of inertia, in^4, about the neutral axis, of the girder and concrete reinforcement if the girder is designed for composite action there, or without the reinforcement if the girder is non-composite there

The allowable range of shear, kips per connector, is

FOR CHANNELS:

$$Z_r = Bw \tag{11.42}$$

FOR WELDED STUDS:

$$Z_r = \alpha d^2 \quad \left(\frac{h}{d} \geq 4\right) \tag{11.43}$$

where w = transverse length of channel, in
d = stud diameter, in
h = overall stud height, in
B = 4 for 100,000 cycles of maximum stress
= 3 for 500,000 cycles
= 2.4 for 2,000,000 cycles
= 2.1 for more than 2,000,000 cycles
α = 13 for 100,000 cycles of maximum stress
= 10.6 for 500,000 cycles
= 7.85 for 2,000,000 cycles
= 5.50 for more than 2,000,000 cycles

The required pitch p_r, in, of shear connectors for fatigue is obtained from

$$p_r = \frac{\Sigma Z_r}{S_r} \tag{11.44}$$

where ΣZ_r is the allowable range of horizontal shear of all connectors at a cross section. Over interior supports of continuous beams, the pitch may be modified to avoid installation of connectors at points of high tensile stress. But the total number of connectors should not be decreased.

Ultimate Strength. The total number of connectors provided for fatigue, in accordance with Eq. (11.44), should be checked for adequacy at ultimate strength under dead load plus live load and impact. The connectors must be capable of resisting the horizontal forces H, kips, in positive-moment regions and in negative- moment regions. Thus, at points of maximum moment, H may be taken as the smaller of the values given by Eqs. (11.45) and (11.46).

$$H_1 = A_s F_y \tag{11.45}$$

$$H_2 = 0.85 f'_c bt \tag{11.46}$$

where A_s = cross-sectional area of steel girder, in^2
F_y = steel yield strength, ksi
f'_c = 28-day compressive strength of concrete, ksi
b = effective width of concrete slab, in
t = slab thickness, in

At points of maximum negative moment, H should be taken as

$$H_3 = A_{rs} F_{ry} \tag{11.47}$$

where A_{rs} = total area of longitudinal reinforcing steel at interior support within effective slab width, in^2, and F_{ry} = yield strength, ksi, of reinforcing steel. The total number of shear connectors required in any region then is

$$N = \frac{1{,}000H}{\phi Q_u} \tag{11.48}$$

where Q_u = ultimate strength of shear connector, lb, and ϕ = reduction factor, 0.85. In Eq. (11.48), the smaller of H_1 or H_2 should be used for H for determining the number of connectors required between a point of maximum positive moment and an end support in simple beams, and between a point of maximum positive moment and a dead-load inflection point in continuous beams. H_3 should be used for H for determining the total number of shear connectors required between a point of maximum negative moment and a dead-load inflection point in continuous beams. $H_3 = 0$ if slab reinforcement is not used in the computation of section properties for negative moment.

FOR CHANNELS:
$$Q_u = 550\left(t_f + \frac{t_w}{2}\right) l\sqrt{f'_c} \tag{11.49}$$

FOR WELDED STUDS:
$$Q_u = 0.4d^2\sqrt{f'_c E_c} \quad \left(\frac{h}{d} > 4\right) \tag{11.50}$$

where E_c = modulus of the concrete, psi = $33w^{3/2}\sqrt{f'_c}$
t_f = average thickness of channel flange, in
t_w = thickness of channel web, in
l = length of channel, in
f'_c = 28-day strength of concrete, psi
w = weight of the concrete, lb/ft^3
d = stud diameter, in
h = stud height, in

Additional Connectors at Inflection Points. In continuous beams, the positive-moment region under live loads may extend beyond the dead-load inflection points, and additional shear connectors are required in the vicinity of those points when longitudinal reinforcing steel in the concrete slab is not used in computing section properties. The number needed is given by

$$N_c = A_{rs}\frac{f_r}{Z_r} \tag{11.51}$$

where A_{rs} = total area, in^2, of longitudinal reinforcement at interior support within effective slab width
f_r = range of stress, ksi, due to live load plus impact in slab reinforcement over support (10 ksi may be used in the absence of accurate computations)
Z_r = allowable range, kips, of shear per connector, as given by Eqs. (11.42) and (11.43)

This number should be placed on either side of or centered about the inflection point for which it is computed, within a distance of one-third the effective slab width.

11.17 COST-EFFECTIVE PLATE-GIRDER DESIGNS

To get cost-effective results from the many different designs of fabricated girders that can satisfy the requirements of specifications, designers should obtain advice from fabricators and contractors whenever possible. Also useful are steel-industry-developed rules-of-thumb intended to help designers. The following recommendations, modified to reflect current trends, should be considered for all designs.

1. Load-and-resistance factor design (LRFD) is the preferred design procedure. Load-factor design (LFD) yields more economical girder designs than does allowable-stress design (ASD).
2. Properly designed for their environment, unpainted weathering-steel bridges are more economical in the long run than those requiring painting. Consider the following grades of weathering steels: ASTM A709 grade 50W, 70W, HPS70W, or 100W. Grade 50W is the most often used.

3. The most economical painted design is that for hybrid girders, using 36-ksi and 50-ksi steels. Painted homogenous girders of 50-ksi steel are a close second. The most economical design with high performance steel (HPS) will also be hybrid, utilizing grade 50W steel for all stiffeners, diaphragm members, and web and flanges, where grade 70W strength is not required. Rolled sections (angles, channels, etc.) are not available in HPS grades.
4. The fewer the girders, the greater the economy. Girder spacing must be compatible with deck design, but sometimes other factors govern selection of girder spacing. For economy, girder spacing should be 10 ft or more.
5. Transverse web stiffeners, except those serving as diaphragm or cross-frame connections, should be placed on only one side of a web.
6. Web depth may be several inches larger or smaller than the optimum without significant cost penalty.
7. A plate girder with a nominally stiffened web—$\frac{1}{16}$ in thinner than an unstiffened web—will be the least costly or very close to it. (Unstiffened webs are generally the most cost-effective for web depths less than 52 in. Nominally stiffened webs are most economical in the 52- to 72-in range. For greater depths, fully stiffened webs may be the most cost-effective.)
8. Web thickness should be changed only where splices occur. (Use standard-plate-thickness increments of $\frac{1}{16}$ in for plates up to 2 in thick and $\frac{1}{8}$-in increments for plates over 2 in thick.)
9. Longitudinal stiffeners should be considered for plate girders only for spans over 300 ft.
10. Not more than three plates should be butt-spliced to form the flanges of field sections up to 130 ft long. In some cases, it is advisable to extend a single flange-plate size the full length of a field section.
11. To justify a welded flange splice, about 700 lb of flange steel would have to be eliminated. However, quenced-and-tempered plates are limited to 50 ft lengths.
12. A constant flange width should be used between flange field splices. (Flange widths should be selected in 1-in increments.)
13. For most conventional cross sections, haunched girders are not advantageous for spans under 400 ft.
14. Bottom lateral bracing should be omitted where permitted by AASHTO specifications. Omit intermediate cross frames where permitted by AASHTO (see LRFD Specification Art, 6.7.4) but indicate on the plans where temporary bracing will be required for girder stability during erection and deck placement. Space permanent intermediate cross frames, if required, at the maximum spacing consistent with final loading conditions.
15. Elastomeric bearings are preferable to custom-fabricated steel bearings.
16. Composite construction may be advantageous in negative-moment regions of composite girders.

Designers should bear in mind that such techniques as finite-element analysis, use of high-strength steels, and load-and-resistance-factor design often lead to better designs.

Consideration should be given to use of 40-in-deep and 42-in-deep rolled sections. These may be cost-effective alternatives to welded girders for spans up to 100 ft or longer. Economy with these beams may be improved with end-bolted cover-plate details that allow use of category B stress ranges (Art. 11.10). Contract documents that allow either rolled beams or welded girders ensure cost-effective alternatives for owners.

With fabricated girders, designers should ensure that flanges are wide enough to provide lateral stability for the girders during fabrication and erection. Flange width should be at least 12 in, but possibly even greater for deeper girders. The AISC recommends that, for shipping, handling, and erection, the ratio of length to width of compression flanges should be about 85 or less.

Designers also should avoid specifying thin flanges that make fabrication difficult. A thin flange is subject to excessive warping during welding of a web to the flange. To reduce warping, a flange should be at least ¾ in thick.

To minimize fabrication and deck forming costs when changes in the area of the top flange are required, the width should be held constant and required changes made by thickness transitions.

11.18 BOX GIRDERS (ASD)

Closed-section members, such as box girders, often are used in highway bridges because of their rigidity, economy, appearance, and resistance to corrosion. Box girders have high torsional rigidity. With their wide bottom flanges (Fig. 11.11), relatively shallow depths can be used economically. And for continuous box girders, intermediate supports often can be individual, slender columns simply connected to concealed cross frames.

While box girders may be multicell (with three or more webs), single-cell girders, as illustrated in Fig. 11.11, are generally preferred. For short spans, such girders can be entirely shop-fabricated, permitting assembly by welding under closely controlled and economical conditions. Longer spans often can be prefabricated to the extent that only one field splice is necessary. One single-cell girder can be used to support bridges with one or two traffic lanes. But usually, multiple boxes are used to carry two or more lanes to keep box width small enough to meet shipping-clearance requirements.

Through the use of shear connectors welded to the top flanges, a concrete deck can be made to work with the box girders in carrying bending stresses. In such cases the concrete may be considered part of the top flange, and the steel top flange need be only wide enough for erection and handling stability, load distribution to the web, and placement of required shear connectors (Fig. 11.11).

Composite box girders are designed much like plate girders (Arts. 11.15 and 11.16). Criteria that are different are summarized in the following. For distribution of live loads to box girders, see Art. 11.7.

Additional criteria apply to curved box-girder bridges.

Girder Spacing. The criteria are applicable to bridges with multiple single-cell box girders. Width center to center of top steel flanges in each girder should nearly equal the distance center to center between adjacent top steel flanges of adjacent boxes. (Width of boxes should nearly equal distance between boxes.) Cantilever overhang of deck, including curbs and parapets, should not exceed 6 ft or 60% of the distance between centers of adjacent top steel flanges of adjacent box girders.

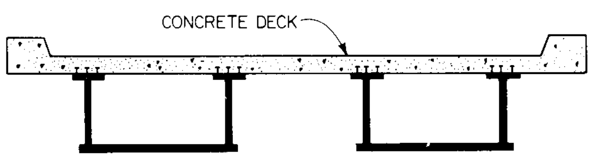


FIGURE 11.11 Composite construction with box girders.

Bracing. Diaphragms, cross frames, or other bracing should be provided within box girders at each support to resist transverse rotation, displacement, and distortion. Intermediate internal bracing for these purposes is not required if stability during concrete placement and curing has been otherwise ensured.

Lateral systems generally are not required between composite box girders. Need for a lateral system should be determined as follows: A horizontal load of 25 psf on the area of the girder exposed in elevation should be applied in the plane of the bottom flange. The resisting section should comprise the bottom flange serving as web, while portions of the box-girder webs, with width equal to 12 times their thickness, serve as flanges. A lateral system should be provided between bottom flanges if the combined stresses due to the 25-psf load and dead load of steel and deck exceed 150% of the allowable stress.

Access and Drainage. Manholes or other openings to the box interior should be provided for form removal, inspection, maintenance, drainage, or access to utilities.

Box-Girder Webs. Web plates may be vertical or inclined. A trapezoidal box generally requires a heavier bottom plate, and sometimes also a heavier concrete slab, but it may reduce the number of girders needed to support a deck. Design shear for an inclined web, kips, may be calculated from

$$V_w = \frac{V_v}{\cos\theta} \tag{11.52}$$

where V_v = vertical shear, kips, on web and θ = angle web makes with the vertical.

Transverse bending stresses due to distortion of the cross section and bottom-flange vibrations need not be considered if the web slope relative to the plane of the bottom flange is 4:1 or more and the bottom-flange width does not exceed 20% of the span. Furthermore, transverse bending stresses due to supplementary loadings, such as utilities, should not exceed 5 ksi. When any of the preceding limits are exceeded, transverse bending stresses due to all causes should be restricted to a maximum stress or stress range of 20 ksi.

Bottom Flange in Tension. Bending stress cannot be assumed uniformly distributed horizontally over very wide flanges. To simplify design, only a portion of such a flange should be considered effective, and the horizontal distribution of the bending stresses may be assumed uniform over that portion.

For simply supported girders, and between inflection points of continuous spans, the bottom flange may be considered completely effective if its width does not exceed one-fifth the span. For wider flanges, effective width equals one-fifth the span.

Unstiffened Compression Flanges. Compression flanges designed for the basic allowable stress of $0.55F_y$ need not be stiffened if the width-thickness ratio does not exceed

$$\frac{b}{t} = \frac{194}{\sqrt{F_y}} \tag{11.53}$$

where b = flange width between webs, in
t = flange thickness, in
F_y = steel yield strength for flange, ksi

When $194/\sqrt{F_y} < b/t \leq 420/\sqrt{F_y}$, but not more than 60, the stress in an unstiffened bottom flange, ksi, should not exceed

$$F_b = F_y \left(0.326 + 0.244 \sin \frac{c\pi}{2}\right) \tag{11.54}$$

$$c = \frac{420 - (b/t)\sqrt{F_y}}{226} \tag{11.55}$$

When b/t $420/\sqrt{F_y}$, the stress, ksi, in the flange should not exceed

$$F_b = \frac{57{,}600}{(b/t)^2} \tag{11.56}$$

b/t preferably should not exceed 60, except in areas of low stress near inflection points.

Longitudinally Stiffened Compression Flanges. When $b/t > 45$, use of longitudinal stiffeners should be considered. When used, they should be equally spaced across the compression flange. The number required depends heavily on the ratio of spacing to flange thickness.

For the flange, including the longitudinal stiffeners, to be designed for the basic allowable stress $0.55F_y$, this ratio should not exceed

$$\frac{w}{t} = \frac{97}{\sqrt{F_y/k}} \tag{11.57}$$

where w = width of flange, in, between longitudinal stiffeners or distance, in, from a web to nearest stiffener and k = buckling coefficient, which may be assumed to be between 2 and 4. For larger values of w/t, but not more than 60 or $210/\sqrt{F_y/k}$, the stress, ksi, in the flange should not exceed

$$F_b = F_y \left(0.326 + 0.224 \sin \frac{c'\pi}{2}\right) \tag{11.58}$$

$$c' = \frac{210 - (w/t)\sqrt{F_y/k}}{113} \tag{11.59}$$

When $210/\sqrt{F_y/k} < w/t \le 60$, the stress, ksi, should not exceed

$$F_b = \frac{14{,}400k}{(w/t)^2} \tag{11.60}$$

Stiffeners should be proportioned so that the depth-thickness ratio of any outstanding element does not exceed

$$\frac{d_s}{t_s} = \frac{82.2}{\sqrt{F_y}} \tag{11.61}$$

where d_s = depth, in, of outstanding element and t_s = thickness, in, of element. The moment of inertia, in^4, of each longitudinal stiffener about an axis through the base of the stiffener and parallel to the flange should be at least

$$I_s = \phi w t^3 \tag{11.62}$$

where $\phi = 0.07k^3n^4$ for $n > 1$
$= 0.125k^3$ for $n = 1$
n = number of longitudinal stiffeners

Longitudinal stiffeners should be extended to locations where the maximum stress in the flange does not exceed that allowed for base metal adjacent to or connected by fillet welds. At least one transverse stiffener should be installed near dead-load inflection points. It should be the same size as the longitudinal stiffeners.

Compression Flanges Stiffened Longitudinally and Transversely. When $w/t > 97/\sqrt{F_y/k}$ and the number of longitudinal stiffeners exceeds two, addition of transverse stiffeners should be considered. They are not necessary, however, if the ratio of their spacing to flange width b exceeds 3. For the flange, including stiffeners, to be designed for the basic allowable stress $0.55F_y$, w/t for the longitudinal stiffeners should not exceed

$$\frac{w}{t} = \frac{97}{\sqrt{F_y/k_1}} \tag{11.63}$$

$$k_t = \frac{[1 + (a/b)^2]^2 + 87.3}{(n + 1)(a/b)^2[1 + 0.1(n + 1)]} \leq 4 \tag{11.64}$$

where a = spacing, in, of transverse stiffeners. For larger values of w/t but not more than 60 or $210\sqrt{F_y/k_1}$, the stress, ksi, in the flange should not exceed

$$F_b = F_y\left(0.326 + 0.224 \sin\frac{c''\pi}{2}\right) \tag{11.65}$$

$$c'' = \frac{210 - (w/t)\sqrt{F_y/k_1}}{113} \tag{11.66}$$

When $210\sqrt{F_y/k_1} < w/t < 60$, the stress, ksi, should not exceed

$$F_b = \frac{14{,}400k_1}{(w/t)^2} \tag{11.67}$$

Spacing of transverse stiffeners should not exceed $4w$ when k_1 has its maximum value of 4.

When transverse stiffeners are used, each longitudinal stiffener should have a moment of inertia I_s as given by Eq. (11.62) with $\phi = 8$. Each transverse stiffener should have a moment of inertia, in^4, about an axis through its centroid parallel to its bottom edge of at least

$$I_t = \frac{0.10(n + 1)^3w^3f_bA_f}{Ea} \tag{11.68}$$

where f_b = maximum longitudinal bending stress, ksi, in flange in panels on either side of transverse stiffener
A_f = area, in^2, of bottom flange, including stiffeners
E = modulus of elasticity of flange steel, ksi

Depth-thickness ratio of outstanding elements should not exceed the value determined by Eq. (11.61).

Transverse stiffeners need not be connected to the flange. But they should be attached to the girder webs and longitudinal stiffeners. Each of these web connections should be capable of resisting a vertical force, kips,

$$R_w = \frac{F_yS_t}{2b} \tag{11.69}$$

where S_t = section modulus, in^3, of transverse stiffener and F_y = yield strength, ksi, of stiffener. Each connection of a transverse and longitudinal stiffener should be capable of resisting a vertical force, kips,

$$R_s = \frac{F_y S_t}{nb} \tag{11.70}$$

Flange-to-Web Welds. Total effective thickness of welds connecting a flange to a web should at least equal the web thickness, except that when two or more diaphragms per span are provided, minimum size fillet welds may be used (see Art. 11.22). If fillet welds are used, they should be placed on both sides of the flange or web.

11.19 HYBRID GIRDERS

When plate girders are to be used for a bridge, costs generally can be cut by using flanges with higher yield strength than that of the web. Such construction is permitted for highway bridges under AASHTO specifications if the girders qualify as hybrid girders. Such girders are cost effective because the web of a plate girder contributes relatively little to the girder bending strength and the web shear strength depends on the depth/thickness ratio.

Hybrid girders, in general, may be designed for fatigue as if they were homogeneous plate girders of the flange steel. Composite and noncomposite I-shaped girders may qualify as hybrid.

Noncomposite girders must have both flanges of steel with the same yield strength. Yield strength of web steel should be lower, but not more than 35% less. Different areas may be used at the same cross section for top and bottom flanges. If, however, the bending stress in either flange exceeds $0.55F_{yw}$, where F_{yw} is the specified minimum yield stress of the web, ksi, the tension-flange area should be larger than the compression-flange area. In composite construction, the transformed area of the effective concrete slab or reinforcing steel should be included in the top-flange area.

Composite girders, in contrast, may have a compression flange of steel with yield strength less than that of the tension flange but not less than that of the web. Yield strength of web steel should be lower, but not by more than 35%, than the yield strength of the tension flange.

Criteria governing design of hybrid girders generally are the same as for homogeneous plate girders (Arts. 11.15 and 11.16). Those that differ follow.

Web. Average shear stress in the web should not exceed the allowable for the web steel.

The bending stress in the web may exceed the allowable for the web steel if the stress in each flange does not exceed the allowable for the flange steel multiplied by a reduction factor R.

$$R = 1 - \frac{\beta\psi(1 - \alpha)^2(3 - \psi + \psi\alpha)}{6 + \beta\psi(3 - \psi)} \tag{11.71}$$

where $\alpha = F_{yw}/F_{yf}$
F_{yw} = minimum specified yield strength of web, ksi
F_{yf} = minimum specified yield strength of flange, ksi
β = ratio of web area to tension-flange area
ψ = ratio of distance, in, between outer edge of tension flange and neutral axis (of the transformed section for composite girders) to depth, in, of steel section

In computation of maximum permissible depth-thickness ratios for a web, f_b should be taken as the calculated bending stress, ksi, in the compression flange divided by R.

In design of bearing stiffeners at interior supports of continuous hybrid girders for which $\alpha < 0.7$, no part of the web should be assumed to act in bearing.

Flanges. In composite girders, the bending stress in the concrete slab should not exceed the allowable stress for the concrete multiplied by R.

In computation of maximum permissible width-thickness ratios of a compression flange, f_b should be taken as the calculated bending stress, ksi, in the flange divided by R.

11.20 ORTHOTROPIC-DECK BRIDGES

In orthotropic-deck construction, the deck is a steel plate overlaid with a wearing surface and stiffened and supported by a rectangular grid. The steel deck assists its supports in carrying bending stresses. Main components usually are the steel deck plate, longitudinal girders, transverse floorbeams, and longitudinal ribs. Ribs may be open-type (Fig. 11.12*a*) or closed (Fig. 11.12*b*).

The steel deck acts as the top flange of the girders (system I, Fig. 11.12*c*). Also, the steel deck serves as the top flange of the ribs (Fig. 11.12*e*) and floorbeams (system II, Fig. 11.12*d*). In addition, the deck serves as an independent structural member that transmits loads to the ribs (system III, Fig. 11.12*e*).

Load Distribution. In determining direct effects of wheel loads on the deck plate, in design of system III for H20 or HS20 loadings, single-axle loads of 24 kips, or double-axle loads of 16 kips each spaced 4 ft apart, should be used. The contact area of one 12- or 8-kip

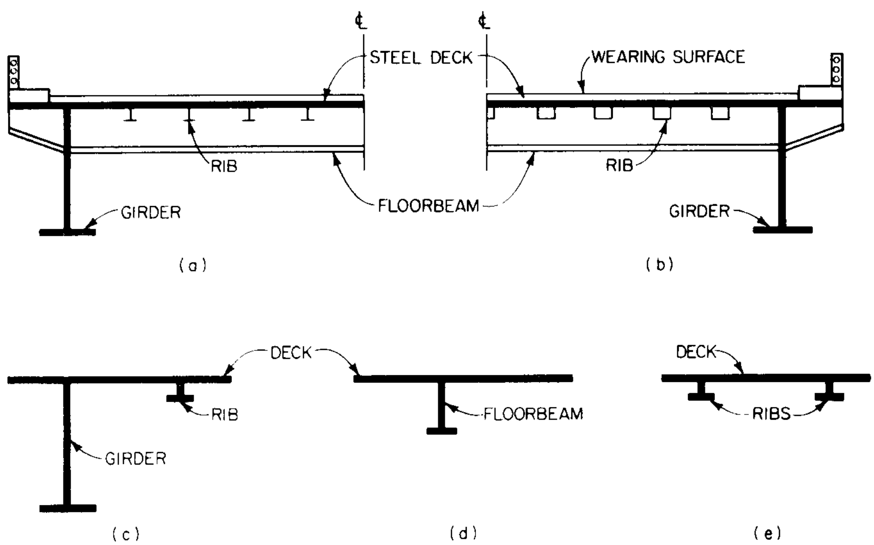


FIGURE 11.12 Orthotropic-plate construction. (*a*) With open ribs. (*b*) With closed ribs. (*c*) Deck and ribs act as the top flange of the main girder. (*d*) Deck acts as the top flange of the floorbeam. (*e*) Deck distributes loads to the ribs.

wheel may be taken as 20 in wide (perpendicular to traffic) and 8 in long at the roadway surface. The loaded area of the deck may be taken larger by the thickness of the wearing surface on all sides, by assuming a 45° distribution of load through the pavement.

Deck Thickness. Usually, the deck plate is made of low-alloy steel with a yield point of 50 ksi. Thickness should be at least ⅜ in and is determined by allowable deflection under a wheel, unless greater thickness is required by design of system I or II. Deflection due to wheel load plus 30% impact should not exceed $1/300$ of spacing of deck supports. Deflection computations should not include the stiffness of the wearing surface. When support spacing is 24 in or less, the deck thickness, in, that meets the deflection limitation is

$$t = 0.07ap^{1/3} \tag{11.72}$$

where a = spacing, in, of open ribs, or maximum spacing, in, of walls of closed ribs and p = pressure at top of steel deck under 12-kip wheel, ksi.

Allowable Stresses (ASD). Stresses in ribs and deck acting as the top flange of the girders and in the ribs due to local bending under wheel loads should be within the basic allowable tensile stress. But when the girder-flange stresses and local bending stresses are combined, they may total up to 125% of the basic allowable tensile stress. Local bending stresses are those in the deck plate due to distribution of wheel loads to ribs and beams. AASHTO standard specifications limit local transverse bending stresses for the wheel load plus 30% impact to a maximum of 30 ksi unless fatigue analysis or tests justify a higher allowable stress. If the spacing of transverse beams is at least 3 times that of the webs of the longitudinal ribs, local longitudinal and transverse bending stresses need not be combined with other bending stresses, as indicated in the following.

Elements of the longitudinal ribs and the portion of the deck plate between rib webs should meet the minimum thickness requirements given in Table 11-25. The stress f_a may be taken as the compressive bending stress due to bending of the rib, bending of the girder, or 75% of the sum of those stresses, whichever is largest. Unless analysis shows that compressive stresses in the deck induced by bending of the girders will not cause overall buckling of the deck, the slenderness ratio L/r of any rib should not exceed

$$\frac{L}{r} = 1000 \sqrt{\frac{1.5}{F_y} - 2.7 \frac{F}{F_y^2}} \tag{11.73}$$

where L = distance, in, between transverse beams
r = radius of gyration, in³, about the horizontal centroidal axis of the rib plus effective area of deck plate
F_y = yield strength, ksi, of rib steel
F = maximum compressive stress, ksi (taken positive) of the deck plate acting as the top flange of the girders

The effective width, and hence the effective area, of the deck plate acting as the top flange of a longitudinal rib or a transverse beam should be determined by analysis of the orthotropic-plate system. Approximate methods may be used. (See, for example, Art. 4.13 or "Design Manual for Orthotropic Steel Plate Deck Bridges," American Institute of Steel Construction.) For the girders, the full width of the deck plate may be considered effective as the top flange if the girder span is at least 5 times the maximum girder spacing and 10 times the maximum distance from the web to the nearest edge of the deck. (For continuous beams, the span should be taken as the distance between inflection points.) If these conditions are not met, the effective width should be determined by analysis.

The elements of the girders and beams should meet requirements for minimum thickness given in Table 11.25 and for stiffeners (Art. 11.12.4).

When connections between ribs and webs of beams, or holes in beam webs for passage of the ribs, or rib splices occur in tensile regions, they may affect the fatigue life of the bridge adversely. Consequently, these details should be designed to resist fatigue as described in Art. 11.10. Similarly, connections between the ribs and the deck plate should be designed for fatigue stresses in the webs due to transverse bending induced by wheel loads.

At the supports, some provision, such as diaphragms or cross frames, should be made to transmit lateral forces to the bearings and to prevent transverse rotation and other deformations.

The same method of analysis used to compute stresses in the orthotropic-plate construction should be used to calculate deflections. Maximum deflections of ribs, beams, and girders due to live load plus impact should not be more than $1/500$ of the span. See also Art. 12.10.

11.21 SPAN LENGTHS AND DEFLECTIONS

Many designers believe that steel girders, because of their lower weight per foot, should have longer spans than concrete beams for a bridge at the same location. This is not necessarily the case. The AISC has conducted studies that show that there are substantial economies for the steel alternative when the spans are kept the same, including the cost of extra substructure units. However, as with any preliminary study, site-specific considerations may indicate otherwise. For example, where the foundation or substructure costs, or both, are extremely high, it is probable that longer steel girders, with fewer substructure units, would be more cost-effective than shorter spans.

Deflection of steel bridges has always been important in design. If a bridge is too flexible, the public often complains about bridge vibrations, especially if sidewalks are present. There is also a concern that bridge vibrations may accelerate fatigue damage or cause premature deck deterioration. In an attempt to satisfy all these concerns, the AASHTO standard specifications include limitations on deflection and depth-span ratios as a means of ensuring sufficient stiffness of bridge members (Art. 11.3.1).

There is some doubt about the need for these limitations, especially relative to the potential for increased deck cracking. Many studies indicate flexibility of the superstructure is not a cause of increased deck cracking. The AISC notes that most European countries do not have live-load deflection limits in their design specifications.

The AASHTO LRFD specifications require that deflections be checked as part of the service limit state and include in the ‘‘Commentary’’ the statement: ‘‘Service limit states are intended to allow the bridge to perform acceptably for its service life. . . . Bridges should be designed to avoid undesirable structural or psychological effects from their deflection and vibrations. While no specific deflection, depth, or frequency limitations are specified herein, except for orthotropic decks, any large deviation from past successful practice regarding slenderness and deflections should be cause for review of the design to determine that it will perform adequately.’’

The LRFD specifications provide optional criteria for deflections that are essentially the same as those in the standard specifications. These provisions apply to all structures, not just steel, as was the case in the past. The LRFD specifications also require checking I-section members for permanent deflections.

11.22 BEARINGS

Bridges should be designed so that a total movement due to temperature change of $1\frac{1}{4}$ in per 100 ft can take place. Also, provisions should be made for changes in length of span resulting from live-load stresses. In spans over 300 ft long, allowance should be made for expansion and contraction in the floor system.

Expansion bearings may be needed to permit such movements. (See also Art. 11.26.) In addition, to control of the movements, at least one fixed bearing is required in each simple or continuous span. A fixed bearing should be firmly anchored against horizontal and vertical movement, but it may permit the end of the member supported to rotate in a vertical plane. An expansion bearing should permit only end rotation and movement parallel to the longitudinal axis of the supported member, unless provisions for transverse expansion are necessary

Allowable bearing on granite is 800 psi and on sandstone or limestone, 400 psi, when the masonry projects 3 in or more beyond the edge of the bearing plate. For smaller projections, only 75% of these stresses is allowed. For reinforced concrete, the basic allowable stress f_c is 30% of the 28-day compressive strength. When the supporting surface is wider on all sides than the loaded area A_1, the allowable stress may be multiplied by $\sqrt{A_2/A_1} \leq 2$, where A_2 is the area of the supporting surface.

Bearings for spans of 50 ft or more should be designed to permit end rotation. For the purpose, curved bearing plates, elastomeric pads, or pin arrangements may be used. Elastomeric bearings are generally preferred. At expansion bearings, such spans may be provided with rollers, rockers, or sliding plates. Shorter spans may slide on metal plates with smooth surfaces.

In all cases, design of supports should ensure against accumulation of dirt, which could obstruct free movement of the span, and against trapping of water, which could accelerate corrosion. Beams, girders, or trusses should be supported so that bottom chords or flanges are above the bridge seat.

Self-lubricating bronze or copper-alloy sliding plates, with a coefficient of friction of 0.10 or less, may be used in expansion bearings instead of elastomeric pads, rollers, or rockers. These plates should be at least ½ in thick and chamfered at the ends.

Rockers generally are preferred to rollers because of the smaller probability of becoming frozen by dirt or corrosion. The upper surface of a rocker should have a pin or cylindrical bearing. The lower surface should be cylindrical with center of rotation at the center of rotation of the upper bearing surface. At the nominal centerline of bearing, the lower portion should be at least 1½ in thick. The effective length of rocker for computing line bearing stress should not exceed the length of the upper bearing surface plus the distance from lower to upper bearing surface. Adequate web material should be provided and arranged to ensure uniform load distribution over the effective length. The rocker should be doweled to the base plate.

Rollers are the alternative when the pressure on a rocker would require it to have too large a radius to keep bearing stress within the allowable. Rollers may be cylindrical or segmental. They should be at least 6 in in diameter. They should be connected by substantial side bars and guided by gearing or other means to prevent lateral movement, skewing, and creeping. The roller nest should be designed so that the parts may be easily cleaned.

Effective bearing area for rockers and rollers equals effective length times effective width. Effective length of bearing area may be taken equal to effective length of rocker, or to roller length plus twice the thickness of the base plate. Effective width of bearing area may be taken as 4 times the base-plate thickness for rockers, or the distance between end rollers plus 4 times the baseplate thickness for rollers. The vertical load may be assumed uniformly distributed over the effective bearing area, except for eccentricity from rocker travel.

Sole plates and masonry plates should be at least ¾ in thick. For bearings with sliding plates but without hinges, the distance from centerline of bearing to edge of masonry plate, measured parallel to the longitudinal axis of the supported member, should not exceed 4 in plus twice the plate thickness. For spans on inclines exceeding 1% without hinged bearings, the bottom of the sole plate should be radially curved or beveled to be level.

Elastomeric pads are bearings made partly or completely of elastomer. They are used to transmit loads from a structural member to a support while allowing movements between the bridge and the support. Pads that are not all elastomer (reinforced pads) generally consist of alternate layers of steel or fabric reinforcement bonded to the elastomer. In addition to the reinforcement, the bearings may have external steel plates bonded to the elastomeric bearings. AASHTO prohibits tapered elastomeric layers in reinforced bearings.

The AASHTO ‘‘Standard Specifications for Highway Bridges’’ contain specifications for the materials, fabrication, and installation of the bearings. The specifications also present two methods for their design, both based on service loads without impact and the shear modulus at 73°F. The grade of elastomer permitted depends on the temperature zone in which the bridge is located. The specifications also require that either (1) a positive slip apparatus be installed and bridge components be able to withstand forces arising from a bearing force equal to twice the design shear force or (2) bridge components be able to sustain the forces arising from a bearing force equal to four times the design shear force. If the shear force exceeds one-fifth the dead-load compressive force, the bearing should be fixed against horizontal movement.

Design should allow for misalignment of girders because of fabrication or erection tolerances, camber, or other sources. It should also provide for subsequent replacement of bearings, when necessary. Also, it should ensure that bearings are not subjected to uplift when in service.

A beam or girder flange seated on an elastomeric bearing should be stiff enough to avoid damaging it. Stiffening may be achieved with a sole plate or bearing stiffeners. I beams and girders symmetrically placed on a bearing do not require such stiffening if the width-thickness ratio b_f/t_f of the bottom flange does not exceed

$$\frac{b_f}{t_f} = 2\sqrt{\frac{F_y}{3.4f_c}} \tag{11.74}$$

where b_f = total width, in, of the flange
t_f = thickness, in, of flange or flange plus sole plate
F_y = minimum yield strength, ksi, of girder steel
f_c = average compressive stress P/A, ksi, due to dead plus live load, without impact

PTFE pads are bearings with sliding surfaces made of polytetrafluoroethylene (PTFE), which may consist of filled or unfilled sheet, fabric with PTFE fibers, interlocked bronze and filled PTFE structures, PTFE-perforated metal composites and adhesives, or stainless-steel mating surfaces. The AASHTO standard specifications contain specifications for the materials, fabrication, and installation of the bearings.

The sliding surfaces of the pads permit translation or rotation by sliding of the PTFE surfaces over a smooth, hard mating surface. This should preferably be made of stainless steel or other corrosion-resistant material. To prevent local stresses on the sliding surface, an expansion bearing should permit rotation of at least 1° due to live load, changes in camber during construction, and misalignment of the bearing. This may be achieved with such devices as hinges, curved sliding surfaces, elastomeric pads, or preformed fabric pads.

PTFE sliding surfaces should be factory-bonded or mechanically fastened to a rigid backup material capable of resisting bending stresses to which the surfaces may be subjected. The surface mating to the PTFE should be an accurate mate, flat, cylindrical, or spherical, as required, and should cover the PTFE completely in all operating positions of the bearing. Preferably, the mating surface should be oriented so that sliding will cause dirt and dust to fall off it.

Pot bearings are used mainly for long-span bridges. They are available as fixed, guided expansion, and nonguided expansion bearings, designed to provide for thermal expansion and contraction, rotation, camber changes, and creep and shrinkage of structural members.

They consist of an elastomeric rotational element, confined and sealed by a steel piston and steel base pot. In effect, a structure supported on a pot bearing floats on a low-profile hydraulic cylinder, or pot, in which the liquid medium is an elastomer.

To facilitate rotation of the elastomeric rotational element, either PTFE sheets are attached to the top and bottom of the elastomeric disk or the element is lubricated with a material compatible with the elastomer. To permit longitudinal or transverse movements, the upper surface of the steel piston is faced with a PTFE sheet and supports a steel sliding-top bearing plate. The mating surface of that plate is faced with polished stainless steel.

Pot bearings have low resistance to bending in their plane. Consequently, a sole plate, beveled if necessary, should be provided on top of the bearing and a masonry plate should be installed on the bottom. A member should not be supported on both a pot bearing and a bearing with different properties.

To ensure contact between the piston and the elastomer, minimum load should be at least 20% of the design vertical load capacity.

Pedestals and shoes, if required, usually are made of cast steel or structural steel. Design should be based on the assumption that the vertical load is uniformly distributed over the entire bearing surface. The difference in width or length between top and bottom bearing surfaces should not exceed twice the vertical distance between them. For hinged bearings, this distance should be measured from the center of the pin.

AASHTO recommends that the web plates and angles connecting built-up pedestals and shoes to the base plate should be at least ⅝ in thick.

If pedestal size permits, webs should be rigidly connected transversely to ensure stability of the components. Webs and pinholes in them should be arranged to keep eccentricity to a minimum. The net section through a pinhole should provide at least 140% of the net area required for the stress transmitted through the pedestal or shoe. All parts of pedestals and shoes should be prevented from lateral movement on the pins.

Nuts with washers should be used to hold pins in place. Length of pins should be adequate for full bearing.

Anchor bolts subject to tension should be designed to engage a mass of masonry that will provide resistance to uplift equal to 150% of the calculated uplift due to service loads or 100% of loading combinations for which live load plus impact is increased 100%, whichever is larger. The bolts, however, may be designed for 150% of the basic allowable stress. Resistance to pullout of anchor bolts may be obtained by use of swage bolts or by placing on each embedded end of a bolt a nut and washer or plate. Minimum requirements for number of bolts for each bearing, diameter, and embedment are given in Table 11.26 for ASD and LRFD. The LRFD specifications does not set minimums.

TABLE 11.26 Minimum Number of Anchor Bolts per Bearing for ASD and LFD

Span, ft	No. of bolts	Diameter, in	Embedment, in
	(*a*) Trusses and girders		
50 or less	2	1	10
51–100	2	1¼	12
101–150	2	1½	15
150 or more	4	1½	15
	(*b*) Rolled beams		
All outer spans	2	1	10

11.23 DETAILING FOR WELDABILITY

Overdetailing of weld sizes and joint configurations can cause unnecessary fabrication and in-service problems and higher costs. Some designers believe "more weld metal is better" and "complete-penetration groove welds are better than fillet welds." But oversizing welds or specifying joint figurations that are not practical can cause weld defects that are otherwise avoidable.

Whenever possible, designers should allow fabricators to select the type of joint to be used and the size of weld (Fig. 11.13). Include maximum and minimum sizes for fillet welds as follows:

Limitations on Fillet-Weld Size. The maximum size of a fillet weld is the same as the material thickness, up to ¼ in. For material ¼ in thick or more, size is limited to 1⁄16 in less than the material thickness, unless the drawings indicate that the weld should be built up to get full throat thickness.

Minimum size of fillet weld is based on the base-metal thickness of the thinner part joined, and single-pass welds must be used. For material ¾ in thick or less, weld size should be at least ¼ in. For thicker material, weld size may not be less than 5⁄16 in. Only if the strength requirement exceeds that provided by the minimum size of fillet weld is it necessary to indicate the size of a fillet weld on the drawings. The "Bridge Welding Code," ANSI/AASHTO/AWS D1.5, provides adequate assurance of proper weld strength and quality. Letting fabricators select joint details for efficient utilization of their plant setup ensures the most costeffective fabrication.

The AASHTO specifications also require that the minimum length of a fillet weld be 4 times its size but at least 1½ in. If a fillet weld is subjected to repeated stress or to a tensile force not parallel to its axis, it should not end at a corner of a part or a member. Instead, it should be turned continuously around the corner for a distance equal to twice the weld size (if the return can be made in the same plane). End returns should not be provided around transverse stiffeners. Seal welds should be continuous.

Welding of Box Girders. Poor detailing of a box girder or other type of enclosed member has been another source of fabrication problems and has contributed to adverse in-service performance when designs have not provided properly for fabrication. For example, designers often specify a complete-penetration groove weld for a corner, and the backing bar needed

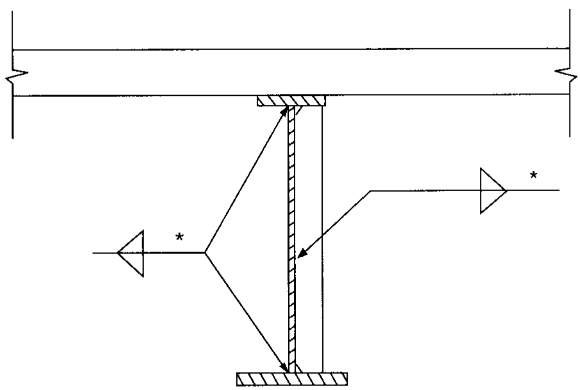


FIGURE 11.13 Symbols indicate welds to be made to a girder. Asterisks indicate that the weld sizes are to be selected by the fabricator. A note should be placed on the drawing to that effect. This does not apply when stress levels control.

to ensure integrity of the weld is not always installed properly. Backing bars are sometimes left discontinuous, and this soon causes a fatigue crack to initiate. Also, when internal stiffeners are required for a box girder, which is frequently the case for large sections, assembly problems are encountered where welds or backing bars are interrupted at the stiffeners. Figure 11.14 shows a detail with backing bar that is not recommended for a box girder and a preferred arrangement that eliminates both the need for a backing bar and for welding to be done inside the box for attachment of the web to the top plate.

The assembly procedure requires first welding of the two webs to the bottom flange. For the purpose, continuous fillet welds are placed on one or both sides. Then, the stiffeners are welded to the webs (also to the compression flange if the member will be subjected to bending). Finally, the top flange is connected to the webs with fillet welds. The advantage of this procedure lies in the fact that it is usually practicable to get a fillet weld of better quality, easier to inspect with a nondestructive test, and less expensive than a complete-penetration weld.

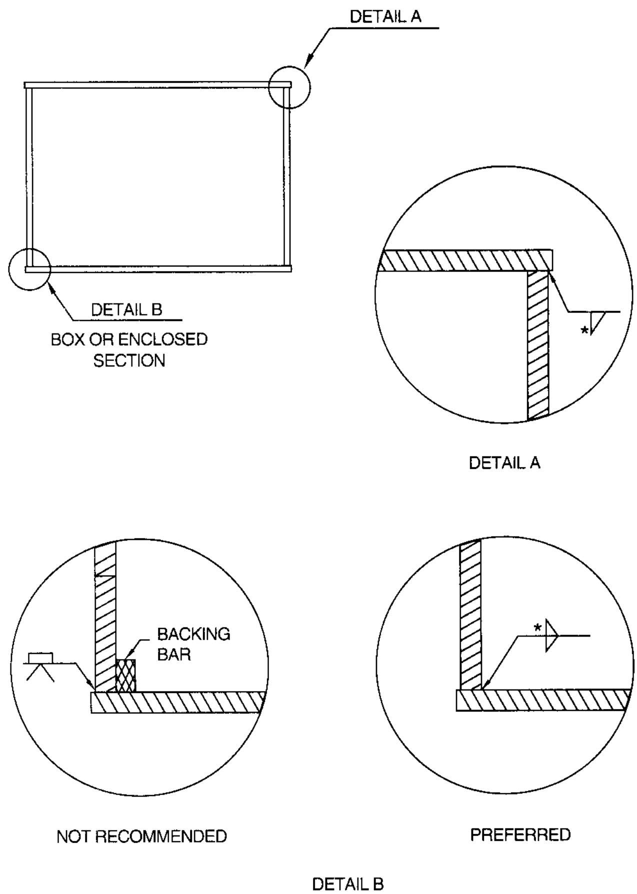


FIGURE 11.14 Corner joints for a box-shape member. Detail *A* requires a fillet weld between web and top flange. Asterisk indicates that the size of the weld is to be selected by the fabricator. This does not apply when stress levels control. Detail *B* shows two schemes for welding of the web to the bottom flange, one not recommended and the other preferred.

Welding of HPS Steels. With the introduction of HPS to the designers inventory of steels, additional weld parameters must be considered. At the present time, HPS is a quenched-and-tempered (Q&T) steel. The LRFD Specification states that the engineer may specify electrode classifications with strengths less than the base metal when detailing fillet welds for Q&T steels. The Bridge Welding Code, AWS D1.5, also allows use of undermatched fillet welds for all steels where the stress is in tension or compression parallel to the weld axis, and shear on the effective area meets AASHTO design requirements. Although undermatched welds are applicable to any design, it is of particular importance for steels with strengths of 70 ksi and higher.

Rules for Fillet Welds. The following rules are recommended for detailing of fillet welds for all girders, particularly those of HPS:

1. Use only minimum size fillet welds, except where greater strength is required.
2. Use undermatched fillet welds (consumables for grade 50 steels) for grade 70 steels and higher.
3. Use non-weathering consumables for all single pass fillet welds (AWS D1.5, Art. 4.1.5) even on unpainted structures.
4. For fillet welds joining steels of two different yield points, use consumables applicable to the lower strength base metal.

11.24 STRINGER OR GIRDER SPACING

One of the major factors affecting the economy of highway bridges with a concrete deck on stringers or longitudinal girders is spacing of the main members. Older bridges typically had spacing of 8 ft or less. Now, however, longer concrete-deck spans (up to 15 ft) are practicable through use of such devices as stay-in-place metal or precast-concrete forms. This makes possible fewer girders. (To eliminate the potential for fracture criticality when I-shape girders are used, there should be at least three.) Although the steel weight per square foot of bridge may be higher with fewer girders, the reduced costs of fabrication, handling, transportation, erecting, and painting, if required, usually provides substantial overall savings.

11.25 BRIDGE DECKS (ASD and LFD)

Highway-bridge decks usually are constructed of reinforced concrete. Often, this concrete is made with conventional aggregate and weighs about 150 lb per cu ft. Sometimes, it is made with lightweight aggregate, resulting in 100 to 110 lb per cu ft concrete. Lightweight aggregate normally consists of slag, expanded shale, or expanded clay.

In some concrete decks, the wearing surface is cast integrally with the structural slab. In others, a separate wearing surface, consisting of asphaltic concrete or conventional concrete, is added after the structural slab has been placed.

In instances where weight saving is important, particularly in movable spans, or in spans where aerodynamic stability is of concern, an open, steel-grid floor is specified. Where compromise is necessary, this grid is partly or completely filled with asphaltic or lightweight concrete to provide protection under the structure or to provide a more suitable riding surface.

For orthotropic-plate structures, it is necessary to provide over the steel deck a wearing surface on which traffic rides. These wearing surfaces are generally of three types: a layered system, stabilized mastic system, or thin combination coatings.

The layered system consists of a steel-deck prime coat, such as zinc metallizing, bituminous-base materials, or epoxy coatings. Over this coat is applied a copper or aluminum foil, or an asphalt mastic, followed by a leveling course of asphalt binder or stabilized mastic, and a surface course of stone-filled mastic asphalt or asphaltic concrete.

The stabilized mastic system consists of a prime coat on the steel, as in the layered system, followed by a layer of mastic, which is choked with rolled-in crushed rock.

Combination coatings contain filled epoxies or alkyd-resin binders in a single coating with silica sand.

A bridge deck serves as a beam on elastic foundations to transfer wheel loads to the supporting structural steel. In orthotropic bridges, the deck also contributes to the load-carrying capacity of longitudinal and transverse structural framing. In composite construction, the concrete deck contributes to the load-carrying capacities of girders. In fulfilling these functions, decks are subject to widely varying stresses and strains, due not only to load but also to temperature changes and strains of the main structure.

In general, bridge decks are designed as flexural members spanning between longitudinal or transverse beams and supporting wheel loads. A wheel usually is considered a concentrated load on the span but uniformly distributed in the direction normal to the span.

Concrete Slabs. The **effective span** S, ft, for a concrete slab supported on steel beams should be taken as the distance between edges of flanges plus half the width of a beam flange.

Allowable Stresses. The allowable compressive stress for concrete in design of slabs is $0.4f'_c$, where f'_c = 28-day compressive strength of concrete, ksi. The allowable tensile stress for reinforcing bars for grade 40 is 20 ksi and for grade 60, 24 ksi. Slabs designed for bending moment in accordance with the following provisions may be considered satisfactory for bond and shear.

Bending Moment. Because of the complexity of determining the exact load distribution, AASHTO specifications permit use of a simple empirical method. The method requires use of formulas for maximum bending moment due to live load (impact not included). Two principal cases are treated depending on the direction in which main reinforcement is placed. The formulas are summarized in Table 11.27. In these formulas, S is the effective span, ft, of the slab, as previously defined.

For rectangular slabs supported along all edges and reinforced in two directions perpendicular to the edges, the proportion of the load carried by the short span may be assumed for uniformly distributed loads as

$$p = \frac{b^4}{a^4 + b^4} \tag{11.75}$$

For a load concentrated at the center,

$$p = \frac{b^3}{a^3 + b^3} \tag{11.76}$$

where a = length of short span of slab, ft, and b = length of long span of slab, ft. If the length of slab exceeds 1.5 times the width, the entire load should be assumed carried by the reinforcement of the short span. The distribution width E, ft, for the load taken by either span should be determined as provided for other slabs in Table 11.27. Reinforcement determined for bending moments computed with these assumptions should be used in the center half of the short and long spans. Only 50% of this reinforcement need be used in the outer quarters. Supporting beams should be designed taking into account the nonuniform load distribution along their spans.

All slabs with main reinforcement parallel to traffic should be provided with edge beams. They may consist of a slab section with additional reinforcement, a beam integral with but deeper than the slab, or an integral, reinforced section of slab and curb. Simply supported edge beams should be designed for a live-load moment, ft-kips, of $1.6S$ for HS20 loading

TABLE 11.27 Live-Load Bending Moments, ft-kips per ft of Width, in Concrete Slabs for ASD and LFD*

Direction of main reinforcement and type of span	Loading	
	HS20	HS15
Perpendicular to traffic ($2 \leq S \leq 24$):		
Simple spans	$0.5(S + 2)$	$0.375(S + 2)$
Continuous spans	$\pm 0.4(S + 2)$	$\pm 0.3(S + 2)$
Cantilevers, $E = 0.8x + 3.75$†	$16x/E$†	$12x/E$†
Parallel to traffic:		
Simple spans:		
$S \leq 50$	$0.900S$	$0.675S$
$50 < S \leq 100$	$1.3S - 20$	$0.750(1.3S - 20)$
Continuous spans	By analysis‡	By analysis‡
Cantilevers, $E = 0.35x + 3.2 \leq 7$†	$16x/E$†	$12x/E$†

* Based on "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

† x = distance, ft, from load to support.

‡ Moments in continuous spans with main reinforcement parallel to traffic should be determined by analysis for the truck or appropriate lane loading. Distribution of wheel loads $E = 4 + 0.06S \leq 7$ ft. Lane loads should be distributed over a width of $2E$.

and $1.2S$ for HS15 loading, where S is the beam span, ft. For positive and negative moments in continuous beams, these values may be reduced 20%.

Distribution reinforcement is required in the bottom of all slabs transverse to the main reinforcement, for distribution of concentrated wheel loads. The minimum amounts to use are the following percentages of the main reinforcement steel required for positive moment:

$$\text{For main reinforcement parallel to traffic,} \quad \frac{100}{\sqrt{S}} \leq 50\%$$

$$\text{For main reinforcement perpendicular to traffic,} \quad \frac{220}{\sqrt{S}} \leq 67\%$$

where S = effective span of slab, ft. When main reinforcing steel is perpendicular to traffic, the distribution reinforcement in the outer quarters of the slab span need be only 50% of the required distribution reinforcement.

Transverse unsupported edges of the slab, such as at ends of a bridge or expansion joints, should be supported by diaphragms, edge beams, or other means, designed to resist moments and shears produced by wheel loads.

The effective length, ft, of slab resisting post loadings may be taken as

$$E = 0.8x + 3.75 \tag{11.77}$$

where no parapet is used, with x = distance, ft, from center of post to point considered. If a parapet is used, $E = 0.8x + 5$.

Steel Grid Floors. For grid floors filled with concrete, the load distribution and bending moments should be determined as for concrete slabs. The strength of the composite steel and concrete slab should be computed by the transformed-area method (Art. 11.16). If necessary to ensure adequate load transference normal to the main grid elements, reinforcement should be welded transverse to the main steel.

For open-grid floors, a wheel load should be distributed normal to the main bars over a distance equal to twice the center-to-center spacing of main bars plus 20 in for H20 loading, or 15 in for H15 loading. The portion of the load assigned to each bar should be uniformly distributed over a length equal to the rear-tire width (20 in for H20 loading and 15 in for H15). The strength of the section should be determined by the moment-of inertia method (Art. 11.15). Supports should be provided for all edges of open-grid floors.

11.26 ELIMINATION OF EXPANSION JOINTS IN HIGHWAY BRIDGES

At expansion bearings and at other points where necessary, expansion joints should be installed in the floor system to permit it to move when the span deflects or changes length. If apron plates are used, they should be designed to bridge the joint and prevent accumulation of dirt on the bridge seats. Preferably, the apron plates should be connected to the end floorbeam. For amount of movement to provide for, see Art. 11.21. However, jointless bridges have many advantages and should be considered where possible.

Short-span bridges usually have expansion joints at one or both abutments. Longer-span structures usually have such joints at pier or off-pier hinges. Although these joints may relieve some forces caused by restraint of thermal movements, the joints have been a major source of bridge deterioration and poor ridability. The LRFD specifications of the American Association of State Highway and Transportation Officials (AASHTO) acknowledges that ''Completely effective joint seals have yet to be developed for some situations. . . .'' To provide more durable bridges, the goal in design should be to minimize the number of joints. One way to do this for multiple-span bridges is to use continuous beams or girders. Another, more general, alternative is to eliminate joints completely.

Some states permit jointless, or integral, steel-girder bridges with spans up to about 400 ft or longer. With this type of construction, restriction of the change in bridge length due to maximum temperature change induces longitudinal forces at fixed piers and abutments. This must be taken into account in design of substructures. Experience has shown, however, that the effect of these forces on superstructure design is negligible and that, with proper detailing, substructure design is relatively unaffected.

Tennessee is a major user of jointless steel-girder bridges for spans of 400 ft or more. Through experience, they have developed details that are able to resist thermal forces and movements (Fig. 11.15), thus eliminating leaking bridge joints. Tennessee has successfully completed a two-span continuous bridge 473 ft long with integral abutments at each end.

The AASHTO ''Standard Specifications for Highway Bridges'' specifies that movement calculations for integral abutments take into account not only temperature changes but also creep of the concrete deck and pavements. The abutments should be designed to sustain the forces generated by restraint to thermal movements developed by the pressures of fills behind the abutments. (The specifications prohibit use of integral abutments constructed on spread footings keyed into rock.) Approach slabs should be connected directly to abutments and wingwalls, to prevent intrusion of water behind the abutments. Nevertheless, means should be provided for draining away water that may get entrapped.

The AASHTO specifications also require that details comply with recommendations in Technical Advisory T5140.13, Federal Highway Administration. These recommendations include the following:

Steel bridges with an overall length less than 300 ft should be constructed continuously and, if unrestained, have integral abutments. (''An unrestrained abutment is one that is free to rotate, such as a stub abutment on one row of piles or an abutment hinged at the footing.''—''Structure Memorandum,'' State of Tennessee.) Greater lengths may be used when experience dictates such designs are satisfactory.

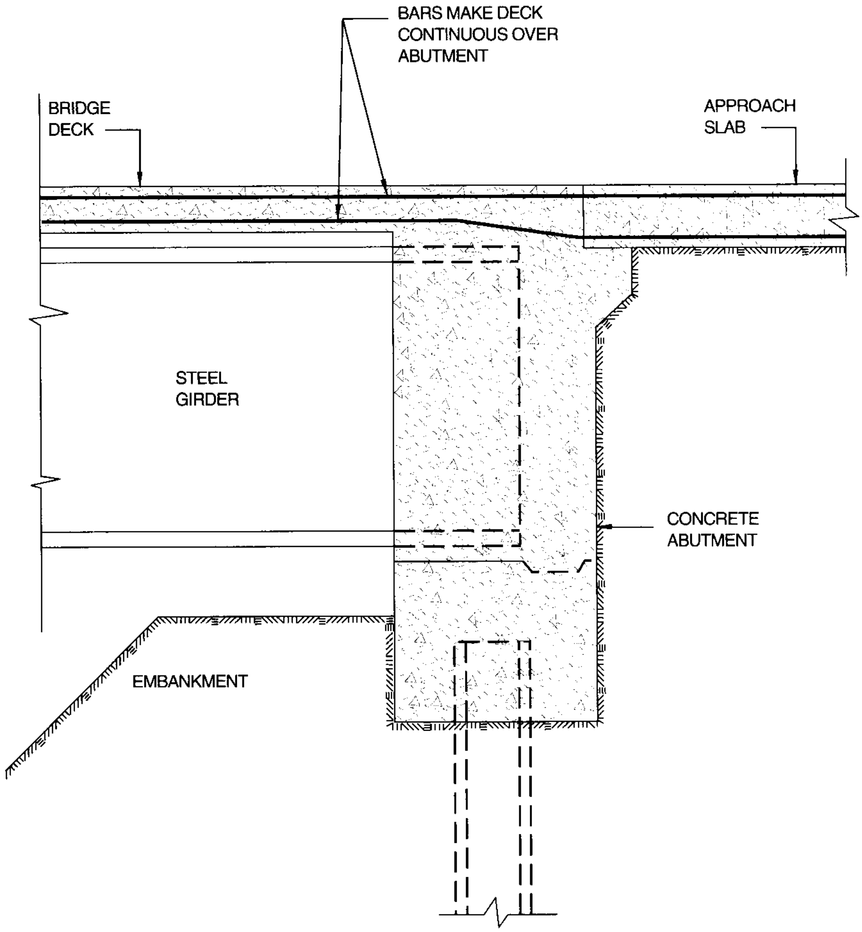


FIGURE 11.15 Details for an integral abutment.

In the area immediately behind integral abutments, traffic will compact the fill where it is partly disturbed by abutment movement, if not prevented from doing so. For the purpose, approach slabs should be provided to span this area. The span length should be at least equal to a minimum of 4 ft for bearing on the soil plus the depth of the abutment (based on the assumption of a 1:1 slope from the bottom of the rear face of the abutment.) The Advisory suggests that a practical slab length is 14 ft.

The Advisory recommends that approach slabs be designed for live-load bending moments as indicated for the case of main reinforcement parallel to traffic in Table 11.27, with S = slab length minus 2 ft.

The Advisory also recommends that the slabs be anchored by steel reinforcement to the superstructure. In addition, positive anchorage should be provided between integral abutments and the superstructure. Figure 11.15 is an example of such construction.

The Advisory calls attention to a detail used by North Dakota that it considers desirable. To accommodate pavement growth and bridge movement, the state inserts a roadway expansion joint 50 ft away from the bridge.

Properly detailed and constructed, jointless bridges eliminate the maintenance that would be required if expansion joints were used, especially corrosion and deterioration of substructure and superstructure because of leakage. Also, jointless bridges provide better ridability. As a bonus, the cost of joints is eliminated. The LRFD Specification encourages the use of jointless bridges to improve "rideability" of the roadway surface, but provides minimal design guidance. However, comprehensive design and detailing provisions for bridges with integral abutments are available from the American Iron and Steel Institute (AISI), as *Integral Abutments for Steel Bridges*. A design procedure for the piles supporting the integral abutment is included.

Where foundation conditions are not considered acceptable for integral abutment bridges, "semi-integral" abutments are acceptable, within the same length limitations. A semi-integral abutment is virtually identical to an integral abutment, except that there is a horizontal joint separating the backwall and beam from the pile footing. Thus, bridges with battered piles or rock foundations are candidates for semi-integral abutments. Semi-integral abutments are also used effectively in bridge rehabilitations to eliminate joints.

11.27 BRIDGE STEELS AND CORROSION PROTECTION

One of the most important decisions designers have to make is selection of the proper grade of steel and corrosion-protection system. These should not only meet structural needs but also provide an economical structure capable of long-term, low-maintenance performance.

Specifications of the American Association of State Highway and Transportation Officials recognize structural steels designated M270 with a specified grade. These are equivalent to ASTM A709 steels of the same grades except for AASHTO-specified mandatory notch toughness. Material properties of M270 steels and other equivalent ASTM steels are listed in Table 11.28. (See also Table 1.1 and Art. 1.1.5). Steels that meet AASHTO M270 requirements are prequalified for use in welded bridges.

Designers should have available AASHTO "Standard Specifications for Transportation Materials and Methods of Sampling and Testing," Part 1, "Specifications," and Part 2, "Tests," to ensure that appropriate material properties are specified for their designs.

High Performance Steels (HPS), a new addition to the family of bridge steels, provide an opportunity to significantly increase reliability while reducing cost. Only Grade HPS70W, with a minimum yield point of 70,000 psi, has been fully developed and is presently available for bridge design. To qualify as HPS, the material has to provide improved weathering characteristics and significantly higher impact toughness. HPS has a corrosion index, I, of 6.5 and higher, thus providing increased resistance to weathering over earlier grades of steels designated as weathering (*W*). Weathering grades are defined as having a corrosion index, I, of 6.0 and higher as calculated using ASTM Standard G101. In addition, Charpy V-notch (CVN) impact properties for this steel usually exceed 100 ft-lbs at −10°F.

11.27.1 Minimum Steel Thickness

Because structural steel in bridges is exposed to the weather, minimum-thickness requirements are imposed on components to obtain a long life despite corrosion. Where steel will be exposed to unusual corrosive influences, the component should be increased in thickness beyond required thickness or specially protected against corrosion.

In highway bridges, structural steel components, except railings, fillers, and webs of certain rolled shapes, should be at least 5⁄16 in thick. Web thickness of rolled beams or channels should be at least 0.23 in (0.25 in for LRFD). Closed ribs in orthotropic-plate decks should be at least 3⁄16 in thick (0.25 in for LRFD). Fillers less than ¼ in thick should not be extended beyond splicing material. In addition, minimum thickness may be governed by slenderness ratios (Table 11.24) or maximum width-thickness or depth-thickness ratios (Table 11.25).

11.27.2 Weathering Steels

A preferred way to achieve economy for bridges is to use steel of a ''weathering'' grade when conditions permit. This is a type of steel that has enhanced atmospheric corrosion resistance *when properly used* and does not require painting under most conditions. Although it costs slightly more per pound than other steels of equivalent grade its initial cost and life-cycle cost is usually less than that of painted structural steel. The weathering grades are available only with yield points of 50 ksi and higher. Before selecting a weathering steel, designers should determine the corrosivity of the environment in which the bridge will be located as a first step. This will determine whether the use of an unpainted steel of grade 50W, 70W, HPS70W, or 100W (Table 11.28 and Arts. 1.1.4 and 1.1.5) is appropriate. These steels provide the most cost-effective grade that can be used in most situations and have proven to be capable of excellent performance even in areas where deicing salts are used. But use of good detailing practices, such as jointless bridges, is imperative to assure adequate performance (Art. 11.26).

The Federal Highway Administration ''Guidelines for the Use of Unpainted Weathering Steel,'' to ensure a long-term and adequate performance of unpainted steels, recommends the following:

If the proposed structure is to be located at a site with any of the environmental or location characteristics noted below, use of uncoated weathering-grade steels should be considered with caution. A study of both the macroenvironment and microenvironment by a corrosion consultant may be required. In all environments, designers must pay careful attention to detailing, specifically as noted in the following recommendations for design details. Also, owners should implement, as a minimum, the maintenance actions as noted in the following.

Environments to be treated with caution include marine coastal areas; regions with frequent high rainfall, high humidity, or persistent fog; and industrial areas where concentrated chemical fumes may drift directly onto structures.

Locations to be treated with caution include grade separations in tunnel-like conditions, where concentration of vehicle exhausts may be highly corrosive; also, low-level water cross-

TABLE 11.28 Highway-Bridge Structural Steels*

Type	Structural[a] steel	High-strength low-alloy steel[a]		Quenched and tempered low-alloy steel[b]		High-yield-strength, quenched and tempered alloy steel[b]	
AASHTO designation	M 270 grade 36	M 270 grade 50	M 270 grade 50W	M 270 grade 70W	In process	M 270 grades 100/100W	
Equivalent ASTM designations	A 709 grade 36	A 709 grade 50	A 709 grade 50W	A 709 grade 70W	A 709 grade HPS70W	A 709 grades 100/100W	
						Plate 2½ in thick or less	Plates over 2½ in thick to 4 in, incl.
Minimum tensile strength, F_u	58	65	70	90	90	110	100
Minimum yield point or minimum yield strength, F_y	36	50	50	70	70	100	90

* Based on specifications of American Association of State Highway and Transportation Officials. See also Table 1.1 and Art. 1.1.5.
[a] For plate thicknesses 4 in or less and all structural shape groups.
[b] Not available as structural shapes.

ings, with clearance of 10 ft or less over stagnant, sheltered water or 8 ft or less over moving water.

Design details for uncoated steel in bridges and other highway structures require careful consideration of the following:

1. Elimination of bridge joints where possible.
2. If expansion joints are used, they must be able to control water that comes on the deck. A trough under the deck joint may serve to divert water away from vulnerable elements.
3. Painting all superstructure steel within a distance of 1½ times the depth of girder from bridge joints.
4. Avoiding use of welded drip bars where fatigue stresses may be critical.
5. Minimizing the number of bridge-deck scuppers.
6. Eliminating details that serve as water and debris "traps."
7. If box girders are used, they should be hermetically sealed, when possible, or provided with weep holes to allow proper drainage and circulation of air. All openings in boxes that are not sealed should be covered or screened.
8. Protecting pier caps and abutment walls to minimize staining.
9. Sealing overlapping surfaces exposed to water, to prevent capillary penetration of moisture.

Maintenance actions advisable include the following:

1. Implementing procedures designed to detect and minimize corrosion.
2. Controlling roadway drainage by diverting roadway drainage away from the bridge structure, cleaning troughs or resealing deck joints, maintaining deck drainage systems, and periodically cleaning and, when needed, repainting all steel within a minimum distance of 1½ times the depth of the girder from bridge joints.
3. Regularly removing all dirt, debris, and other deposits that trap moisture.
4. Regularly removing all vegetation and other matter that can prevent the natural drying of wet steel surfaces.
5. Maintaining covers and screens over access holes.

The preceding recommendations are applicable to all structures, painted or unpainted, to ensure satisfactory performance. Unpainted structures that have been in existence for 30 or more years in environments consistent with these recommendations have provided excellent service, testifying to the adequacy of the weathering grades of steel. ("Performance of Weathering Steel in Highway Bridges—A Third Phase Report", American Iron and Steel Institute, Washington, DC, 1995.)

11.27.3 Paint Systems

Where weathering grades of steel are not appropriate, only high-performance paint systems should be specified for corrosion protection. Designers should be aware, however, that recommendations for paint systems change periodically, primarily due to the need for consideration of environmental impacts. Lead-based paints, for example, are no longer acceptable due to their health hazard. Also, concern for the effect of volatile organic compounds on the ozone in the atmosphere has caused a change from mineral-based to water-based paints. Consequently, designers should ensure that only current technology is specified in contract documents.

The AASHTO "Guide for Painting Steel Structures" provides state-of-the-art information for the painting of new bridge steels, as well as paint removal and repainting of existing steel bridges.

11.28 CONSTRUCTABILITY

Sometimes, unnecessary problems develop during construction of a bridge that could have easily been prevented with an appropriate design. Also, the construction procedures used by a contractor may lock in stresses unaccounted for in design that will adversely influence the service life of the bridge. Two specific areas where difficulties have occurred have been in construction of curved girder bridges and in deck-concrete placing sequences, especially when the bridge has a large skew.

As part of bridge design, the designers should assume an erection and concrete placing sequence and check for construction stresses. The assumed methods should be included on the contract plans for the contractor's information, with the understanding that deviations will be accepted subject to the ability of the contractor to demonstrate that no adverse stresses will result from the proposed method.

The AASHTO LRFD specifications, to ensure that designers properly consider constructibility, specify that bridges be designed so that fabrication and erection can be performed without undue difficulty or distress and that effects of locked-in construction forces are within tolerable limits. When the method of construction of a bridge is not self-evident, or could induce unacceptable locked-in stresses, the designer should propose at least one feasible method on the plans. If the design requires some strengthening or temporary bracing or support during erection by the selected method, the plans should indicate the need thereof.

To provide for the above, designers should check for what is essentially a construction limit state. For the purpose, the following factored load should be used:

$$1.25[D + 1.5L + 1.25W + 1.0\Sigma \text{ (other forces as appropriate)}]$$

where D is the dead load, L is the live load, and W is the wind load. This concept should be applied to all designs, regardless of which specification is used.

11.29 INSPECTABILITY

Inspectability of all bridge members and connections is an essential design-stage consideration. This is especially apparent when the structure includes enclosed sections, such as box girders. Bridge service life has been impaired in the past when designers, concerned with stress distribution, either did not include access holes or made them so small it was impossible for an inspector to perform an adequate inspection. To ensure inspectability, experienced bridge inspectors should review the bridge design at an early stage of development.

Another consideration is safety of inspectors and traffic using the bridge during the inspection. A preferred method of inspection has been use of a type of crane that allows easy access to underbridge members. But, on routes with very high traffic volumes, the presence of an inspection vehicle on the bridge creates a safety hazard to both inspection personnel and the traveling public. Other means of inspection should be provided in these instances, such as inspection ladders, walkways, catwalks, covered access holes, and provision for lighting, if necessary.

11.30 REFERENCE MATERIALS

Besides the ''Standard Specifications for Highway Bridges'' and the ''LRFD Bridge Design Specifications'' referred to frequently in preceding articles, the American Association of State Highway and Transportation Officials publishes numerous reference books, guide specifications, manuals, interim specifications, periodicals and other reference materials useful for design, fabrication, and erection of steel highway bridges. Obtain the latest catalog listing these from AASHTO, 444 N. Capitol St., NW, Washington, DC 20001.