SECTION 11
DESIGN CRITERIA FOR BRIDGES

PART 2
RAILROAD BRIDGE DESIGN

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11.31 STANDARD SPECIFICATIONS

The primary purpose of railroad bridges is to safely handle track loadings without causing train delays or track slow orders. Recommended practices for the design of railroad bridges are now promulgated by the American Railway Engineering and Maintenance-of-Way Association (AREMA), 8201 Corporate Drive, Suite 1125, Landover, Maryland, 20785-2230, as part of their *Manual.* The recommended practices given in Chapter 15 of the AREMA *Manual* were prepared and updated by Committee 15 of the American Railway Engineering Association (AREA) for many years. AREMA now carries on this work through the same committee personnel. The information presented in this article is primarily directed toward the design of fixed bridges. The design of movable bridges, which is covered in Chapter 15, Part 6 of the AREMA *Manual,* embodies many engineering disciplines not generally required for fixed bridges.

11.32 DESIGN METHOD

Railroad bridges are generally designed by the service load/allowable stress method. AREMA recommendations are based on an 80-year design service life. However, guidance is provided for determining other service life expectancies when design parameters differ from those used in preparing the recommendations.

11.33 OWNER'S CONCERNS

The railroad bridge designer is frequently involved in planning for the replacement of an existing bridge that is carrying operating tracks. The designer must know the owner's tolerance for detouring trains and/or the time the track can be out of service and make these constraints part of the design-erection procedure.

Grade separation projects to take vehicular and pedestrian traffic under operated tracks requires the bridge designer to understand and utilize the owner's requirements to ensure safe train operations. Railroad owners may provide their own design criteria to supersede or augment AREMA recommendations. Also, a state Department of Transportation (DOT) may use its specifications for part of the design. Designers need to understand the interests of all parties as well as their responsibility to the bridge owner. Note that the term "underpass" is sometimes used, denoting a structure that carries the railroad traffic over the other entity.

11.34 DESIGN CONSIDERATIONS

Design considerations for steel railroad bridges differ somewhat from those for highway bridges. Railroad bridges have a higher live-to-dead load ratio because the mass of the railroad loading is generally large, relative to that of the bridge. In case of accidents, rail traffic cannot steer away from damaged bridge components, but highway traffic can frequently be moved to other lanes while repairs are made. Rail traffic cannot be readily detoured; it is impossible on some rail lines and very disruptive and expensive on others. Thus, railroad bridge design should consider the ease of bridge repairs.

Unit trains, a "consist" made up of cars of the same kind and weight, can create a high number of similar loadings in a component with the passage of one train. Thus, the fatigue life of design details (Art. 11.38) is especially important under these conditions.

11.34.1 Open Deck Bridges

In railroad bridges of open deck design where the track is supported on a pair of stringers, the stringers should be spaced not less the 6.5 ft apart. The nominal bridge tie length is 10.0 ft. Where multiple stringers are used, they should be spaced to uniformly support the track load and provide stability.

11.34.2 Stringer and Floorbeam End Connections

Stringer and floorbeam end connections should be designed to provide for flexure in the outstanding leg of the connection angles. Connection angles should be not less than ½ in thick and the outstanding leg should be 4 in or greater in width. For stringers, in open and ballast deck construction, the gage distance, in, from the back of the connection angle to the first line of fasteners, over the top one-third of the depth of the stringer, should be not less than $\sqrt{Lt/8}$ where L is the length of the stringer span, in, and t is the angle thickness, in.

11.34.3 Deflections

Simple span deflection should be computed for the live load plus impact that produces the maximum bending moment at midspan. The maximum deflection should not exceed 1/640 of the span length, center-to-center of supports. The gross moment of inertia may be used for prismatic flexural members.

11.34.4 Safety

Safety devices required by the owner and by regulations must be provided for in the earliest stage of design. Safety devices may include such items as walkways, hand railings, vandal fences, ladders, grab-irons, bridge end-posts, clearance signs, refuge booths, stanchions, and fall protection fittings. A bridge located within 300 ft of a switch generally requires a walkway.

11.34.5 Skewed Bridges

Many railroads restrict the bridge skew angle. Generally, all bridge ends must be designed to provide structural support, at right angle to the centerline of track, for the end ties. This requires the bridge backwall to be designed at the same time as the spans.

11.34.6 Clearances

Appropriate clearances must be provided for in the design of all structures. Through-girder and through-truss bridges should provide a minimum of 9.0 ft horizontal side clearance, measured from the centerline of track. A minimum vertical clearance of 23.0 ft above the plane of the top of the high rail should be provided in through-truss bridges. The designer should verify clearance requirements with the owner.

11.34.7 Bridge Bearings

Masonry plates should have a minimum of 6 in of clearance from the free edge of concrete or masonry supports. Improved specifications for railroad bridge bearings are being developed to better utilize the available materials. Refer to Chapter 19 of the AREMA *Manual* for current requirements.

11.35 DESIGN LOADINGS

Bridges must be designed to carry the specified dead loads, live loads and impact, as well as centrifugal, wind, other lateral loads, loads from continuous welded rail, longitudinal loads and earthquake loads. The forces and stresses from each of these specified loads should be a separate part of the design calculations. Also, because rail cars have changed in size and weight over the years and frequently are run in unit consists, the designer should be alert to live loadings that may be more severe than those used in some specifications (Art. 11.35.2).

11.35.1 Dead Loads

Dead loads should be calculated based on the weight of the materials actually specified for the structure. The dead load for rail and fastenings may be assumed as 200 lb per ft of track. Unit weights of other materials may be taken as follows:

Material	Weight lb/cu ft
Timber	60
Ballast	120
Concrete	150
Steel	490

Note that walkway construction may add significantly to the dead load. Also, when a long body rail casting, such as expansion joints, are specified for a bridge, the castings should be supported only on one span of the stringers.

11.35.2 Live Load

Railroad bridges have been designed for many years using specified Cooper E Loadings. See Fig. 11.16*a* for the wheel arrangement and the trailing load for the Cooper E80 loading, which includes 80 kip axle loads on the drivers. This configuration can be moved in either direction across a span to determine the maximum moments and shears. With the continuing increase in car axle loads, AREMA has also adopted the Alternate Live Load on four axles shown in Fig. 11.16*b*. It recommends that bridge design be based on the E80 or the Alternate Loading, whichever produces the greater stresses in the member. A table of live load moments, shears, and reactions for both the E80 and the Alternate Loading may be found in the Appendix of Chapter 15 of the AREMA *Manual.* The table values are presented in terms of wheel loads (one-half of an axle load).

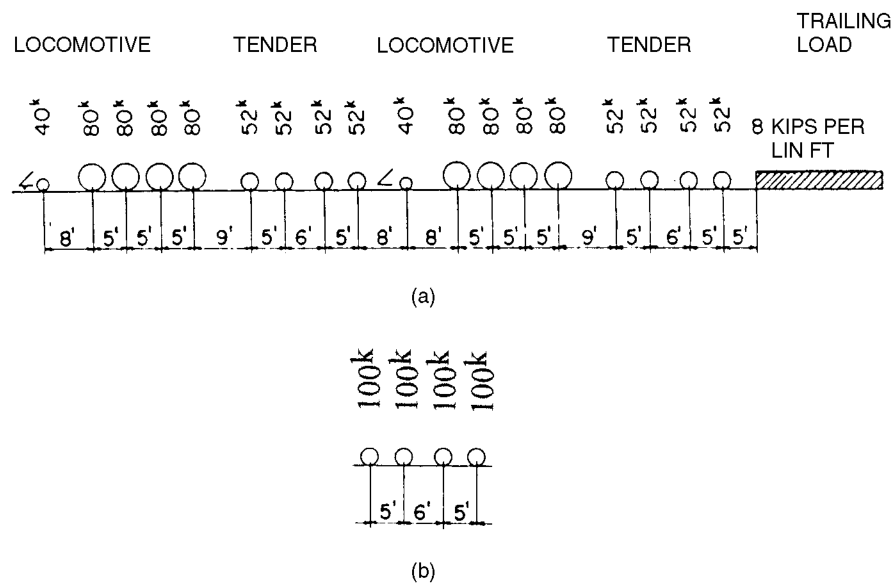


FIGURE 11.16 Loadings for design of railway bridges. (*a*) Cooper E80 load. (*b*) Alternate live load on four axles. (Adapted from *AREMA Manual,* American Engineering and Maintenance-of Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.)

Some owners may elect to use loadings other than E80 in some cases. Such loadings may be directly proportioned from the E80 loading according to the axle load on the drivers. For example, an owner specifying a new through truss or girder span may specify an E95 loading for the floor system and hangers, and an E80 loading for the rest of the structure. It is considered good practice to keep the bridge design loading well above the economical loading capacity of rolling stock and track structure.

11.35.3 Load Path

The path of the load from the wheels through the rail and into the tie, is either directly to the supporting beams, or through a ballast bed to a deck and thence into the supporting beams. Direct fixation of the rails to supporting members is not considered here.

Figure 11.17*a* provides a sectional view of an open-deck through-girder span. This type of construction should provide a clear space between ties of no more than 6 in. The guard timber shown at the end of the tie has the function of keeping the ties uniformly spaced and preventing tie skewing. Tie skewing must be prevented because it closes the gage between the rails. Hook bolts or tie anchor assemblies, not shown in the sketch, are used to fasten the tie to the support beam. The guard timbers are fastened to the ties with ⅝-in-diameter washerhead drive spikes, through bolts, or lag bolts.

Figure 11.17*b* provides a sectional view of a ballast-deck through-girder span. Many such spans are designed with closely spaced floorbeams, thus eliminating the stringers.

11.35.4 Load on Multi-Track Structures

To account for the effect of multiple tracks on a structure, the proportion of full live load on the tracks should be taken as follows:

Two tracks—Full live load.

Three tracks—Full live load on two tracks, one-half live load on third track.

Four tracks—Full live load on two tracks, one-half live load on one track, one-quarter live load on remaining track.

The tracks selected for these loads should be such that they produce the maximum live load stress in the member under consideration. For bridges carrying more than four tracks, the track loadings should be specified by the owner's engineer.

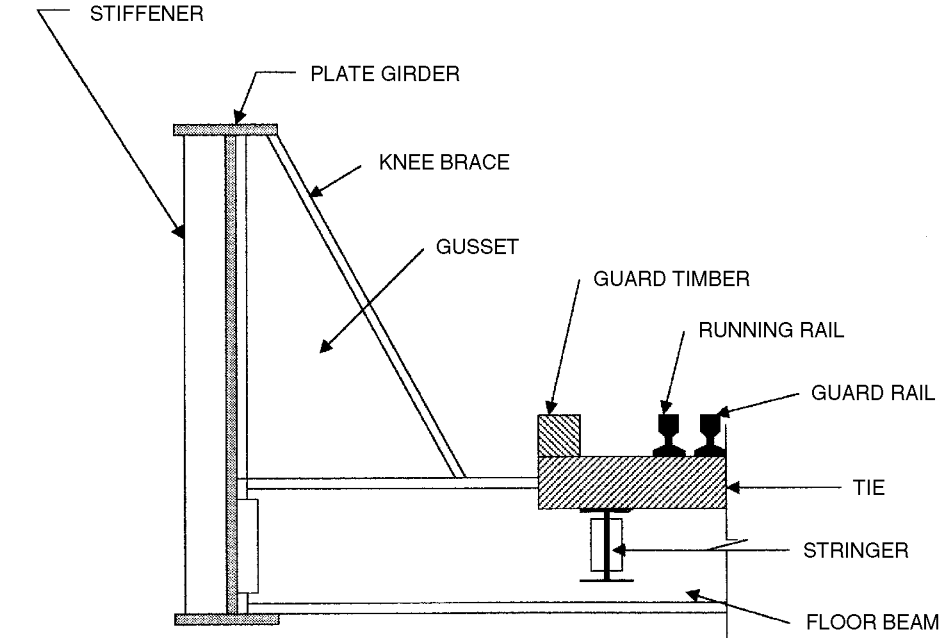
11.35.5 Impact Load

Impact loads, I, are expressed as a percentage of the specified axle load and should be applied downward or upward at the top of the rail. For open-deck bridge construction, the percentages are obtained from the applicable equations given below. For ballast-deck bridges designed according to specifications, use 90% of the impact load given for open deck bridges.

For rolling equipment without hammer blow (diesel or electric locomotives, tenders, rolling stock):

For $L < 80$ ft:

$$I = RE + 40 - \frac{3L^2}{1600} \tag{11.78}$$



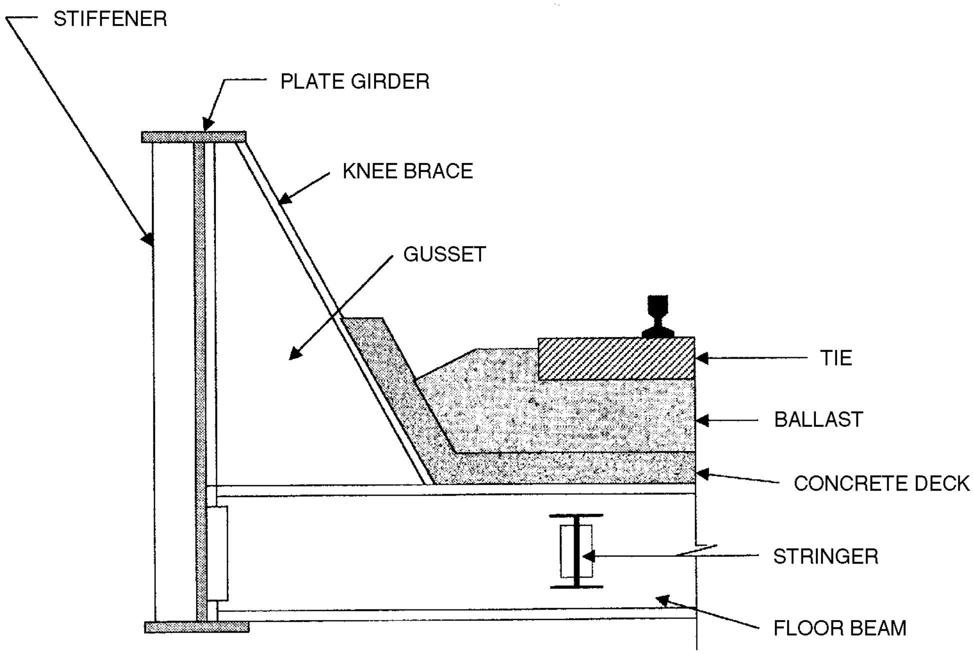


FIGURE 11.17 Part section of through-girder railway bridges. (*a*) Open deck construction. (*b*) Ballast deck construction.

For $L \geq 80$ ft:

$$I = RE + 16 + \frac{600}{L - 30} \tag{11.79}$$

For steam locomotives (hammer blow):

For girders, beam spans, stringers, floor beams, floor beam hangers, and posts of deck trusses that carry floor beam loads only:

For $L < 100$ ft:

$$I = RE + 60 - \frac{L^2}{500} \tag{11.80}$$

For $L \geq 100$ ft:

$$I = RE + 10 + \frac{1800}{L - 40} \tag{11.81}$$

For truss spans:

$$I = RE + 15 + \frac{4000}{L - 25} \tag{11.82}$$

In the above equations, $RE = 10\%$ (RE represents the rocking effect, acting as a couple with a downward force on one rail and an upward force on the other rail, thus increasing or decreasing the specified load); for stringers, transverse floor beams without stringers, longitudinal girders and trusses, L = length, ft, center to center of supports; for floor beams, floor beam hangers, subdiagonals of trusses, transverse girders, supports for longitudinal and transverse girders, and viaduct columns, L = length, ft, of the longer supported stringer, longitudinal beam, girder, or truss.

On multi-track bridges, the impact should be applied as follows:

When load is received from two tracks:

For $L \leq 175$ ft:

Full impact on two tracks.

For 175 ft $\leq L \leq$ 225 ft:

Full impact on one track and a percentage of full impact on the other track as given by (450-2L)

For L >225 ft:

Full impact on one track and no impact on other track.

When load is received from more than two tracks:

For all values of L:

Full impact on any two tracks.

For all design checks for fatigue, use the *mean impact* expressed as a percentage of the values given by the above equations, as follows:

$L \leq 30$ ft

100%

$L > 30$ ft

65%

11.35.6 Longitudinal Load

The longitudinal loads from trains on bridges are generally attributed to tractive or braking effort. With the current use of high adhesion locomotives and the development of better braking systems, bridges may be subject to greater longitudinal loads than in the recent past. The current AREMA recommendation is to assume the longitudinal load as 15% of the specified live load without impact for braking and 25% for traction.

Field measurements are being made on selected bridges to determine longitudinal loads associated with high adhesion locomotives. Until additional information is available for non-continuous rail across bridges, such as on structures with lift joints or expansion joints, the designer can consider locomotives as developing a draw bar effort of $0.90 \times 0.37 \times$ weight of the locomotive axles. Bridges in pull-back, push-in areas and on grades requiring heavy tractive effort, may experience greater than normal longitudinal loads.

The longitudinal load should be applied to one track only and should be distributed to the various components of the supporting structure, taking relative stiffnesses into account where appropriate, as well as the type of bridge bearings. The braking effort is assumed to act at 8 ft above the top of the rail, and tractive effort at 3 ft above the top of the rail.

11.35.7 Centrifugal Load

On curves, a centrifugal force corresponding to each axle should be applied horizontally through a point 6 ft above the top of the rail. This distance should be measured in a vertical plane along a line that is perpendicular to and at the midpoint of a radial line joining the tops of the rails. This force should be taken as a percentage C of the specified axle load without impact. Any eccentricity of the centerline of track on the support system requires the live load to be appropriately distributed to all components.

$$C = 0.00117S^2D \tag{11.83}$$

where S = train speed, mph
D = degree of curve = $5729.65/R$
R = radius of curve, ft

When the superelevation is 3 in less than that at which the resultant flange pressure between wheel and rail is zero,

$$C = 0.00117S^2D = 1.755(E + 3) \tag{11.84}$$

In that case, the actual superelevation, in, is given by

$$E = \frac{S^2D}{1500} - 3 = \frac{C - 5.625}{1.755}$$

and the permissible speed, mph, by

$$S = \sqrt{\frac{1500(E + 3)}{D}} \tag{11.85}$$

On curves, each axle load on each track should be applied vertically through the point defined above, 6 ft above top of rail. Impact should be computed and applied as indicated previously.

Preferably, the section of the stringer, girder, or truss on the high side of the superelevated track should be used also for the member on the low side, if the required section of the low-side member is smaller than that of the high-side member.

If the member on the low side is computed for the live load acting through the point of application defined above, impact forces need not be increased. Impact forces may, however, be applied at a value consistent with the selected speed, in which case the relief from centrifugal force acting at this speed should also be taken into account.

11.35.8 Lateral Loads From Equipment

In the design of bracing systems, the lateral force to provide the effect of the nosing of equipment, such as locomotives (in addition to the other lateral forces specified), should be a single moving force equal to 25% of the heaviest axle load (E80 configuration). It should be applied at the base of the rail. This force may act in either lateral direction at any point of the span.

On spans supporting multiple tracks, the lateral force from only one track should be used. Resulting vertical forces should be disregarded.

The resulting stresses to be considered are axial stresses in the members bracing the flanges of stringers, beams and girders, axial stresses in the chords of trusses and in members of cross frames of these spans, and the stresses from lateral bending of flanges of longitudinal flexural members, which have no bracing system.

The effects of the lateral load should be disregarded in considering lateral bending between brace points of flanges, axial forces in flanges, and the vertical forces transmitted to the bearings.

Stability of spans and towers should be calculated using a live load, without impact, of 1200 lb per ft. On multitrack bridges, this live load should be positioned on the most leeward side.

The lateral bracing of the compression chord of trusses, flanges of deck girders, and between the posts of viaduct towers, should be proportioned for a transverse shear force in any panel of 2.5% of the total axial force in both members in that panel, plus the shear force from the specified lateral loads.

11.35.9 Wind Load

AREMA recommended practices consider wind to be a moving load acting in any horizontal direction. On unloaded bridges, the specified load is 50 psf acting on the following surfaces:

Girder spans: 1½ times vertical projection

Truss spans: vertical projection of span plus any portion of leeward truss not shielded by the floor system

Viaduct towers and bents: vertical protection of all columns and tower bracing

On loaded bridges, a wind load of 30 psf acting as described above, should be applied with a wind load of 0.30 kip per ft acting on the live load of one track at a distance of 8 ft above the top of the rail. On girder and truss spans, the wind force should be at least 0.20 kip per ft for the loaded chord or flange and 0.15 kip per ft for the unloaded chord or flange.

The above specified loads were generally based on traditional rail cars with a vertical exposure of approximately 10 ft. Today, equipment such as double stack containers may have a vertical exposure of 20 ft and move in long blocks of cars. The designer should consider locations where high wind velocity and vehicle exposure may justify using greater loadings.

11.35.10 Earthquake Loads

Single panel simple span bridges designed in accordance with generally accepted practices for anchor bolts, bridge seat widths, edge distance on masonry plates, continuous rail, etc. may not require analysis for earthquake loads. In other cases, earthquake loads may be very important. The designer must take into account the owner's requirements and should refer to AREMA Chapter 9, ''Seismic Design for Railway Structures,'' for specific requirements.

11.35.11 Load From Continuous Welded Rail

Evaluation of the loads to be taken in the bridge components from continuous welded rail is very subjective. The sources of internal stress in the rail are generally temperature, braking, tractive effort of locomotives, rail creep, load from track curvature, and gravity in long track grades. The loads generated by these conditions depend upon the type of fastenings used. Thus, the bridge designer must be familiar with the fastening systems for rail and ties on open deck and ballast deck bridges. The rail must be adequately constrained against vertical and lateral movement as well as longitudinal movement, unless provision is made for expansion and contraction of the rail at one or more points on the bridge. Railroad bridge owners may have their own specifications for fastening rail on bridges that the designer must follow. Also, refer to AREMA Chapter 15, Part 8, for recommended practices.

11.35.12 Combination Loads Or Wind Load Only

Every component of substructure and superstructure should be proportioned to resist all combinations of forces applicable to the type of bridge and its site. Members subjected to stresses from dead, live, impact, and centrifugal loads should be designed for the smaller of the basic allowable unit stress or the allowable fatigue stress.

With the exception of floorbeam hangers, members subjected to stresses from other lateral or longitudinal forces, as well as to dead, live, impact, and centrifugal loads, may be proportioned for 125% of the basic allowable unit stresses, without regard for fatigue. But the section should not be smaller than that required with basic unit stresses or allowable fatigue stresses, when those lateral or longitudinal forces are not present. Note that there are two loading cases for wind: 50 psf on the unloaded bridge, or 0.30 kip per ft on the train on one track and 30 psf on the bridge.

Components subject to stresses from wind loads only should be designed for the basic allowable stresses. Also, no increase in the basic allowable stresses in high strength bolts should be taken for connections of members covered in this article.

11.35.13 Distribution of Loads Through Decks

The AREMA *Manual* contains recommended practices for distribution of the live loads described in Art. 11.35.2 to the ties in open deck construction and to the deck materials in ballast deck bridges. Attention is called to the provision that, in the design of beams and girders, the live load must be considered as a series of concentrated loads.

On open-deck bridges, ties within a length of 4 ft, but not more than three ties, may be assumed to support a wheel load. For ballasted-deck structures, live-load distribution is based on the assumption of standard crossties at least 8 ft long, about 8 in wide, and spaced not more than 2 ft on centers, with at least 6 in of ballast under the ties. For deck design, each axle load should be uniformly distributed over a length of 3 ft plus the minimum distance from bottom of tie to top of beams or girders, but not more than 5 ft or the minimum axle spacing of the loading. In the lateral direction, the axle load should be uniformly distributed over a length equal to the length of tie plus the minimum distance from the bottom of tie to top of beams or girders. Deck thickness should be at least ½ in for steel plate, 3 in for timber, and 6 in for reinforced concrete.

For ballasted concrete decks supported by transverse steel beams without stringers, the portion of the maximum axle load to be carried by each beam is given by

$$P = \frac{1.15AD}{S} \qquad S \geq d \tag{11.86}$$

where A = axle load
S = axle spacing, ft
D = effective beam spacing, ft
d = beam spacing, ft

For bending moment, within the limitation that D may not exceed either axle or beam spacing, the effective beam spacing may be computed from

$$D = d\,\frac{1}{1 + d/aH}\left(0.4 + \frac{1}{d} + \sqrt{\frac{H}{12}}\right) \tag{11.87}$$

where a = beam span, ft
$H = nI_b/ah^3$
n = ratio of modulus of elasticity of steel to that of concrete
I_b = moment of inertia of beam, in^4
h = thickness of concrete deck, in

For end shear, $D = d$. At each rail, a concentrated load of $P/2$ should be assumed acting on each beam.

D should be taken equal to d for bridges without a concrete deck or where the concrete slab extends over less than the center 75% of the floorbeam.

If $d > S$, P should be tile maximum reaction of the axle loads with the deck between beams acting as a simple span.

For ballasted decks supported on longitudinal girders, axle loads should be distributed equally to all girders whose centroids lie within a lateral width equal to length of tie plus twice the minimum distance from bottom of tie to top of girders.

Design requirements for use of timber and concrete for bridge decks is included in Chapters 7 and 8 of the AREMA *Manual*.

The designer should be aware of any pertinent requirements of the bridge owner for such items as concrete slab overhang, derailment conditions, composite action, waterproofing and drainage.

11.36 COMPOSITE STEEL AND CONCRETE SPANS

Simple span bridges with steel beams and concrete deck are sometimes designed on the basis of composite action. Specific provisions are given in the AREMA *Manual,* Chapter

15, Part 5. Additionally, many owners have special provisions intended to assure that the steel beams have sufficient strength to carry specified loads in the event the concrete deck is damaged in a derailment or other event.

11.37 BASIC ALLOWABLE STRESSES

Table 11.29 lists the allowable stresses for railroad bridges recommended in the AREMA *Manual.* The stresses, ksi, are related to the specified minimum yield stress F_y, or the specified minimum tensile strength F_u, ksi, of the material except where stresses are independent of the grade of steel. The basic stresses may be increased for loading combinations (Art. 11.35.12), or may be superseded by allowable fatigue stresses (Art. 11.38).

Allowable stresses for welds for railroad bridges are given in Table 11.30. These stresses may also be increased for loading combinations (Art. 11.35.12), or may be superseded by allowable fatigue stresses (Art. 11.38). The designer should review the AREMA *Manual* for complete provisions, including prohibited types of welds and joints. Special provisions may apply for fracture critical members.

TABLE 11.29 Basic Allowable Stresses for Railroad Bridges[a]

Loading condition	Allowable stress, psi[b]
Tension:	
Axial, net section	$0.55F_y$
Floorbeam hangers, including bending, net section with:	
Rivets in end connections	14,000
High-strength bolts in end connections	19,800
Bending, extreme fiber of rolled shapes, girders, and built-up sections, net section	$0.55F_y$
Compression:	
Axial, gross section, in:	
Stiffeners of plate girders (check also as column)	$0.55F_y$
Splice material	$0.55F_y$
Compression members centrally loaded:	
when $KL/r \le 3388/\sqrt{F_y}$	$0.55F_y$
when $3388/\sqrt{F_y} < KL/r < 27{,}111/\sqrt{F_y}$	$0.60F_y - \left(\dfrac{0.55F_y}{1662}\right)^{3/2} \dfrac{KL}{r}$
when $KL/r \le 27{,}111/\sqrt{F_y}$	$\dfrac{147{,}000{,}000}{(KL/r)^2}$
where: KL = effective length of compression member, in	
K = ⅞ for members with pin-end conditions	
K = ¾ for members with riveted, bolted, or welded end connections	
r = applicable radius of gyration of compression member, in	
Compression in extreme fibers of I-type members subjected to loading perpendicular to the web	$0.55F_y$

TABLE 11.29 Basic Allowable Stresses for Railroad Bridges[a] (*Continued*)

Loading condition	Allowable stress, psi[b]

Compression in extreme fibers of welded built-up plate or rolled beam flexural members symmetrical about the principal axis in the plane of the web (other than box-type flexural members), and compression in extreme fibers of rolled channels, the larger of the values computed by (Note 1)

$$0.55F_y\left[1 - \frac{(L/r_y)^2F_y}{1{,}800{,}000}\right] \qquad \text{or} \qquad \frac{10{,}500{,}000}{Ld/A_f}$$

but not to exceed $0.55F_y$

where: I = distance between points of lateral support for compression flange, in
r_y = minimum radius of gyration of the compression flange and that portion of the web area on the compression side of the axis of bending, about an axis in the plane of the web, in
A_f = area of the smaller flange excluding any portion of the web, in^2
d = overall depth of member, in

Compression in extreme fibers of riveted or bolted built-up flexural members symmetrical about the principal axis in the plane of the web, other than box-type flexural members (Note 1)

$$0.55F_y\left[1 - \frac{(L/r_y)^2F_y}{1{,}800{,}000}\right]$$

Compression in extreme fibers of box type welded, riveted or bolted flexural members symmetrical about the principal axis midway between the webs and whose proportions meet the provisions of AREMA Articles 1.6.1 and 1.6.2 (Note 1)

$$0.55F_y\left[1 - \frac{(L/r)_e^2F_y}{1{,}800{,}000}\right]$$

where $(L/r)_e$ is the effective slenderness ratio of the box type flexural member as determined by

$$(L/r)_e = \sqrt{\frac{3.95S_xL\sqrt{\Sigma s/t}}{A\sqrt{I_y}}}$$

where: L = distance between points of lateral support for compression flange, in
S_x = section modulus of box type member about its major axis, in^3
A = total area enclosed within center lines of box type member webs and flanges, in^2
s/t = ratio of width of any flange or depth of web component to its thickness. (Neglect any portion of flange that projects beyond the box section.)
I_y = moment of inertia of box type member about its minor axis, in^4

TABLE 11.29 Basic Allowable Stresses for Railroad Bridges[a] (*Continued*)

Loading condition	Allowable stress, psi[b]
Diagonal tension in webs of girders and rolled beams at sections where maximum shear and bending occur simultaneously	$0.55F_y$
Stress in extreme fibers of pins	$0.83F_y$
Shear in webs of rolled beams and plate girders, gross section	$0.35F_y$
Shear in ASTM A 325 bolts	17,000 (Note 2)
Shear in ASTM A 490 bolts	21,000 (Note 2)
Shear in power driven ASTM A 502 Grade 1 rivets	13,500
Shear in power driven ASTM A 502 Grade 2 rivets	20,000
Shear in hand driven ASTM A 502 Grade 1 rivets	11,000
Shear in pins	$0.42F_y$
Bearing on power driven ASTM A Grade 1 rivets:	
in single shear	27,000
in double shear	36,000
Bearing on power driven ASTM A 502 Grade 2 rivets, on material with yield point F_y:	
in single shear	$0.75F_y$
but not to exceed	40,000
in double shear	$1.00F_y$
but not exceed	50,000
(Rivets driven by pneumatically or electrically operated hammers are considered power driven.)	
Bearing on hand driven A 502 Grade 1 rivets	20,000
Bearing on pins where F_y = yield point of the material on which the pin bears, or of the pin material, whichever is less	$0.75F_y$
Bearing on ASTM A 325 and ASTM A 490 bolts:	
the smaller of (Note 3)	$\dfrac{LF_u}{2d}$ or $1.2F_u$
where: L = distance, in, measured in line of force from the centerline of a bolt to the nearest edge of an adjacent bolt or to the end of the connected part toward which the force is directed d = diameter of bolts, in F_u = lowest specified minimum tensile strength of connected part, ksi	
Bearing on milled stiffeners and other steel parts in contact	$0.83F_y$
Bearing between rockers and rocker pins where F_y = yield point of the material in the rocker or pin, whichever is	$0.375F_y$
Bearing on net area of self-lubricating bronze plate	2,000
Bolts subjected to combined tension and shear	$F_v \leq S_a(1 - f_t A_b / T_b)$

TABLE 11.29 Basic Allowable Stresses for Railroad Bridges[a] (*Continued*)

Loading condition	Allowable stress, psi[b]
where: F_v = allowable shear stress, reduced due to combined stress, psi S_a = allowable shear stress, when loaded in shear only, psi f_t = average tensile stress due to direct load, psi A_b = nominal bolt area, in^2 T_b = minimum tension of installed bolts, lb	
Bearing on expansion rollers and rockers, lb per lin in:	
for diameters up to 25 in	$\dfrac{F_y - 13{,}000}{20.000} 600d$
for diameters from 25 in to 125 in	$\dfrac{F_y - 13{,}000}{20.000} 3000\sqrt{d}$
where: d = diameter of roller or rocker, in F_y = yield point of steel in roller or base, whichever is less	

[a] For steel castings, allowable stresses in compression and bearing are same as those of structural steel of the same yield point. Other allowable stresses are 75% of those of structural steel of the same yield point.

[b] For bearing on expansion rollers and rockers, values are expressed as lb per lin in.

Note 1: The expression $0.55F_y[1 - (L/r)_e^2 F_y/1{,}800{,}000]$ is applicable only for members with solid rectangular flanges and for standard I-shaped beams.

Note 2: Applicable for surfaces with clean mill scale free of oil, paint, lacquer or other coatings and loose oxide, for standard size holes as specified in AREMA *Manual,* Art. 3.2.5. Where the Engineer has specified special treatment of surfaces or other than standard holes in a slip-criical connection, the allowable stresses in AREMA Commentary, Table 9.2, may be used if approved by the Engineer.

Note 3: For single bolt in line of force or connected materials with long slotted holes, 1.0 F_u is the limit. A value of allowable bearing pressure F_p on the connected material at a bolt greater than permitted can be justified provided deformation around the bolt hole is not a design consideration and adequate pitch and end distance L, are provided according to $F_p = LF_u/2d \leq 1.5F_u$.

Source: Adapted from *AREMA Manual,* American Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.

TABLE 11.30 Allowable Stresses on Welds

Type of weld	Electrode tensile strength class, psi	Allowable stress, psi[a]
Groove welds in tension or compression of base metal	[b]	$0.35F_y$
Groove welds in shear of base metal	[b]	$0.35F_y$
Fillet welds in shear (force applied in any direction)	60,000 70,000 80,000	16,500[c] 19,000[c] 22,000[c]

[a] F_y refers to yield point of base metal.

[b] Use matching weld metal.

[c] Also limited to 0.35 times F_y of base metal.

Source: Adapted from *AREMA Manual,* American Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.

11.37.1 Allowable Bearing Pressures on Masonry

For bearing assemblies with specified edge distances, with or without shock pads, the following allowable bearing stresses may be used for the indicated supporting material:

Concrete —0.25 of specified compressive strength
Granite —800 psi
Sandstone—400 psi
Limestone—400 psi

11.37.2 High Strength Bolts

Steel fabrication may be detailed for ⅞ in diameter A325 or A490 bolts in $^{15}\!/_{16}$ in diameter holes. The designer should determine the owner's requirements for fastener sizes, materials, use of oversize or slotted holes, etc. Often, ⅞ in diameter A325 high strength bolts are used because bridge owners generally have maintenance equipment for installing and removing these fasteners. Attention is directed to the AREMA *Manual,* Chapter 15 Commentary, for additional information.

High strength bolts must be installed to specified minimum tension values. The required tension for installed bolts of various sizes is given in Table 11.31.

11.38 FATIGUE DESIGN

Repetitive loading from locomotives and rolling stock can cause fatigue cracks to develop and grow in steel bridges. To guard against this possibility, a cyclic stress life is assigned to bridges, and allowable fatigue stresses are specified for various details as subsequently discussed. Often, however, the damage is caused by secondary loadings that were not considered in design. For example, live loads may deflect one girder more that an adjacent girder in a multigirder bridge. This can cause the cross frames connecting the girders to induce large out-of-plane distortions and transverse bending stresses in the girder webs. Such conditions can usually be avoided by careful detailing.

The number of stress cycles (N) assigned to bridge members for design is based on the bridge span length or the number of loaded tracks, depending upon the component. It is

TABLE 11.31 Required Tension for Installed Bolts in Railroad Bridges

	Tension, kips	
Bolt size, in	A 325 bolts	A 490 bolts
¾	28	35
⅞	39	49
1	51	64
1⅛	56	80
1¼	71	102

Source: Adapted from *AREMA Manual,* American Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.

TABLE 11.32 Number of Constant Stress Cycles for Design of Railroad Bridges

Component[a]	Span length, ft	No. of loaded tracks	Specified no. of stress cycles (N)
Truss chord members	$L > 100$	—	2,000,000
Longitudinal flexural members	$L \leq 100$	—	>2,000,000
Floor beams	—	1	2,000,000
		2	>2,000,000
Truss hangers and sub-diagonals that carry floor beam reactions only	—	1	2,000,000
		2	>2,000,000
Truss web members	—	1	2,000,000
		2	>2,000,000

[a] Includes member connections.
Source: Adapted from *AREMA Manual,* American Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.

assumed that the structure has been designed for specified loadings in accordance with recognized acceptable practices. As indicated in Table 11.32, the number of stress cycles specified for the various components falls into one of two categories—either 2,000,000 cycles or over 2,000,000 cycles. For bridge span lengths greater than 300 ft the number of relevant load cycles should be reviewed in accordance with the AREMA *Manual,* Commentary, Chapter 15.

The allowable fatigue stress range for various details has been determined by tests of large scale members. The details have been classified in categories designated A through F. In design, the member must be proportioned so that the stress range at each detail does not exceed the allowable range, which depends on N. Table 11.33 lists the allowable fatigue stress ranges for the various details in other than fracture critical members. The allowables are for bridges designed for E80 live loads. The details for the various categories are depicted

TABLE 11.33 Allowable Fatigue Stress Range for Details in Non-Fracture Critical Members

	Allowable fatigue stress range, ksi, for number of constant stress cycles	
Stress category	2,000,000	>2,000,000
A	24 ksi	24 ksi
B	15	16
B′	14.5	12
C	13	10
D	10	7
E	8	4.5
E′	5.8	2.6
F	9	8

Source: Adapted from *AREMA Manual,* American Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.

in the AREMA *Manual* and should be carefully reviewed in usage that may differ from highway bridge design. Also, refer to the AREMA *Manual* for the fatigue design of members or components designated as fracture critical. (See Art. 11.10 and 6.22.)

11.39 FRACTURE CRITICAL MEMBERS

For railway bridges, fracture critical members (FCM) are those members or components of members loaded in tension whose failure would be expected to result in collapse of the bridge, or would prevent the bridge from performing its design function. If the bridge cannot carry the assigned rail traffic, it is considered not performing its design function.

Tension components include all portions of tension members and the portion of flexural members subjected to tension stresses. Attachments welded to a tension component of a FCM and having a length of 4 in or more, measured in the direction of the tension stress, should be considered a part of the tension component and thus as a FCM.

11.39.1 Fracture Control Plan

Provisions for a formalized Fracture Control Plan should generally be included in the design specifications. The essence of the program is stated in six areas intended to cover special requirements for materials, fabrication, welding, inspection, and testing. The plan provisions given in the AREMA *Manual* are as follows:

1. Assign responsibility for designating which steel railway bridge members or member components, if any, fall in the category of Fracture Critical.
2. Require that fabrication of Fracture Critical Members or member components be done in plants having personnel, organization, experience, procedures, knowledge and equipment capable of producing quality workmanship.
3. Require that all welding inspectors demonstrate their competency to assure that welds in Fracture Critical Members or member components are in compliance with this plan.
4. Require that all nondestructive testing personnel demonstrate their competency to assure that, tested elements of Fracture Critical Members or member components are in compliance with this plan.
5. Specify material toughness values for Fracture Critical Members or member components.
6. Supplement recommendations for welding contained elsewhere in [AREMA *Manual*] Chapter 15, Steel Structures and in [American Welding Society] AWS D 1.5.

The designer should be familiar with the AREMA definition of the ''Engineer'' as being the chief engineering officer of the owning Company, or his authorized representatives, and secondly of their assignment of design and review responsibilities. These are stated by the AREMA *Manual* as follows:

1. Quite apart from the Fracture Control Plan, the Engineer is responsible, for the suitability of the design of the railway bridge; for the selection of the proper materials; for choosing adequate details; for designating appropriate weld requirements; and for reviewing shop drawings and erection plans to determine conformance with the contract documents.
2. As a part of the Fracture Control Plan, the Engineer is also responsible: for determining which, if any, bridge members or member components are in the FCM category; for evaluating each bridge design to determine the location of any FCM's that may exist; for the clear delineation on the contract plans of the location of all FCM's; for reviewing shop drawings to determine that they correctly show the location and extent of FCM's;

and for verifying that this Fracture Control Plan is properly implemented in compliance with contract documents at all stages of fabrication and erection.

3. Welding procedure specifications are considered an integral part of shop drawings and shall be reviewed for each contract.

11.39.2 Qualification Certifications

The fabricator for the structural steel should be certified under the American Institute of Steel Construction Quality Certification Program, Category III, Major Steel Bridges, or another program deemed suitable by the designer and acceptable to the owner. Welding inspector Qualifications and Certification, and Non-Destructive Testing Personnel Qualification and Certification, are detailed in the AREMA *Manual* and are generally known to fabricators of steel railway bridges. The designer may not participate in the fabrication or erection of the bridge; however, the calculations and plan preparation should contemplate the requirements for Fracture Critical construction.

11.39.3 Welding Requirements

Welding requirements should be in accord with AWS D 1.5 and the special requirements of the AREMA *Manual.* The designer should consider the effect of variables that pertain to Fracture Critical Fabrication, such as:

1. Minimum service temperature.
2. Material designation and grade.
3. Material thickness and requirements for Charpy V-notch impact testing.
4. Welding procedures, including preheat/interpass temperature requirements, moisture content for electrodes, hydrogen limits for wire and coating, qualification test plates, and procedure qualification for welding and repair welding.

The AREMA *Manual* Commentary provides both explanatory information on various articles and also supplemental recommendations, including useful charts and tables, as well as an index to help find specific provisions on welding.

11.40 IMPACT TEST REQUIREMENTS FOR STRUCTURAL STEEL

The resistance to fracture of steel for bridge fabrication is generally measured by the Charpy V-notch test. The grade of material, thickness of material, service temperature, and method of fastening (riveted, bolted, or welded construction) are considered in specifying the impact energy and temperature for the material. Impact requirements for A36 steel are given in Table 11.34 for non-fracture critical members and for fracture critical members. Note that the impact test requirements are more severe for FCM's. These and other requirements are provided in tabular form in the AREMA *Manual,* Chapter 15. Attention is directed to the requirements for welded fabrication.

11.41 GENERAL DESIGN PROVISIONS

The general rules that follow are based on the AREMA *Manual.* They should be used where applicable but may be modified to reflect specific owner's requirements.

TABLE 11.34 Charpy V-Notch Impact Requirements for A36 Structural Steel

Minimum service temperature, °F	Temperature zone designation	Minimum average energy, ft-lb
(a) Up to 6 in thickness, for use in other than fracture critical members.		
0 and above	1	15 at 70°F
−1 to −30	2	15 at 40°F
−31 to −60	3	15 at 10°F
(b) Up to 1½ in thickness, for use in fracture critical members.		
0 and above	1	25 at 70°F
−1 to −30	2	25 at 40°F
−31 to −60	3	25 at 10°F

Source: Adapted from *AREMA Manual,* American Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, MD 20785-2230.

11.41.1 Thickness of Material

Steel should generally not be less than 0.335 in thick, but fillers may be thinner. Where components are subject to corrosive conditions, they should be made thicker than otherwise required or should be protected. Some owners have adopted a more conservative minimum thickness of 0.50 in, except for fillers. Gusset plates used to connect chord and web members in trusses should be proportioned for the force transferred and should not be less than 0.50 in thick.

11.41.2 Slenderness Ratios

Slenderness ratios, expressed as the length divided by the least radius of gyration, should not exceed the following:

Main compression members	100
Wind and sway bracing in compression	120
Single lacing	140
Double lacing	200
Tension members	200

11.41.3 Fasteners and Net Section

The nominal diameter of fasteners should be used as the effective diameter. The effective bearing area of rivets and pins should be taken as the diameter multiplied by the length in bearing. For countersunk rivets, the bearing length should be reduced by one-half the depth of the countersink.

Fasteners should be arranged symmetrically about the axis of the member. The net section of a part should be taken as the thickness multiplied by the least net width of the part. The net section of a riveted or bolted tension member is the sum of the net sections of its parts, computed as the net width times the thickness.

The net width for a chain of holes extending across a part should be taken as the gross width, less the sum of the diameters of all holes in the chain, plus a quantity for each space in the chain computed as:

$$S^2/4\,g \tag{11.88}$$

where S = pitch of two successive holes in the chain in the direction of tensile stress and
g = gage of same two holes, in the transverse direction.

The net section of the part is determined by using the chain of holes that gives the least width. The net width should not be considered as more than 85% of the gross width. The diameter of the holes should be taken as ⅛ in more than the nominal size of the fastener.

Bolted or riveted connections should have not less than three fasteners per plane of connection, or the equivalent strength in welding. Fillet welds are preferred and should be parallel and symmetrical to the direction of the force.

Field connections should be made using rivets or high strength bolts. Field welds may be used for minor connections which are not subject to live load forces and for joining sections of deck plate or other items that do not function as part of the load carrying structure. Otherwise, field welding should not be used for connections.

Welds acting in the same connection with rivets and/or bolts should be proportioned and aligned to carry the entire force. Rivets and high strength bolts working in the same connection plane should be considered as sharing the force. When the connection is subjected to fatigue conditions, the stress category and allowable stress for rivets should be used for both types of fasteners.

11.42 COMPRESSION MEMBERS

Compression members should be configured so the main elements of the section are connected directly to the gusset plates, pins, or other members.

For members consisting of parts connected by lacing or solid cover plates, the minimum thickness of the web plate, should not be less than:

$$t_m = \frac{b\sqrt{F_y}}{6000\sqrt{P_c/f}} \tag{11.89}$$

and $\sqrt{P_c/f}$ must not exceed 2.0.

The thickness of the cover plate should not be less than

$$t_m = \frac{b\sqrt{F_y}}{7500\sqrt{P_c/f}} \tag{11.90}$$

and $\sqrt{P_c/f}$ must not exceed 2.0.

In the above expressions:

t_m = minimum thickness, in
b = unsupported distance between the nearest line of fasteners or welds, or between the roots of rolled flanges, in
P_c = allowable stress for the member in axial compression, psi
f = actual stress in compression, psi
F_y = yield point for the material, psi

11.42.1 Outstanding Elements In Compression

The width, in, of the outstanding elements of compression members should not exceed the following values expressed in terms of the element thickness, t, in, and the material yield point, F_y, psi.

Legs of angles or flanges of beams or tees:

For stringers and girders where ties rest on the flange: $\dfrac{1900t}{\sqrt{F_y}}$

For main members subject to axial force and for stringers and girders where ties do not rest on the flange: $\dfrac{2300t}{\sqrt{F_y}}$

For bracing and other secondary members: $\dfrac{2700t}{\sqrt{F_y}}$

Plates: $\dfrac{2300t}{\sqrt{F_y}}$

Stems of tees: $\dfrac{3000t}{\sqrt{F_y}}$

The width of the plate element should be taken as the distance from the free edge to the center of the first line of fasteners or welds. Angle legs and tee stems should be taken as the full nominal dimension. The flange of beams and tees should be measured from the free edge to the toe of the fillet. If the projecting element exceeds the above width but could be made to conform if a part of its width were considered removed, and if that reduced section would be satisfactory for stress requirements, the element should be considered acceptable.

11.43 STAY PLATES

Compression and tension members with segments connected with lacing bars should be detailed with stay plates. Lacing and perforated cover plates should be designed for the lateral shear force normal to the member. The total shear force should include forces due to the weight of the member, other imposed forces and for compression members, 2.5% of the compressive axial force; but not less than $AF_y/150$ where A = member area required for axial compression (in^2) and F_y = yield point for material (psi). See AREMA *Manual,* Chapter 15, Section 1.6 for additional recommendations for laced members.

11.44 MEMBERS STRESSED PRIMARILY IN BENDING

Rolled beams and fabricated girder spans provide economical bridges for railways. Laying out the lateral bracing system and diaphragms or cross frames is the first step in the design procedure. Some of the requirements for the bracing have grown out of railroad bridge experience, including some fatigue failures that required changes in previously acceptable practices.

11.44.1 Lateral Bracing

A system of bottom lateral bracing should be provided for all spans more than 50 ft in length. Deck spans employing four or more beams per track, where the beams are less than 72 in deep, and where a reinforced concrete deck is integrated with the beams by shear connectors or where a cast in place concrete deck engages not less than 1 in of the beam flange thickness, do not require the bottom lateral bracing.

There should be top lateral bracing in all deck spans and in through spans, provided head room is adequate.

Where the floor system is designed to do so, it may be used to provide the required lateral bracing in its plane.

Double system bracing may be treated as acting simultaneously in a panel, provided the members meet the requirements for tension and compression.

Top flanges of through plate girders should be braced by brackets (knees). Brackets should be attached to the top flange of the floor beams and to stiffeners on the girders. The bracket should be as wide as practical and should extend to the top flange. In proportioning the bracket, some designers use a load acting horizontally through the flange equal to 2.5% of the maximum allowable capacity of the flange. This can be expressed as $0.025 \times 0.55\ F_y \times$ flange area. For spans that have solid floors, the bracket spacing should not exceed 12 ft.

11.44.2 Cross Frames and Diaphragms For Deck Spans

Although cross frames and diaphragms have generally not been specifically designed for lateral distribution of loads, such distribution is inherent in typical construction.

The following are recommended practices:

1. Longitudinal girders and beams that are more than 42 in deep and which are spaced more than 48 in apart should be braced with cross frames. Cross frame diagonals should make an angle with the vertical that does not exceed 60 degrees. Minimum steel thickness and number of fasteners should be indicated in the design. Cross frames or diaphragms should be used at the ends of spans and should be proportioned for lateral and centrifugal loads, as well as jacking loads, if required. Where girders or beam ends frame into a floor-beam, cross frames or diaphragms are not required.

2. Cross frames and diaphragms, and their connections should be adequate to resist forces induced by out-of-plane bending and lateral loads. Connection plates for cross frames and diaphragms between beams or girders subject to out-of-plane bending should be adequately fastened to the web and both the top and bottom flanges of the beams or girders. Diaphragm and cross frame spacing may be made coincident with the stiffener spacing. The requirement to fasten the connection plate to the tension flange of the girder requires special attention in welded fabrication.

3. Longitudinal beams and girders of depth and spacing that do not require cross frames should be braced with rolled shape diaphragms that are as deep as the beam or girder will permit. Wide flange sections are frequently used, but channels may be an economical alternate. The connection for these diaphragms should be designed to carry shear of at least 50% of the shear capacity of the diaphragm.

4. On ballasted deck bridges utilizing closely spaced transverse floor beams, the beams should be connected with one or more lines of longitudinal diaphragms for each track.

5. The spacing of diaphragms or cross frames should be as follows:

For open deck construction:	18 ft maximum
For ballast deck construction with top lateral bracing:	18 ft maximum

For ballast deck construction without top lateral bracing:	12 ft maximum
For ballast deck construction with cast in place concrete decks that are integrated with the beams or girders:	24 ft maximum

Where a cast-in-place concrete deck is used and the girders and beams are 54 in deep or less, a concrete diaphragm may be used, provided the reinforcing extends through the web and is developed in the adjacent concrete.

11.44.3 Net and Gross Sections

Plate girders, rolled beams and other members loaded in bending that produces tension on one face of the member and compression on the other should be proportioned by the moment of inertia method. Considering the neutral axis as the center of gravity of the gross section, tensile stress should be calculated using the moment of inertia of the entire net section and compressive stress should be calculated using the moment of inertia of the entire gross section.

11.44.4 Considerations for Flanges

Where not fully supported laterally, the compression flange of a flexural member should be supported at points so that the ratio of the distance between points and radius of gyration of the flange plus the part of the web on the compression side of the neutral axis does not exceed $29{,}000/\sqrt{F_y}$, where F_y is the yield point of the material, psi.

In open deck construction, ties may be seated on the top flange. Tie deflection loads the flange non-uniformly with the passage of each wheel. The minimum thickness for flange angles should be ⅝ in if cover plates are used and ¾ in where cover plates are not used. Flanges of plate girders should be proportioned without the use of side plates.

Where cover plates are used, at least one plate of each flange should run the full length of the span. Partial length cover plates should be avoided, but where they are used, they should extend far enough beyond the theoretical end to develop the plate and to a section where the stress in the flange, without the cover plate, is not greater than the allowable fatigue stress.

In welded construction, only one plate should be used for the flange. Side plates should not be used. Flange plate width and thickness may be varied in the length of the member using appropriate welds and transitions. Where the ties will sit on the top flange, consider the following:

1. Wider flanges are subject to more flexure as the tie deflects.
2. The top surface of the flange should be one plane; therefore, adjust the depth of web if the flange thickness changes.
3. If the flange width changes, adjust tie (dapping) to fit the sections.
4. The flange width should accommodate tie hold down devices without fouling guard timbers. Tie hold downs should go on the field side of the flange to avoid tie skew.

Only one cover plate should be used on the flange of a rolled beam. The cover plate should be of one thickness, full length and should be connected to the beam flange with continuous fillet welds. The cover plate thickness should be not more than 1.5 times the thickness of the beam flange and should meet the minimum thickness requirements. Beam flanges supporting ties frequently experience mechanical wear and corrosion. Tie bearing area on the cover plate should be at least as great as the bearing area of the tie plate.

11.44.5 Web Plates

Web plates of beams and girders must meet the minimum thickness requirements. Depending on detailing and service conditions, web plates are prone to spot corrosion. The thickness of the web plate should not be less than ⅙ the thickness of the flange, nor less than a thickness (in) of ($\sqrt{F_y}/30{,}000$) times the clear distance between the flanges, (in) where F_y is the material yield point, (psi).

11.44.6 Flange-to-Web Connections

Flange-to-web joints of welded plate girders should be the same for top and bottom flanges. For open deck construction, use full penetration groove welds. For ballast deck construction, with either steel plate or composite concrete, use full penetration groove welds or continuous fillet welds.

Flange angles in riveted or bolted construction should be connected to the web with enough fasteners to transfer the horizontal shear force to the flange section. The connection must also be designed for the effects of any load applied directly to the flange, at any point. When ties bear on the flange, a wheel load plus 80% impact should be assumed to be distributed over 3 ft of flange length. On ballast deck construction, the same load should be assumed to be spread over 5 ft of flange length.

11.44.7 Stiffeners at Bearing Points

Stiffeners should be provided in pairs, opposite each other, at the centerline of the end bearing of plate girders and beams. Appropriately positioned pairs of stiffeners should be placed at all points of concentrated loads. Stiffener width should be as wide as the flange will accommodate, and the stiffener connection to the web should have the capacity to transmit the load. Angle stiffeners should not be crimped. Plate stiffeners should be clipped top and bottom to clear the fillet of the flange to web interface.

The outstanding element of the bearing stiffener should meet the width to thickness requirements for compression elements. Bearing stiffeners may be designed as a column, using the pair of stiffeners and a strip of the web whose width is equal to 25 times the thickness of the web. For stiffeners located at the end of the web, the web column width should be taken as 12 times its thickness.

The effective length should be taken as ¾ of the stiffener length in determining L/r. Stiffeners should also be designed for bearing, without considering any part of the web and using only the end area actually in contact with the flange. Where stiffeners are welded to the flange with full penetration groove welds, the bearing area should be taken as the length of the weld times the thickness of the stiffener.

11.44.8 Intermediate Stiffeners

In railway bridges, intermediate stiffeners are frequently spaced such that they can be used as connection plates for cross frames and diaphragms. This practice, and the requirement that the connection plate be attached to the top and bottom flange, results in low allowable fatigue stresses when welded fabrication is used. The designer should review this detail early in the design process. A bolted connection may prove more economical.

Where the web depth between flanges (in) exceeds $11{,}400t/\sqrt{F_y}$, it should be stiffened by pairs of plates or angles. Also, the clear distance between pairs of stiffeners should not exceed 72 in or $10{,}500t/\sqrt{f_v}$. In these expressions, t = web thickness, (in) F_y = web material yield point, psi, and f_v = shear stress in gross section of web at point under review, psi.

Intermediate stiffeners on one side of the web may be used instead, on condition that they provide a moment of inertia equal to that of the minimum acceptable pair of plates or angles. They should be connected to the outstanding portion of the compression flange and, if they are used as connection plates for cross frames or diaphragms, they must be connected to the bottom flange. In open deck construction when ties bear on the top flange, experience shows that the flange should be stiffened on the track side of the web.

The width (in) of the outstanding element of a stiffener should not exceed 16 times its thickness and should not be less than 2 in plus $\frac{1}{30}$ of the girder depth (in).

11.45 OTHER CONSIDERATIONS

11.45.1 Expansion

The design of steel railway bridges should allow for a change in length due to temperature change of 1 in per 100 ft of span. Also, provision should be made for change in length from live load. In truss spans more than 300 ft long, allowance should be made for expansion of the floor system. The use of high adhesion locomotives may justify a more vigorous review of floor system expansion in even shorter spans.

11.45.2 End Bearings

In accord with current practice, spans more than 70 ft long should have hinged bearings at both ends and rollers or rockers at the expansion end. Spans 70 ft or less should be designed to slide on self-lubricating bronze plates at least ½ in thick. All end bearings should be secure against lateral and vertical movement and, when founded on masonry, should be raised above the seat on metal bolsters or pedestals. New provisions are being developed by a committee of AREMA established "to formulate specific and detailed recommendations for the construction of bearings for non-movable railway bridges."

11.45.3 Bridge Deck Drainage

On ballast deck construction, the deck drainage is part of the steel design. In general, deck drainage is gathered in scuppers or drops and carried in closed conduits to drops at the piers or abutments. Deck drains are frequently specified to be ductile iron pipe, supported in brackets to secure it against movement from vibration. Drainage systems require a grade of at least 1%.

Water accumulation must be kept away from expansion ends on ballast deck bridges where the bridge movement is accommodated by steel plates sliding under the ballast. Stainless steel plate and fasteners should be used in these expansion joints. At abutments, where bridges are built in a track grade, the water coming to the bridge should be intercepted in designed drains in the embankment behind the back wall.

11.45.4 Protective Coatings

Steel bridges should be protected with a quality paint system, metallic coating, or other protective system approved by the owner. A full system shop coating may be appropriate for some structures. Many bridge owners have developed their own coating specifications. Design details should enhance the service life of the coating by providing for drainage and avoiding accumulation of dirt and debris.

11.45.5 Miscellaneous

Railway bridges may require provisions for walkways, handrails, access ladders, lights, signals, and signs; supports for conduits, communication lines, and fiber optic ducts; as well as track equipment, and other ancillary devices. The bridge designer should make provision for such items as directed by the owner. Bridge plates showing ownership and date of erection may also be required.