Sorgenfrei, D.F., Marianos, Jr. W.N. "Railroad Bridges."
Bridge Engineering Handbook.
Ed. Wai-Fah Chen and Lian Duan
Boca Raton: CRC Press, 2000

23

Railroad Bridges

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23.1 Introduction

23.1.1 Railroad Network

The U.S. railroad network consists predominantly of privately owned freight railroad systems classified according to operating revenue, the government-owned National Railroad Passenger Corporation (Amtrak), and numerous transit systems owned by local agencies and municipalities.

Since the deregulation of the railroad industry brought about by the 1980 Staggers Act, there have been numerous railway system mergers. By 1997 there remained 10 Class I (major) Railroads, 32 Regional Railroads, and 511 Local Railroads operating over approximately 150,000 track miles. The 10 Class I Railroads comprise only 2% of the number of railroads in the United States but account for 73% of the trackage and 91% of freight revenue.

By far the present leading freight commodity is coal, which accounts for 25% of all the carloads. Other leading commodities in descending order by carloads are chemicals and allied products, farm products, motor vehicles and equipment, food and sundry products, and nonmetallic minerals. Freight equipment has drastically changed over the years in container type, size and wheelbase, and carrying capacity. The most predominant freight car is the hopper car used with an open top for coal loading and the covered hopper car used for chemicals and farm products. In more recent years special cars have been developed for the transportation of trailers, box containers, and automobiles. The

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average freight car capacity (total number of freight cars in service divided by the aggregate capacity of those cars) has risen approximately 10 tons each decade with the tonnage ironically matching the decades, i.e., 1950s — 50 tons, 1960s — 60 tons, and so on. As the turn of the century approaches, various rail lines are capable of handling 286,000 and 315,000-lb carloads, often in dedicated units.

In 1929 there were 56,936 steam locomotives in service. By the early 1960s they were nearly totally replaced by diesel electric units. The number of diesel electric units has gradually decreased as available locomotive horsepower has increased. The earlier freight trains were commonly mixed freight of generally light railcars, powered by heavy steam locomotives. In more recent years that has given way to heavy railcars, unit trains of common commodity (coal, grain, containers, etc.) with powerful locomotives. Newer locomotives generally have six axles, weigh 420,000 lbs, and can generate up to 8000 Hp.

These changes in freight hauling have resulted in concerns for railroad bridges, many of which were not designed for these modern loadings. The heavy, steam locomotive with steam impact governed in design considerations. Present bridge designs are still based on the steam locomotive wheel configuration with diesel impact, but fatigue cycles from the heavy carloads are of major importance.

The railroad industry records annual route tonnage referred to as "million gross tons" (MGT). An experienced railroader can fairly well predict conditions and maintenance needs for a route based on knowing the MGT for that route. It is common for Class I Railroads to have routes of 30 to 50 MGT with some coal routes in the range of 150 MGT.

Passenger trains are akin to earlier freight trains, with one or more locomotives (electric or diesel) followed by relatively light cars. Likewise, transit cars are relatively light.

23.1.2 Basic Differences between Railroad and Highway Bridges

A number of differences exist between railroad and highway bridges:

1. The ratio of live load to dead load is much higher for a railroad bridge than for a similarly sized highway structure. This can lead to serviceability issues such as fatigue and deflection control governing designs rather than strength.
2. The design impact load on railroad bridges is higher than on highway structures.
3. Simple-span structures are preferred over continuous structures for railroad bridges. Many of the factors that make continuous spans attractive for highway structures are not as advantageous for railroad use. Continuous spans are also more difficult to replace in emergencies than simple spans.
4. Interruptions in service are typically much more critical for railroads than for highway agencies. Therefore, constructibility and maintainability without interruption to traffic are crucial for railroad bridges.
5. Since the bridge supports the track structure, the combination of track and bridge movement cannot exceed the tolerances in track standards. Interaction between the track and bridge should be considered in design and detailing.
6. Seismic performance of highway and railroad bridges can vary significantly. Railroad bridges have performed well during seismic events.
7. Railroad bridge owners typically expect a longer service life from their structures than highway bridge owners expect from theirs.

23.1.3 *Manual for Railway Engineering,* AREMA

The base document for railroad bridge design, construction, and inspection is the American Railway Engineering Maintenance of Way Association (AREMA) *Manual for Railway Engineering* (*Manual*) [1].

Early railroads developed independent specifications governing the design loadings, allowable strains, quality of material, fabrication, and construction of their own bridges. There was a proliferation of specifications written by individual railroads, suppliers, and engineers. One of the earliest general specifications is titled *Specification for Iron Railway Bridges and Viaducts,* by Clarke, Reeves and Company (Phoenix Bridge Company). By 1899 private railroads joined efforts in forming AREMA. Many portions of those original individual railroad specifications were incorporated into the first manual titled *Manual of Recommended Practice for Railway Engineering and Maintenance of Way* published in 1905. In 1911 the Association dropped "Maintenance of Way" from its name and became the American Railway Engineering Association (AREA); however, in 1997 the name reverted back to the original name with the consolidation of several railroad associations.

The *Manual* is not deemed a specification but rather a recommended practice. Certain provisions naturally are standards by necessity for the interchange of rail traffic, such as track gauge, track geometrics, clearances, basic bridge loading, and locations for applying loadings. Individual railroads may, and often do, impose more stringent design requirements or provisions due to differing conditions peculiar to that railroad or region of the country, but basically all railroads subscribe to the provisions of the *Manual*.

Although the *Manual* is a multivolume document, bridge engineering provisions are grouped in the *Structural Volume* and subdivided into applicable chapters by primary bridge material and special topics, as listed:

Chapter 7	Timber Structures
Chapter 8	Concrete Structures and Foundations
Chapter 9	Seismic Design for Railway Structures
Chapter 10	Structures Maintenance & Construction (New)
Chapter 15	Steel Structures
Chapter 19	Bridge Bearings
Chapter 29	Waterproofing

The primary structural chapters each address bridge loading (dead load, live load, impact, wind, seismic, etc.) design, materials, fabrication, construction, maintenance/inspection, and capacity rating. There is uniformity among the chapters in the configuration of the basic live load, which is based on the Cooper E-series steam locomotive. The present live-load configuration is two locomotives with tenders followed by a uniform live load as shown in Fig. 23.1. There is not uniformity in the chapters in the location and magnitude of many other loads due to differences in the types of bridges built with different materials and differences in material behavior. Also it is recognized that each chapter has been developed and maintained by separate committee groups of railroad industry engineers, private consulting engineers, and suppliers. These committees readily draw from railroad industry experiences and research, and from work published by other associations such as AASHTO, AISC, ACI, AWS, APWA, etc.

23.2 Railroad Bridge Philosophy

Railroad routes are well established and the construction of new railroad routes is not common; thus, the majority of railroad bridges built or rehabilitated are on existing routes and on existing right-of-way. Simply stated, the railroad industry first extends the life of existing bridges as long as economically justified. It is not uncommon for a railroad to evaluate an 80- or 90-year-old bridge, estimate its remaining life, and then rehabilitate it sufficiently to extend its life for some economical period of time.

Bridge replacement generally is determined as a result of a lack of load-carrying capacity, restrictive clearance, or deteriorated physical condition. If bridge replacement is necessary, then simplicity, cost, future maintenance, and ease of construction without significant rail traffic disruptions typically govern the design. Types of bridges chosen are most often based on the capability of a railroad to do its own construction work. Low-maintenance structures, such as ballasted deck prestressed concrete box-girder spans with concrete caps and piles, are preferred by some railroads. Others may prefer weathering steel elements.

In a review of the existing railroad industry bridge inventory, the majority of bridges by far are simple-span structures over streams and roadways. Complex bridges are generally associated with crossing major waterways or other significant topographical features. Signature bridges are rarely constructed by railroads. The enormity of train live loads generally preclude the use of double-leaf bascule bridges and suspension and cable-stayed bridges due to bridge deflection and shear load transfer, respectively. Railroads, where possible, avoid designing skewed or curved bridges, which also have inherent deflection problems.

When planning the replacement of smaller bridges, railroads first determine if the bridge can be eliminated using culverts. A hydrographic review of the site will determine if the bridge opening needs to be either increased or can be decreased.

The *Manual* provides complete details for common timber structures and for concrete box-girder spans. Many of the larger railroads develop common standards, which provide complete detailed plans for the construction of bridges. These plans include piling, pile bents, abutments and wing walls, spans (timber, concrete, and steel), and other elements in sufficient detail for construction by in-house forces or by contract. Only site-specific details such as permits, survey data, and soil conditions are needed to augment these plans.

Timber trestles are most often replaced by other materials rather than in kind. However, it is often necessary to renew portions of timber structures to extend the life of a bridge for budgetary reasons. Replacing pile bents with framed bents to eliminate the need to drive piles or the adding of a timber stringer to a chord to increase capacity is common. The replacement of timber trestles is commonly done by driving either concrete or steel piling through the existing trestle, at twice the present timber span length and offset from the existing bents. This is done between train movements. Either precast or cast-in-place caps are installed atop the piling beneath the existing timber deck. During a track outage period, the existing track and timber deck is removed and new spans (concrete box girders or rolled steel beams) are placed. In this type of bridge renewal, key factors are use of prefabricated bridge elements light enough to be lifted by railroad track mounted equipment (piles, caps, and spans), speed of installation of bridge elements between train movements, bridge elements that can be installed in remote site locations without outside support, and overall simplicity in performing the work.

The railroad industry has a large number of 150 to 200 ft span pin-connected steel trusses, many with worn joints, restrictive clearances, and low carrying capacity, for which rehabilitation cannot be economically justified. Depending on site specifics, a common replacement scenario may be to install an intermediate pier or bent and replace the span with two girder spans. Railroad forces have perfected the technique of laterally rolling out old spans and rolling in new prefabricated spans between train movements.

Railroads frequently will relocate existing bridge spans to other sites in lieu of constructing new spans, if economically feasible. This primarily applies to beam spans and plate girder spans up to 100 ft in length.

In general, railroads prefer to construct new bridges online rather than relocating or doglegging to an adjacent alignment. Where site conditions do not allow ready access for direct span replacement, a site bypass, or runaround, called a "shoofly" is constructed which provides a temporary bridge while the permanent bridge is constructed.

The design and construction of larger and complex bridges is done on an individual basis.

23.3 Railroad Bridge Types

Railroad bridges are nearly always simple-span structures. Listed below in groupings by span length are the more common types of bridges and materials used by the railroad industry for those span lengths.

Short spans	to 16 ft	Timber stringers
		Concrete slabs
		Rolled steel beams
	to 32 ft	Conventional and prestressed concrete box girders and beams
		Rolled steel beams
	to 50 ft	Prestressed concrete box girders and beams
		Rolled steel beams, deck and through girders
Medium spans, 80 to 125 ft		Prestressed concrete beams
		Deck and through plate girders
Long spans		Deck and through trusses (simple, cantilever, and arches)

Suspension bridges are not used by freight railroads due to excessive deflection.

23.4 Bridge Deck

23.4.1 General

The engineer experienced in highway bridge design may not think of the typical railroad bridge as having a deck. However, it is essential to have a support system for the rails. Railroad bridges typically are designed as either open deck or ballast deck structures. Some bridges, particularly in transit applications, use direct fixation of the rails to the supporting structure.

23.4.2 Open Deck

Open deck bridges have ties supported directly on load-carrying elements of the structure (such as stringers or girders). The dead loads for open deck structures can be significantly less than for ballast deck structures. Open decks, however, transfer more of the dynamic effects of live load into the bridge than ballast decks. In addition, the bridge ties required are both longer and larger in cross section than the standard track ties. This adds to their expense. Bridge tie availability has declined, and their supply may be a problem, particularly in denser grades of structured timber.

TABLE 23.1 Weight of Rails, Inside Guard Rails, Ties, Guard Timbers, and Fastenings for Typical Open Deck (Walkway not included)

Item	Weight (plf of track)
Rail (136 RE):	
(136 lb/lin. yd × 2 rails/track × 1 lin. yd/3 lin. ft)	91
Inside guard rails:	
(115 lb/lin. yd × 2 rails/track × 1 lin. yd/3 lin. ft)	77
Ties (10 in. × 10 10 ft bridge ties):	
(10 in. × 10 in. × 10 ft × 1 ft^2/144 in.^{2} × 60 lb/ft^3 × 1 tie/14 in. × 12 in./1 ft)	357
Guard Timbers (4 × 8 in.):	
(4 in. × 8 in. × 1 ft × 1 ft^2/144 in.^{3} × 60 lb/1 ft^3 × 2 guard timbers/ft)	27
Tie Plates (7¾ × 14¾ in. for rail with 6 in. base):	
24.32 lb/plate × 1 tie/14 in. × 12 in./ft × 2 plates/tie)	42
Spikes (⅝ × ⅝ in. × 6 in. reinforced throat)	
(0.828 lb/spike × 18 spikes/tie × 1 tie/14 in. × 12 in./1 ft)	13
Miscellaneous Fastenings (hook bolts and lag bolts):	
(Approx. 2.25 lb/hook bolt + 1.25 lb/lag screw × 2 bolts/tie × 1 tie/14 in. × 12 in./ft)	6
Total weight	613

TABLE 23.2 Weight of Typical Ballast Deck

Item	Weight (plf of track)
Rail (136 RE):	
(136 lb/lin. yd. × 2 rails/track × 1 lin. yd/3 lin. ft)	91
Inside Guard Rails:	
(115 lb/lin. yd × 2 rails/track × 1 lin. yd/3 lin. ft)	77
Ties (neglect, since included in ballast weight)	—
Guard Timbers (4 × 8 in.):	
(4 in. × 8 in. × 1 ft × 1 ft^2/144 in.^{2} × 60 lb/1 ft^3 × 2 guard timbers/ft)	27
Tie Plates (7¾ × 14¾ in. for rail with 6 inc. base):	
(24.32 lb/plate × 1 tie/19.5 in. × 12 in./ft × 2 plates/tie)	30
Spikes (⅝ × ⅝ × 6 in. reinforced throat)	
(0.828 lb/spike × 18 spikes/tie × 1 tie/19.5 in. × 12 in./1 ft)	9
Ballast (assume 12 in. additional over time)	
(Approx. 120 lb/ft^3 × 27 in. depth/12 in./1 ft × 16 ft)	4320
Waterproofing:	
(Approx. 150 lb/ft^3 × 0.75 in. depth/12 in./1 ft × 20 ft)	188
Total weight:	4742

23.4.3 Ballast Deck

Ballast deck bridges have the track structure supported on ballast, which is carried by the structural elements of the bridge. Typically, the track structure (rails, tie plates, and ties) is similar to track constructed on grade. Ballast deck structures offer advantages in ride and maintenance requirements. Unlike open decks, the track alignment on ballast deck spans can typically be maintained using standard track maintenance equipment. If all other factors are equal, most railroads currently prefer ballast decks for new structures.

In ballast deck designs, an allowance for at least 6 in. of additional ballast is prudent. Specific requirements for additional ballast capacity may be provided by the railroad. In addition, the required depth of ballast below the tie should be verified with the affected railroad. Typical values for this range from 8 to 12 in. or more. The tie length used will have an effect on the distribution of live-load effects into the structure. Ballast decks are also typically waterproofed. The weight of waterproofing should be included in the dead load. Provisions for selection, design, and installation of waterproofing are included in Chapter 29 of the AREMA *Manual*.

23.4.4 Direct Fixation

Direct fixation structures have rails supported on plates anchored directly to the bridge deck or superstructure. Direct fixation decks are much less common than either open decks or ballast decks and are rare in freight railroad service. While direct fixation decks eliminate the dead load of ties and ballast, and can reduce total structure height, they transfer more dynamic load effects into the bridge. Direct fixation components need to be carefully selected and detailed.

23.4.5 Deck Details

Walkways are frequently provided on railroad bridge decks. They may be on one or both sides of the track. Railroads have their own policies and details for walkway placement and construction.

Railroad bridge decks on curved track should allow for superelevation. With ballast decks, this can be accomplished by adjusting ballast depths. With open decks, it can require the use of beveled ties or building the superelevation into the superstructure.

Continuous welded rail (CWR) is frequently installed on bridges. This can affect the thermal movement characteristics of the structure. Check with the affected railroad for its policy on anchorage of CWR on structures. Long-span structures may require the use of rail expansion joints.

23.5 Design Criteria

23.5.1 Geometric Considerations

Railroad bridges have a variety of geometric requirements. The AREMA *Manual* has clearance diagrams showing the space required for passage of modern rail traffic. It should be noted that lateral clearance requirements are increased for structures carrying curved track. Track spacing on multiple-track structures should be determined by the affected railroad. Safety concerns are leading to increased track-spacing requirements.

If possible, skewed bridges should be avoided. Skewed structures, however, may be required by site conditions. A support must be provided for the ties perpendicular to the track at the end of the structure. This is difficult on open deck structures. An approach slab below the ballast may be used on skewed ballast deck bridges.

23.5.2 Proportioning

Typical depth-to-span length ratios for steel railroad bridges are around 1:12. Guidelines for girder spacing are given in Chapter 15 of the *Manual*.

23.5.3 Bridge Design Loads

23.5.3.1 Dead Load

Dead load consists of the weight of the structure itself, the track it supports, and any attachments it may carry. Dead loads act due to gravity and are permanently applied to the structure. Unit weights for calculation of dead loads are given in AREMA Chapters 7, 8, and 15. The table in Chapter 15 is reproduced below:

Unit Weights for Dead Load Stresses

Type	Pounds per Cubic Foot
Steel	490
Concrete	150
Sand, gravel, and ballast	120
Asphalt-mastic and bituminous macadam	150
Granite	170
Paving bricks	150
Timber	60

Dead load is applied at the location it occurs in the structure, typically as either a concentrated or distributed load.

The *Manual* states that track rails, inside guard rails, and rail fastenings shall be assumed to weigh 200 pounds per linear foot (plf) of track. The 60 pound per cubic foot weight given for timber should be satisfactory for typical ties. Exotic woods may be heavier. Concrete ties are sometimes used, and their heavier weight should be taken into account if their use is anticipated.

In preliminary design of open deck structures, a deck weight of 550 to 650 plf of track can be assumed. This should be checked with the weight of the specific deck system used for final design. Example calculations for track and deck weight for open deck and ballast deck structures are included in this chapter.

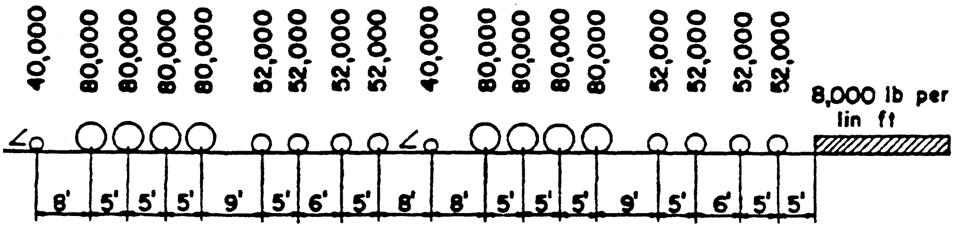

FIGURE 23.1 Cooper E8O live load.

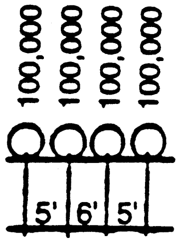

FIGURE 23.2 Alternate live load.

Railroad bridges frequently carry walkways and signal and communication cables and may be used by utilities. Provisions (both in dead load and physical location) may need to be made for these additional items. Some structures may even carry ornamental or decorative items.

23.5.3.2 Live Load

Historically, freight railroads have used the Cooper E load configuration as a live-load model. The Cooper E80 load is currently the most common design live load. The E80 load model is shown in Figure 23.1. The 80 in E80 refers to the 80 kip weight of the locomotive drive axles. An E60 load has the same axle locations, but all loads are factored by 60/80. Some railroads are designing new structures to carry E90 or E100 loads.

The Cooper live-load model does not match the axle loads and spacings of locomotives currently in service. It did not even reflect all locomotives at the turn of the 20th century, when it was introduced by Theodore Cooper, an early railroad bridge engineer. However, it has remained in use throughout the past century. One of the reasons for its longevity is the wide variety of rail rolling stock that has been and is currently in service. The load effects of this equipment on given spans must be compared, as discussed in Section 23.6. The Cooper live-load model gives a universal system with which all other load configurations can be compared. Engineering personnel of each railroad can calculate how the load effects of each piece of equipment compare to the Cooper loading.

The designated steel bridge design live load also includes an "Alternate E80" load, consisting of four 100-kip axles. This is shown in Figure 23.2. This load controls over the regular Cooper load on shorter spans.

A table of maximum load effects over various span lengths is included in Chapter 15, Part 1 of the AREMA *Manual*.

23.5.3.3 Impact

Impact is the dynamic amplification of the live-load effects on the bridge caused by the movement of the train across the span. Formulas for calculation of impact are included in Chapters 8 and 15 of the AREMA manual. The design impact values are based on an assumed train speed of 60 mph. It should be noted that the steel design procedure allows reduction of the calculated impact for ballast deck structures. Different values for impact from steam and diesel locomotives are used. The steam impact values are significantly higher than diesel impact over most span lengths.

Impact is not applied to timber structures, since the capacity of timber under transient loads is significantly higher than its capacity under sustained loads. Allowable stresses for timber design are based on the sustained loads.

23.5.3.4 Centrifugal Force

Centrifugal force is the force a train moving along a curve exerts on a constraining object (track and supporting structure) which acts away from the center of rotation. Formulas or tables for calculation of centrifugal force are included in Chapters 7, 8, and 15 of the AREMA manual. The train speed required for the force calculation should be obtained from the railroad.

Although the centrifugal action is applied as a horizontal force, it can produce overturning moment due to its point of application above the track. Both the horizontal force and resulting moment must be considered in design or evaluation of a structure.

The horizontal force tends to displace the structure laterally:

- For steel structures (deck girders, for example), it loads laterals and cross frames.
- For concrete structures (box girders, for example), the superstructure is typically stiff enough in the transverse direction that the horizontal force is not significant for the superstructure.

For all bridge types, the bearings and substructure must be able to resist the centrifugal horizontal force.

The overturning moment tends to increase the live-load force in members on the outside of the curve and reduce the force on inside members. However, interior members are not designed with less capacity than exterior members. Substructures must be designed to resist the centrifugal overturning moment. This will increase forces toward the outside of the curve in foundation elements. The centrifugal force is applied at the location of the axles along the structure, 6 ft above the top of rail, at a point perpendicular to the center of a line connecting the rail tops. The effect of track superelevation may compensate somewhat for centrifugal force. The plan view location of the curved track on the bridge (since railroad bridge spans are typically straight, laid out along the curve chords) can also be significant. Rather than applying the centrifugal force at each axle location, some railroads simply increase the calculated live-load force by the centrifugal force percentage, factor in the effect of the force location above the top of rail, and use the resulting value for design.

23.5.3.5 Lateral Loads from Equipment

This item includes all lateral loads applied to the structure due to train passage, other than centrifugal force. The magnitude and application point of these loads varies among Chapters 7, 8, and 15. For timber, a load of 20 kips is applied horizontally at the top of rail. For steel, a load of one quarter of the heaviest axle of the specified live load is applied at the base of rail. In both cases, the lateral load is a moving concentrated load that can be applied at any point along the span in either horizontal direction. It should be noted that lateral loads from equipment are not included in design of concrete bridges. However, if concrete girders are supported on steel or timber substructures, lateral loads should be applied to the substructures.

Lateral loads from equipment are applied to lateral bracing members, flanges of longitudinal girders or stringers without a bracing system, and to chords of truss spans. Experience has shown that very high lateral forces can be applied to structures due to lurching of certain types of cars. Wheel hunting is another phenomenon that applies lateral force to the track and structure. Damaged rolling stock can also create large lateral forces.

It should be noted that there is not an extensive research background supporting the lateral forces given in the AREMA *Manual*. However, the lateral loads in the *Manual* have historically worked well when combined with wind loads to produce adequate lateral resistance in structures.

23.5.3.6 Longitudinal Force from Live Load

Longitudinal forces are typically produced from starting or stopping trains (acceleration or deceleration) on the bridge. They can be applied in either longitudinal direction. These forces are transmitted through the rails and distributed into the supporting structure.

Chapters 7, 8, and 15 all take the longitudinal force due to braking to be 15% of the vertical live load, without impact. The chapters differ slightly in their consideration of the acceleration (traction) aspect of the force. Chapter 7 uses 25% of the drive axle loads for traction, while Chapters 8 and 15 use 25% of the axles of the regular Cooper E80 train configuration. In each chapter, the braking and traction forces are compared, and the larger value used in design. Chapters 7, 8, and 15 differ in the point of application of the longitudinal force. Chapter 7 applies it 6 ft above the top of rail. Chapters 8 and 15 apply the braking force at 8 ft above the top of rail and the traction force 3 ft above the top of rail.

All three chapters recognize that some of the longitudinal force is carried through the rails off the structure. (The extent of this transfer depends on factors such as rail continuity, rail anchorage, and the connection of the bridge deck to the span.) Where a large portion of the longitudinal force is carried to the abutments or embankment, Chapter 7 allows neglecting longitudinal force in the design of piles, posts, and bracing of bents. Chapters 8 and 15 allow taking the applied longitudinal force as half of what was initially calculated on short (<200 feet) ballast deck bridges with short spans (<50 feet), if the continuity of members or frictional resistance will direct some of the longitudinal force to the abutments.

Chapters 8 and 15 also state that the longitudinal load is to be applied to one track only, and can be distributed to bridge components based on their relative stiffness and the types of bearings. For multiple-track structures, it may be prudent to include longitudinal force on more than one track, depending on the bridge location and train operation at the site.

Longitudinal force is particularly significant in long structures, such as viaducts, trestles, or major bridges. Large bridges may have internal traction or braking trusses to carry longitudinal forces to the bearings. Viaducts frequently have braced tower bents at intervals to resist longitudinal force.

The American Association of Railroads (AAR) is currently conducting research on the longitudinal forces in bridges induced by the new high-adhesion locomotives now coming into service. In addition, the introduction of new mechanical systems such as the load-empty brake and electronically controlled brakes are affecting the longitudinal forces introduced into the track. Transit equipment can have high acceleration and deceleration rates, which can lead to high longitudinal forces on transit structures.

23.5.3.7 Wind Loading

Wind loading is the force on the structure due to wind action on the bridge and train. Chapters 7, 8, and 15 deal with wind on the structure slightly differently:

1. *Timber*: Use 30 psf as a moving horizontal load acting in any direction.
2. *Concrete*: Use 45 psf as a horizontal load perpendicular to the track centerline.
3. *Steel*: As a moving horizontal load:
   a. Use 30 psf on loaded bridge.
   b. Use 50 psf on unloaded bridge.

The application areas of the wind on structure vary as well:

1. *Timber*: For trestles, the affected area is 1.5 times the vertical projection of the floor system. For trusses, the affected area is the full vertical projection of the spans, plus any portion of the leeward trusses not shielded by the floor system. For trestles and tower substructures, the affected area is the vertical projections of the components (bracing, posts, and piles).
2. *Steel*: Similar to timber, except that for girder spans 1.5 times the vertical projection of the span is used.
3. *Concrete*: Wind load is applied to the vertical projection of the structure. Note that 45 psf = 1.5 (30 psf).

For all materials, the wind on the train is taken as 300 plf, applied 8 ft above the top of rail.

The 30-psf wind force on a loaded structure and 50-psf force on an unloaded structure used in Chapter 15 reflect assumptions on train operations. It was assumed that the maximum wind velocity under which train operations would be attempted would produce a force of 30 psf. Hurricane winds, under which train operations would not be attempted, would produce a wind force of 50 psf.

For stability of spans and towers against overturning due to wind on a loaded bridge, the live load is reduced to 1200 plf, without impact being applied. This value represents an unloaded, stopped train on the bridge.

It should be noted that Chapter 15 has a minimum wind load on loaded bridges of 200 plf on the loaded chord or flange and 150 plf on the unloaded chord or flange.

Virtually every bridge component can be affected by wind. However, wind is typically most significant in design of

1. Lateral bracing and cross frames
2. Lateral bending in flanges
3. Vertical bending in girders and trusses due to overturning
4. Tower piles or columns
5. Foundations

23.5.3.8 Stream Flow, Ice, and Buoyancy

These loads are experienced by a portion of the structure (usually a pier) because of its location in a body of water. These topics are only specifically addressed in Chapter 8, because they apply almost entirely to bridge substructures, which typically consist of concrete.

Buoyancy, stream flow, and ice pressure are to be applied to any portion of the structure that can be exposed to them. This typically includes piers and other elements of the substructure. Buoyancy can be readily calculated for immersed portions of the structure.

While the AREMA *Manual* does not address design forces for stream flow and ice pressure, other design criteria, such as the AASHTO *LRFD Bridge Design Specification* does include procedures for calculating them. The designer can use these sources for guidance until specific forces are included by AREMA.

Spans may be floated off piers due to buoyancy, stream flow, and ice pressure. Loaded ballast cars are sometimes parked on bridges during floods or ice buildup to resist this. Drift or debris accumulation adjacent to bridges can be a significant problem, reducing the flow area through the bridge and effectively increasing the area exposed to force from stream flow.

Two other factors concerning waterways must be considered. The first is vessel collision (or, more correctly) allision with piers. Pier protection is covered in Part 23, Spans over Navigable Streams, of Chapter 8. These requirements should be addressed when designing a bridge across a navigable waterway. The second factor to be considered is scour. Scour is a leading cause of bridge failure. The AASHTO *LRFD Bridge Design Specification* contains scour analysis and protection guidelines. Hydraulic studies to determine required bridge openings should be performed when designing new structures or when hydrologic conditions upstream of a bridge change.

23.5.3.9 Volume Changes

Volume changes in structures can be caused by thermal expansion or contraction or by properties of the structural materials, such as creep or shrinkage. Volume changes in themselves, if unrestrained, have relatively little effect on the forces on the structure. Restrained volume changes, however, can produce significant forces in the structure. The challenge to the designer is to provide a means to relieve volume changes or to provide for the forces developed by restrained changes.

Chapter 7 does not specifically state thermal expansion movement requirements. Due to the nature of the material and type of timber structures in use, it is unlikely that thermal stresses will be significant in timber design. Chapter 15 requires an allowance of 1 in. of length change due to temperature per every 100 ft of span length in steel structures. Chapter 8 provides the following table for design temperature rise and fall values for concrete bridges:

Climate	Temperature Rise	Temperature Fall
Moderate	30°F	40°F
Cold	35°F	45°F

It should be noted that the tabulated values refer to the temperature of the bridge concrete. A specific railroad may have different requirements for thermal movement.

Expansion bearings are the main design feature typically used to accommodate volume changes. Common bearing types include:

1. Sliding steel plates
2. Rocker bearings
3. Roller bearings (cylindrical and segmental)
4. Elastomeric bearing pads

Provision should be made for span length change due to live load. For spans longer than 300 ft, provision must be made for expansion and contraction of the bridge floor system within the trusses.

For concrete structures, provisions need to be made for concrete shrinkage and creep. Specific guidelines are given in Chapter 8, Parts 2 and 17 for these properties. It is important to remember that creep and shrinkage are highly variable phenomena, and allowance should be made for higher-than-expected values. It also should be noted the AREMA *Manual* requires 0.25 in^2/ft minimum of reinforcing steel in exposed concrete surfaces.

Chapter 8 also requires designing for longitudinal force due to friction or shear resistance at expansion bearings. This is in recognition of the fact that most expansion bearings have some internal resistance to movement. This resistance applies force to the structure as the bridge expands and contracts. The AREMA *Manual* contains procedures for calculating the shear force transmitted through bearing pads. Loads transmitted through fixed or expansion bearings should be included in substructure design.

Bearings must also be able to resist wind and other lateral forces applied to the structure. Chapter 19 of the AREMA *Manual for Railway Engineering* covers bridge bearings. It is included in the 1997 *Manual*, and should be applied for bearing design and detailing.

It should be noted that movement of bridge bearings affects the tolerances of the track supported by the bridge. This calls for careful selection of bearings for track with tight tolerances (such as high-speed lines). Maintenance requirements are also important when selecting bearings, since unintended fixity due to freezing of bearings can cause significant structural damage.

23.5.3.10 Seismic Loads

Seismic design for railroads is covered in Chapter 9 of the *Manual*. The philosophical background of Chapter 9 recognizes that railroad bridges have historically performed well in seismic events. This is due to the following factors:

1. The track structure serves as an effective restraint (and damping agent) against bridge movement.
2. Railroad bridges are typically simple in their design and construction.
3. Trains operate in a controlled environment, which makes types of damage permissible for railroad bridges that might not be acceptable for structures in general use by the public.

Item 3 above is related to the post-seismic event operation guidelines given in Chapter 9. These guidelines give limits on train operations following an earthquake. The limits vary according to earthquake magnitude and distance from the epicenter. For example, following an earthquake of magnitude 6.0 or above, all trains within a 100-mile radius of the epicenter must stop until the track and bridges in the area have been inspected and cleared for use. (Note that specific railroad policies may vary.)

Three levels of ground motion are defined in Chapter 9:

- Level 1 — Motion that has a reasonable probability of being exceeded during the life of the bridge.
- Level 2 — Motion that has a low probability of being exceeded during the life of the bridge.
- Level 3 — Motion for a rare, intense earthquake.

Three performance limit states are given for seismic design of railroad bridges. The serviceability limit state requires that the structure remain elastic during Level 1 ground motion. Only moderate damage and no permanent deformations are acceptable. The ultimate limit state requires that the structure suffer only readily detectable and repairable damage during Level 2 ground motion. The survivability limit state requires that the bridge not collapse during Level 3 ground motion. Extensive damage may be allowed. For some structures, the railroad may elect to allow for irreparable damage, and plan to replace the bridges following a Level 3 event.

An in-depth discussion of seismic analysis and design is beyond the scope of this section. Guidelines are given in Chapter 9 of the manual. Base acceleration coefficient maps for various return periods are included in the chapter. It should be noted that no seismic analysis is necessary for locations where a base acceleration of 0.1 g or less is expected with a 475-year return period. For most locations in North America, therefore, a seismic analysis would not be needed.

Section 1.4 of Chapter 9 addresses seismic design. Important structures (discussed in its Section 1.3.3) should be designed to resist higher seismic loads than nonimportant structures.

Even if no specific seismic analysis and design is required for a structure, it is good practice to detail structures for seismic resistance if they are in potentially active areas. Specific concerns are addressed in Chapter 9. Provision of adequate bearing areas and designing for ductility are examples of inexpensive seismic detailing.

23.5.4 Load Combinations

A variety of loads can be applied to a structure at the same time. For example, a bridge may experience dead load, live load, impact, centrifugal force, wind, and stream flow simultaneously. The AREMA *Manual* chapters on structure design recognize that it is unlikely that the maximum values of all loads will be applied concurrently to a structure. Load combination methods are given to develop maximum credible design forces on the structure.

Chapter 7, in Section 2.5.5.5, Combined Stresses, states: "For stresses produced by longitudinal force, wind or other lateral forces, or by a combination of these forces with dead and live loads and centrifugal force, the allowable working stresses may be increased 50%, provided the resulting sections are not less than those required for dead and live loads and centrifugal force."

Chapter 15, in Section 1.3.14.3, Allowable Stresses for Combinations of Loads or Wind Loads Only, states:

a. Members subject to stresses resulting from dead load, live load, impact load and centrifugal load shall be designed so that the maximum stresses do not exceed the basic allowable stresses of Section 1.4, of Basic Allowable Stresses, and the stress range does not exceed the allowable fatigue stress range of Article 1.3.13.
b. The basic allowable stresses of Section 1.4, Basic Allowable Stresses, shall be used in the proportioning of members subject to stresses resulting from wind loads only, as specified in Article 1.3.8.

c. Members, except floorbeam hangers, which are subject to stresses resulting from lateral loads, other than centrifugal load, and/or longitudinal loads, may be proportioned for stresses 25% greater than those permitted by paragraph a, but the section of the member shall not be less than that required to meet the provisions of paragraph a or paragraph b alone.
d. Increase in allowable stress permitted by paragraph c shall not be applied to allowable stress in high strength bolts.

Chapter 8, in Part 4 on Pile Foundations, defines primary and secondary loads. Primary loads include dead load, live load, centrifugal force, earth pressure, buoyancy, and negative skin friction. Secondary (or occasional) loads include wind and other lateral forces, ice and stream flow, longitudinal forces, and seismic forces. Section 4.2.2.b allows a 25% increase in allowable loads when designing for a combination of primary and secondary loads, as long as the design satisfies the primary load case at the allowable load.

These three load combination methods are based on service load design. Chapter 8, in Part 2, Reinforced Concrete Design, addresses both service load and load factor design

Chapter 8, Section 2.2.4 gives several limitations on the load combination tables. For example, load factor design is not applicable to foundation design or for checking structural stability. In addition, load factors should be increased or allowable stresses adjusted if the predictability of loads is different than anticipated in the chapter.

For stability of towers, use the 1200 plf vertical live load as described in the Wind Loading section.

As a general rule, the section determined by a load combination should never be smaller than the section required for dead load, live load, impact, and centrifugal force. It is important to use the appropriate load combination method for each material and component in the bridge design. Combination methods from different sections and chapters should not be mixed.

23.5.5 Serviceability Considerations

23.5.5.1 Fatigue

Fatigue resistance is a critical concern in the design of steel structures. It is also a factor, although of less significance, in the design of concrete bridges. A fatigue design procedure, based on allowable stresses, impact values, number of cycles per train passage, fracture criticality of the member, and type of details, is applied to steel bridges. Fatigue can be the controlling design case for many new steel bridges.

23.5.5.2 Deflection

Live load-deflection control is a significant serviceability criterion. Track standards limit the amount of deflection in track under train passage. The deflection of the bridge under the live load accumulates with the deflection of the track structure itself. This total deflection can exceed the allowable limits if the bridge is not sufficiently stiff. The stiffness of the structure can also affect its performance and longevity. Less stiff structures may be more prone to lateral displacement under load and out-of-plane distortions. Specific deflection criteria are given in Chapter 15 for steel bridges. Criteria for concrete structures are given in Chapter 8 using span-to-depth ratios.

Long-term deflections should also be checked for concrete structures under the sustained dead load to determine if any adverse effects may occur due to cracking or creep.

23.5.5.3 Others

Other serviceability criteria apply to concrete structures. Reinforced concrete must be checked for crack control. Allowable stress limits are given for various service conditions for prestressed concrete members.

23.6 Capacity Rating

23.6.1 General

Rating is the process of determining the safe capacity of existing structures. Specific guidelines for bridge rating are given in Chapters 7, 8, and 15 of the AREMA *Manual*. Ratings are typically performed on both as-built and as-inspected bridge conditions. The information for the as-built condition can be taken from the bridge as-built drawings. However, it is important to check the current condition of the structure. This is done by performing an inspection of the bridge and adjusting the as-built rating to include the effects of any deterioration, damage, or modifications to the structure since its construction. Material property testing of bridge components may be very useful in the capacity rating of an older structure.

Structure ratings are normally presented as the Cooper E value live load that the bridge can safely support. The controlling rating is the lowest E value for the structure (based on a specific force effect on a critical member or section). For example, a structure rating may be given as E74, based on bending moment at the termination of a flange cover plate.

As discussed in the Live Load section, there are a wide variety of axle spacings and loadings for railroad equipment. Each piece of equipment can be rated to determine the maximum force effects it produces for a given span length. The equipment rating is given in terms of the Cooper load that would produce the equivalent force effect on the same span length. Note that this equivalent force effect value will probably be different for shear and moment on each span length.

In addition to capacity ratings, fatigue ratings can be performed on structures to estimate their remaining fatigue life. These are typically only calculated for steel structures. Guidelines for this can be found in the commentary section of Chapter 15.

23.6.2 Normal Rating

The normal rating of the structure is the load level which can be carried by the bridge for an indefinite time period. This indefinite time period can be defined as its expected service life. The allowable stresses used for normal rating are the same as the allowable stresses used in design. The impact effect calculation, however, is modified from the design equation. Reduction of the impact value to reflect the actual speed of trains crossing the structure (rather than the 60 mph speed assumed in the design impact) is allowed. Formulas for the impact reduction are included in the rating sections of the AREMA *Manual* chapters.

23.6.3 Maximum Rating

The maximum rating of the structure is the maximum load level which can be carried by the bridge at infrequent intervals. This rating is used to check if extraheavy loads can cross the structure. Allowable stresses for maximum rating are increased over the design allowable values.

The impact reduction for speed can be applied as for a normal rating. In addition, "slow orders" or speed restrictions can be placed on the extraheavy load when crossing the bridge. This can allow further reduction of the impact value, thus increasing the maximum rating of the structure. (Note that this maximum rating value would apply only at the specified speed.)

References

AREMA, *Manual for Railway Engineering,* AREMA, Landover, MD, 1997.

American Association of Railroads, *Railroad Facts,* 1997 Edition, Washington, D.C., 1997.

Waddell, J.A.L., *Bridge Engineering,* 1916.