Railway Structures

Brian Lindamood, P.E.
Hanson – Wilson Inc
Ft. Worth, TX 76137
balindamood@wilsonco.com

Ronald G. Berry, P.E.
Burlington Northern Santa Fe Railway
Kansas City, KS 66106-1124
Ronald.berry@bnsf.com

Dallas R. Richards, P.E.
HSMM Inc.
Roanoke, VA 24016-4607
drichards@hsmm.com

James McLeod, P. Eng.
UMA Group, Inc.
Edmonton, AB. T5S 1G3
jmcleod@umagroup.com

Steven Sumner, P.E., S.E.
Metra
Chicago, IL 60661
ssumner@metrarr.com

Joseph E. Riley, P.E.
Metra
Chicago, IL 60661
ssumner@metrarr.com

William Riehl, III, P.E.
Niemeyer & Associates
Jacksonville, FL 32258-2182
wsriehl@attbi.com

Christian J. Brown, P.E.
HNTB
Kansas City, MO 64105-1310
cbrown@hntb.com

Daniel Thatcher
HNTB
Indianapolis IN 46204-5178
dthatcher@hntb.com

Charley Chambers, P.E.
Hanson – Wilson Inc
Bellevue, WA 98004-6905
edchambers@hanson-inc.com

Patrick O. McCarthy, P.E.
H. W. Lochner, INC
Chicago, IL 60606-2806
The following section is not intended to instruct any person through the process of designing a railway bridge. It is intended to merely guide the engineer in the peculiarities relating to the design of railway bridges and structures as they relate to the design guidelines set forth by AREMA and general railway practice. It does in fact assume a base level of knowledge pertaining to the design of structures and bridge systems. As most of the bridge design in North America is generally related to roadways, the majority of comparisons drawn relate to roadway bridges to provide a sense of scope.

Most of the recommended practice relating to railway structures is contained within Volume 2 of the AREMA Manual for Railway Engineering. Chapters within this volume are divided by the three primary materials in use for railway structures including timber, steel and concrete (masonry being included in the latter). Other Manual chapters relate additional information, including seismic loading and bridge bearings. However, there remains some structural related information in other chapters, including utility protection and metal pipe loading in Chapter 1, and structural design of overhead catenary systems in Chapter 33.

The engineer, prior to the design of railway structures, must understand several considerations relating to material within the AREMA Manual for Railway Engineering. Though North American railways closely follow the subject matter within the AREMA Manual for Railway Engineering, all have specific areas or points of design where they deviate. Such information should be gathered prior to design. Secondly, while there are some common design elements and considerations relating to the application of loading of railway structures across the three major design areas (timber, concrete and steel), specific application and the magnitude of these loads does vary from chapter to chapter.
This chapter contains a basic description of the types of railway structures and their design considerations. The purpose is to inform engineers of design considerations for railway structures that are different from their non-railway counterparts. Due to variations in design standards between the different railways, consult the controlling railway for their governing standard before starting design.

8.1 Introduction to Railway Structures

Railway structures encompass a wide array of construction intended to support the track itself or house railway operations. Common examples of track carrying structures are bridges, trestles, viaducts, culverts, scales, inspection pits, unloading pits and similar construction. Examples of common ancillary structures are drainage structures, retaining walls, tunnels, snow sheds, repair shops, loading docks, passenger stations and platforms, fueling facilities, towers, catenary frames and the like.

While the design of ancillary structures for the railway environment may introduce considerations not found in their non-railway counterparts, these considerations are usually well defined in the governing railway’s standards. Accordingly, this chapter will focus primarily on track carrying structures.

When designing railway structures, the various sources of their loads must be considered, as they would be with any other similar, non-railway structure. In addition to the dead load of the structure itself, there are the usual live loads from the carried traffic. To these are added the dynamic components of the traffic such as impact, centrifugal, lateral and longitudinal forces.

Then there are the environmental considerations such as wind, snow and ice, thermal, seismic and stream flow loads. Finally, because railway structures must perform under heavier loads, have longer service lives, and dissimilar maintenance constraints
compared to their highway counterparts, other factors, including fatigue and maintenance issues, tend to influence railway structure design more than roadway structures.

Once the designer has established the first pass at the load environment for the subject structure, the primary difference between a highway structure and a railway structure should become obvious. In the typical railway structure, the live load dominates all of the other design considerations. For the engineer accustomed to highway bridge design, where the dead load of the structure itself tends to drive the design considerations, this marks a substantial divergence from the norm. Specifically, the unacceptability of high deflections in railway structures, maintenance concerns and fatigue considerations render many aspects of bridge design common to the highway industry unacceptable in the railway environment. Chief among these are welded connections and continuous spans.

8.2 Major Bridge Components

In general terms, the major components of track carrying structure are very similar to their non-railway counterparts. In addition to the types of construction, the engineer must also choose from among the available material alternatives. Generally, these are limited to timber, concrete and steel, or a combination of the three. Exotic materials can also be considered, but they are beyond the scope of this book. Each material has its specific advantages. Timber is economical, but has strength and life limitations. Structural timber of the size and grade traditionally used for railway structures is getting more difficult to obtain at a price competitive with concrete or steel. Concrete is also economical, but its strength to weight ratio is poor. Steel has a good strength to weight ratio, but is expensive. The material chosen for the spans will generally determine the designation of the bridge. For instance, steel beam spans on timber piles will be considered a steel bridge.

The point where one form of construction with a certain type of material becomes advantageous over another is a matter of site conditions, span length, tonnage carried and railway preference. While initial cost of construction is a major point in the decision process, the engineer must keep in mind such additional factors as construction under traffic and the long-term maintainability of the final design.

8.2.1 Substructure

The substructure consists of abutments and piers and includes the foundations supporting them. The substructure transmits to the underlying soil the forces comprising the dead load of the superstructure and substructure, the live load effect of passing traffic, and the forces from wind, water, etc. The substructure will generally consist of pile foundations, spread footings, piers and abutments and/or any combination of the three.

Investigate Underlying Soil and Geologic Conditions

Before proceeding with the design of a railway bridge, a careful investigation of the underlying soils should be made. Chapter 8, Part 22 of the AREMA Manual for Railway Engineering provides extensive recommendations on subsurface geotechnical exploration. Analysis of information obtained by borings may reveal the necessity or advisability of driving test piles. If conditions demand the use of pile foundations, the relative merits of treated timber, concrete and steel piles should be carefully weighed.

The stability of the substructure is obviously essential to that of the structure as a whole. Its condition should be under observation at all times, and special inspections should be performed during and after freshets, ice gorges, cloudbursts and other unusual happenings, which could have the potential of seriously impacting the safety of the structure. Immediately following such an occurrence, bridge piers and abutments should be examined carefully for evidence of scour or other adverse condition.

Piling

Today, most railway bridge foundations begin with driven piles or caissons. Piles may support some other footing component such as piers or tower legs or they may continue to become part of the bent as in trestle construction. While new construction typically favors either precast concrete or steel H-piles, timber piles still have a use in the repair of existing structures and for temporary construction. Concrete piles are usually used for large, heavy structures and are very durable, but are difficult to splice. Steel H-piles are easier to drive and splice, and work well in end bearing when driven into rock to resist settlement. Concrete-filled pipe piles have greater bearing capacity, but are more difficult to drive and splice. Pipe piles possess large moment of inertia; therefore, they are suitable for resisting lateral forces. Alternatively, caissons are large diameter reinforced concrete shafts, usually steel lined and installed by drilling. They are capable of supporting very large loads with minimum settlement.

Piling may be placed in two general classifications:

1. A bearing/friction pile, which is a timber, concrete or a steel structural element, is driven, jetted or otherwise embedded on end into the ground to support a load.
2. A sheet pile, which forms a continuous interlocking line of timber, concrete or steel piles, is driven close together to form a wall designed to resist the lateral pressure of water, earth or other materials. Timber and concrete sheet piles are tongued-and-grooved, while steel sheet piles are usually interlocking.

Piles are further distinguished by terminology describing their purpose. For example, batter or spur piles are driven at an angle to resist a combination of vertical and lateral forces. Guide or anchor piles are used to provide lateral support for timbers and walers. Fender piles are used to protect masonry structures, such as piers. Piles are usually driven by diesel hammers, with or without the use of water jets, or by driving in pre-bored holes, or in some cases by the use of hydrostatic pressure. Certain types of concrete piles are cast in place.

The capacity of a pile as a structural member is based on allowable stresses established in the AREMA Manual for Railway Engineering, Specifications for Timber Structures, Chapter 7; in Part 2, Reinforced Concrete Design, Chapter 8; or in the Specifications for Steel Structures, Chapter 15.

**Timber Piles**

Timber piles, when large enough and properly braced, can safely sustain loads ranging from 15 tons to 20 tons each. Consideration must be given to the imposition of bending moments from imposed lateral forces, which may be necessary for a pile to resist. The relative straightness of the pile also must be considered, since a vertical force on a crooked pile produces eccentric loading with accompanying bending stresses.

Embedded in moist ground or submerged in water, timber piles are relatively immune to decay. Timber piles exposed to the air without treatment or other special protection will decay within a few years. Treated or untreated timber pile, however, is susceptible to certain marine organisms found in warm waters.

The wood selected for piling should be of a nature that it will not tend to disintegrate under the impact of the driving hammer, and which will offer the maximum resistance to decay. White oak, cypress and long-leaf yellow pine are particularly suited to this purpose. Spruce and hemlock are also adaptable, and tamarack is extensively used with satisfactory results in the western section of North America. The general requirements for First-Class timber piles (suitable for railway bridge applications) are noted in Chapter 7, Section 1.9.3 of the AREMA Manual for Railway Engineering.

Each pile consists of the trunk or bole of an individual tree, and the ordinary range of length is from 20 to 60 ft. In special cases, local conditions may make it necessary to penetrate to exceptional depth to obtain footing on a sound-bearing stratum. The above lengths may be exceeded, either by single piles or by splicing two or more ordinary length piles. Piles up to 120 ft in length have been produced. Often, wooden cleats are used for splices to secure longer lengths when needed. Piles falling into the
classification of Second-Class may be used for cofferdams, falsework, temporary work and light foundations. The AREMA Manual for Railway Engineering, Chapter 7, Section 1.9, Tables 7-1-1 and 7-1-2 provide required dimensional sizing for both timber piles used for frictional resistance and end bearing.

Timber piling above ground or water level is subject to decay, even when treated. One or two defective piles can be spliced into the bent without re-framing the entire bent. However, a number of criteria should be entertained before doing so, including:

- Condition of the existing piles below the ground line
- Height and length of the bridge
- Density, weight and speed of traffic
- Grade and alignment of the bridge
- Service life remaining in the structure as a unit

Splicing in a pile or post is a satisfactory means of continuing a bridge in service until maximum service has been secured from other piles in the bridge.

**Steel Piles**

Steel piles may be divided in two general classifications: rolled "H" and tubular sections; the latter usually being filled with concrete after driving.

Steel piling, whether on dry land or in submerged locations, may be driven to form bents and encased in concrete to form a pier, thus enhancing the strength and providing protection for the steel.

**H-Beam Sections**

H-beam piles (Figure 8-2) are rolled metal sections, possessing wide flanges, and are designed especially for pile loading. H-beam piles provide strength both in tension and compression with smaller cross-sectional area than other types of piles for a given load. They are well adapted to deep penetration because of their relatively small point area. Their volume displacement is also small. Other advantages of H-beam piles include their relative immunity to breakage under
the impact of the hammer and their ability to penetrate hard formations such as coarse
gravel, compacted sand, and uneven soft rock or shale.

This type of pile is very well adapted to serve as a bearing pile at locations where the
soil strata above suitable bearing material (such as rock or hard pan) is shallow and
affords little frictional resistance. Disadvantages include susceptibility to corrosion and
under stray current conditions, electrolytic action. Used in friction bearing alone, H-
beam piles will generally require a greater length of penetration than a displacement
type of pile of the same load-bearing capacity.

**Tubular Steel Sections**

Tubular steel pile, filled with plain or reinforced concrete, is frequently used for special
types of bridge pier construction. Other types of tubular steel pile use a cold-rolled
fluted section, which also may be tapered.

**Concrete Piles**

Concrete piles are relatively immune to the ordinary forces of deterioration and decay
in the atmosphere and to the attacks of marine borer in the water. They also have a
greater bearing capacity for the individual pile in comparison with timber piles. A
concrete pile can be designed to suit the actual conditions under which it is to be used.
The use of large dimension concrete piling sometimes will permit a reduction in the
dimension of the foundation to accommodate restricted space. Concrete piles are also
used satisfactorily for trestle bents and sheet piling.

Concrete piles are capable of supporting loads up to 40 or 50 tons each. Ordinary
diameters range from 10 inches to 24 inches with lengths from 20 ft to 60 ft. In some
cases, concrete piles in lengths up to 120 ft have been cast and driven successfully with
special equipment required for driving. Concrete piles are of two general types: precast
and those that are cast-in-place.

**Precast Concrete Piles**

Piles of this type are so named because they are cast, centrifugal cast or extruded prior
to use. They are driven in much the same manner as timber piles. Precast piles (Figure
8-3) represent a wide variety of detail in design and reinforcement. The transverse
cross-section may be square, hexagonal, octagonal, round, etc., and may vary from
complete uniformity from end to end, to a taper of as much as 1/4 in. per ft (down to
8” for a 10” diameter pile). The taper required is a function of the type of soil into
which the pile is to be driven. Piles, which are to bear on a hard stratum and to act
substantially as columns, should be of uniform cross-section; while those which are to
be embedded in soft material and to derive their support from the skin friction of the
surrounding material, should be tapered.
Concrete piles may be driven in the same manner as timber or steel piles. A driving head or anvil fitted with cushion blocks should be used to protect the pile head from the direct impact of the moving part of the hammer. While concrete piles of proper strength will withstand hard driving, the hammering should be continued only for the period necessary to secure the penetration required to support the designed load. Water jets, either with separate jet pipes or holes within the pile are sometimes used to assist in driving concrete, as well as other pile types in very hard strata.

When it is impossible to drive a concrete pile to its full depth, it may be cut off by using a concrete friction saw and by torching the reinforcement with an oxy-acetylene torch. It is desirable to leave enough reinforcement above the cut-off so that a bond may be secured with the concrete footing or with the concrete cap, which rests on the pile. When a concrete pile is too short, removing a portion of the concrete near the top and utilizing the exposed reinforcement to provide a bond with the extension may splice it.

**Cast-In-Place Concrete Piles**

The cast-in-place concrete pile comprises a column of concrete formed by pouring the concrete into a metal shell or tube previously driven to the required depth. The advantages of this type of pile over the precast pile are found in the avoidance of pile damage during handling and driving. The disadvantage of this type of pile is that they are not suitable for use immediately upon placing, but must cure before loading. There is also the potential for fracture due to the shifting of the soil during the driving of additional piles in the vicinity. Dense, high-strength plain or reinforced concrete and heavy shells must be used.

Reinforcement is generally used for cast-in-place piles subject to lateral forces. Where this is done, the reinforcement should be fabricated and accurately placed as a unit, in order that the pile actually conforms to the design.
Abutments and Piers

Other common foundations are piers and abutments. The earliest designs of bridge piers and abutments included outer walls of masonry, usually limestone or granite, with the inner core filled with old rubble. Current designs are usually of reinforced concrete (Figure 8-4). These piers and abutments may rest on driven piles or may be “gravity walls” supported only by spread footings.

The design and location of the abutments and piers are dependent on the general design of the structure as a whole. Local conditions such as the natural features at the point of crossing, the type of traffic (train consist) to which the structure will be subjected, and legal requirements and property rights will govern the design.

The rights of adjacent property owners, the requirements of public travel, water-borne traffic and the jurisdiction of public regulatory bodies must receive due consideration in advance of the completion of the design and certainly before construction begins. If the bridge crosses a navigable stream within the United States or a wetland is impacted, the U.S. Army Corps of Engineers, the United States Coast Guard (in some cases) and numerous state and local regulatory agencies have jurisdiction and the proper permits must be secured. (See Chapter 11, Environmental Regulations & Permitting of the Practical Guide to Railway Engineering) Simple economics may not always govern the design of the structure in the long run.

Abutments

The three primary types of abutments are the "wing," the "U" and the "T." Modifications of these types are the "breast," the "buried," the "arch," and the "hollow" or "box" abutments. All types possess one characteristic feature, the body or face portion, commonly called the breast, which supports the bridge seat.

The "wing" abutment is the type most widely used where the embankment is not a high fill. It consists of a simple breast wall, flanked by wings. The wings may be turned backwards at an angle of approximately 30 degrees or more with the face of the breast, when required by local conditions. The upper surface of the wings is sloped to conform to the natural slope of the surcharge that it is retaining. The counterfort and buttress types of abutments are modifications of the "wing" abutment.
The "U" abutment is characterized by two wings, which extend backwards from the ends of the breast and at right angles to its face. This type is sometimes modified into the so-called "pulpit" abutment, where the wing length is long enough only to keep the bridge seat clean of the surcharge material behind the abutment.

The "T" abutment is similar to the breast type with the addition of a stem, which extends backwards from the center of the rear face to the top of the embankment slope, and is used to stabilize the breast and to bridge the slope of the embankment.

The "breast" type of abutment is a modification of the "wing" abutment in which the wings are eliminated and square ends are provided. It is commonly used at locations where the embankment is relatively low and water flow is negligible.

The "buried" abutment has an opening through the wall, where the surcharge spills around the ends and through the wall opening. This construction is desirable when the approach fill is very high because the continuous fill through the wall results in a material reduction of pressure behind an otherwise solid wall.

The "arch" abutment may be considered a modification of the "U" abutment, where the parallel sidewalls consist of one or more arches. This type is adapted to locations where embankments are so high that "wing" and "U" abutments would be uneconomical. The number and size of the arches are dependent upon the height of the bridge and the type of superstructure.

The "hollow" or "box" abutment was a type frequently adopted in grade separation work, at points where city streets are carried beneath railway tracks. Such a unit consists of a concrete box provided with a solid rear wall, floor and top. The front is usually open and is composed of two or more columns, or an arch. This type of abutment bridges the sidewalk and supports the ends of the railway span.

**Design of Abutments**

Abutments must be stable against overturning in front of the footing or in the face of the wall, and must be safe against crushing, sliding on the foundation or on any horizontal section through the structure. Abutments may be of the gravity wall design, where the abutment is so proportioned such that no reinforcement steel other than temperature steel is required; or they may be of the semi-gravity style, where the unit is so proportioned that some steel reinforcement is required along the back and along the lower side of the toe. The resultant force on the base of a wall or abutment should be considered to fall within the middle third of the structure if it is founded on soil and within the middle half of the structure if founded on rock, masonry or piling. The vertical loads to be carried are the live loads (except for impact), dead loads from the weight of the span and weight of the abutment and part of the earth on the footing, depending on the design of the abutment. The lateral forces parallel to the axis of the bridge are the train-produced longitudinal forces and the surcharge pressure from the earth due to both its weight and live load. At right angles to the axis of the bridge are
the wind loads to be included from the superstructure. See AREMA Manual for Railway Engineering, Chapter 8, Section 5.3 for more details regarding applied loads.

**Piers**

Piers constitute the intermediate supports for multiple-span bridges. They should rest on stable, unyielding foundations with their bases well below frost line, and also below the elevation of any possible scouring action. Most of the older piers are of the mass type, either solid or cellular, and are built of stone masonry, concrete or reinforced concrete. They require for their construction, the use of cofferdams or caissons conforming to the relative size of each pier and, in depth, to the elevation of suitable bearing strata.

Cofferdams generally are rectangular in shape and are built to expose the earth strata below the ground surface or the excavation within the enclosed area. They are watertight to the extent required and need strength to resist pressures from the outside. The cofferdam should be designed such that the combined cost of construction, maintenance and pumping is held to a minimum. Those of relatively small size and depth are sheeted with single or double-row sheeting, while steel sheet piling are commonly used for larger and deeper cofferdams.

Today, use of the mass-type piers in new construction has given way to more suitable and less costly types of pier. These include:

- Metal-shell cylinder piers and reinforced-concrete cylinder piers
- Precast reinforced-concrete piles
- Steel pipe piles (straight or tapered) filled with concrete
- Rolled H-section steel piles

Caissons are used in the construction of bridge piers at locations where other types of piers are impracticable due to the depth of water or overburden above the elevation of suitable bearing strata.

**8.2.2 Superstructure**

The superstructure is the portion of a bridge supporting and conveying the live load to the substructure on which it rests. As a structural element, it is the portion of the bridge spanning the opening. The superstructure consists of arches, slabs, beams, girders, trusses or troughs, and such floor systems and bracing as may be required. Superstructures may be divided into two general classes: steel spans and concrete spans (which include stone masonry).
The nature of the obstacle being crossed will drive most superstructure design decisions with the ultimate goal to achieve the least overall lifecycle cost for the structure. For short (height) structures, trestle construction is favored due to the economies of pile bents. Conversely, taller structures over good footing are likely to be viaducts with longer spans supported by towers. Where there is insufficient clearance over navigable waterways, moveable spans may be necessary. The addition of longer or moveable spans to clear main channels does not significantly affect the design of the balance of the structure. However, as the structure becomes taller, the economies of pile bents are diminished due to the need to strengthen the relatively slender components.

The alternative to conventional trestle construction is trestle on towers, otherwise known as viaducts. Trestle on towers can offer a significant reduction in footprint for only a moderate increase in span requirements. It is customary for the spans to be of alternating lengths, with the short span over the tower equal to the leg spacing at the top of the tower. This ensures that each span remains a simple span with full bearing at the ends of the span. Of course, trestle construction represents the typical site conditions. More demanding site conditions may require exotic solutions. For example, very tall, very short (length) conditions may lend themselves to arch construction, whereas for transit operations, very long main span requirements may lend themselves to suspension type construction and some trestles on towers may be better constructed as a series of arches.

8.2.3 Bridge Deck

The bridge deck is that portion of a railway bridge that supplies a means of carrying the track rails. In comparison to the rest of the superstructure design, bridge deck decisions are relatively simple. The choices are open deck and ballast deck. On open deck bridges (Figure 8-5), the rails are anchored directly to timber bridge ties supported directly on the floor system of the superstructure. On ballasted bridge decks, the rails are anchored directly to timber track ties supported in the ballast section. The ballasted bridge decks require a floor to support the ballast section and such floors are designated by their types, such as timber floors, structural plate floors, buckle plate floors or concrete slab floors, all of which transfer loads directly to the superstructure.
Variations from the two general types of bridge deck construction consist of track rails anchored directly to steel or concrete-slab superstructures (direct fixation) and the several types of concrete-encased beam spans or concrete-filled steel-trough superstructures on which the ballast section is placed. The latter types of structures have many examples still in service today, but are not generally cost-effective for new construction.

Some might consider the notion of bridge railings to be an odd bridge design consideration. Railway bridges traditionally have not been designed for the conveyance of anything other than railway traffic, which does not in and of itself, require any sort of railing whatsoever. Recently, however, a greater focus upon railway worker safety has resulted in railings being widely incorporated.

Open Bridge Decks

Many different considerations enter into the choice of open or ballast decks, and the selection usually is governed by the requirements of each individual structure. Open decks are less costly and are free draining (Figure 8-6), but their use over streets and highways requires additional measures such as canopies, plates or wooden flooring to protect highway traffic from falling objects, water or other materials during the movement of trains.

Open-deck construction establishes a permanent elevation for the rails. Normal surfacing and lining operations, particularly in curves, eventually result in line swings leading into the fixed bridge. The grade frequently is raised to the extent that the bridge eventually becomes low. The bridge dumps are of a different modulus than the rigid deck. Thus, it becomes difficult to maintain surface off of the bridge as well. This equates to extensive maintenance costs that shortly will surpass the first cost savings gained by installing an open deck bridge over a ballast deck bridge. In welded rail, tight rail conditions can occur at the fixed ends of an open deck bridge, thus requiring an increased level of surveillance in hot weather.

Requirements for Ties

For ballast deck structures, bridge ties are no different than those found in traditional track construction. However, in track constructed with concrete ties, the track is often times transitioned to timber ties before crossing the structure. Some railway companies and agencies have had difficulty with fouled ballast, track alignment and deck surface
damage resulting from the use of concrete ties on bridges. Individual railway companies have established policies relating to the use of concrete ties on or around bridge structures that should be reviewed prior to design.

Bridge ties are commonly 10 – 12 feet long and range from 8” by 8” in swection to 8” by 14.” Longer ties are utilized for bridge walkways. The tie spacing is typically 4 inches between ties for open deck bridges and the usual track tie spacing for ballast deck bridges. It must be recognized that the tie functions as a beam and it must withstand bending and shear stresses, hold the rail to gage and transfer the rail load to the supporting members of the floor system. Open deck bridge ties typically utilize a softwood species of timber.

**Superelevation on Decks**

The superelevation of curved track on a bridge is obtained by:

- Sloping the pile or post cut-off of timber trestles.
- Tilting the superstructures of I-beam and shallow girder spans.
- Framing the floor system out of level transversely on through spans.
- Tapering the bridge ties.
- Increasing the depth of ballast under one rail on a ballasted-deck bridge.

Framing the floor system involves significant detailing and fabrication and is not often performed. The other methods are commonly employed.

High speeds in all classes of train service greatly intensify the problems connected with superelevation and alignment on curves. The eccentricity between the curve alignment and that of the bridge structure produces differences in stress in similar members of a floor system, dependent upon their location. Careful analysis must be done to insure that none of these members are overstressed.

**Bridge Tie Framing**

Bridge ties sometimes are dapped where they contact the supporting steel as an aid in maintaining good track alignment over the bridge. This necessitates adzing the tie bottom at each flange edge, which may result in undesirable horizontal shear cracks extending inward from the bottom of the dap. AREMA recommends that the dap not exceed the flange width by greater than ½ inch and that the depth of the dap be not more than ½ inch. Where dapping is practiced, the depth should be held to the very minimum required and careful check should be made to determine that the remaining depth of the tie is ample to carry the loads. Where cover plates do not extend the full length of the span, wood shims or steel plates may be added to bring the tie support
surface to the same plane, or specially-framed wood shims of proper thickness may be fastened with metal straps to the bottoms of bridge ties to bring all ties to the required surface. Procedures for dapping and/or shimming ties for superelevation are covered in Chapter 7 of the AREMA Manual for Railway Engineering, Section 1.14.7.

**Ballasted Decks**

A ballasted deck (Figure 8-7) provides a better riding track. The track modulus is consistent on the dumps of the bridge as well as across the bridge. Thus, one is unlikely to have surface runoff problems on the bridge dumps. Surfacing and lining operations can continue across the bridge unimpeded. However, care must be exercised to maintain a permanent grade line in the vicinity of and over a ballasted deck bridge to be certain that excessive quantities of ballast are not accumulated on the bridge structure through track raises during successive reballasting operations.

Ballasted decks (Figure 8-8), irrespective of the type of bridge floor, afford a considerable measure of protection to the steel floor system against damage from derailed car wheels traveling across the bridge. Over roadways, vehicles and the public are protected from dropping ballast and material off of the cars.

**Ballast**

The depth of ballast contributes to the satisfactory functioning of ballasted decks on railway bridges. It is generally agreed that 6 inches to 12 inches of ballast under the ties is adequate and that more than 12 inches is undesirable because of the potential of overload involved, except when provision is made in the design for a greater load. Many designers calculate the dead load on the basis of 18 inches to 24 inches of ballast to accommodate future raises.
Bridge Floors

A bridge floor for a ballasted-deck span is designed, such that as nearly as possible, the bridge track section replicates the track conditions on the adjoining bridge dumps. The ballast pan must have sufficient capacity to carry the heavy dead load of the floor and the ballast, and also to properly distribute the live and dead loads from various types of bridge floors to the supporting superstructure. The arrangement of the members in a floor system supporting a bridge floor is different from the arrangement of longitudinal stringers and transverse floorbeams, which make up the floor system of many open-deck spans.

A bridge floor for a ballasted deck may conform to one of several types including:

a) Concrete segmented girder spans incorporating the concrete ballast pan within the segmented unit.

b) Creosoted timber planks laid transversely to the track and supported on a suitable floor system of timber or steel.

c) Reinforced concrete slabs laid transversely to the track and supported on deck-plate girder spans or on a specially designed floor system of through-girder and through-truss spans.

d) Structural plates (Figure 8-9) supported by stringers (longitudinal I-beams) for short spans, and floor beams (I-beams placed transversely to the track and framed to longitudinal members) for longer spans.

e) Structural troughs placed longitudinally for short spans and transversely for the longer spans, with concrete filling in the down troughs and covering the entire floor area.

f) Structural plates, which bare on a series of transverse tees, the vertical legs of which vary in height and also are tapered. The result is a floor sloped for drainage in both directions, and the bases of which are supported by wide flange beam sets or structural plates, which bear on transverse I-beams supported on deck girders.

Trough Floors

The steel-trough bridge floor has been used in the past primarily for ballasted deck structures over city streets, particularly in connection with track elevation work.
Although more cost effective designs are available today for new construction, there are large numbers of this type of deck structure still in existence in major urban areas. Longitudinal troughs are used at locations where crossings intersect at approximately right angles and where columns are permitted at the center of the street. Such troughs are supported on cross girders framed to the columns, while the outside legs of the two outside troughs are extended upward to form the ballast stops.

After erection, the down-troughs are filled with concrete, which also covers the entire area to a depth of about 3 inches above the tops of the troughs at the end of the bridge, and about 4 inches above the troughs at the center of the bridge. Along the sides, the concrete filling is flared up against the ballast stops for varying distances above the top of the rail. The concrete filling is sloped for drainage in such a manner as to permit delivery of the water to drain pipes located below the bridge seat level. Suitable reinforcement should be provided immediately below the top of the concrete filling, particularly in the area above the cross girders; otherwise, deflection under live loads will cause transverse cracks in the concrete. The use of trough floors at locations, where the intersecting angle with the street is acute, or where roadways of unusual width are required, necessitates placing the troughs transversely to the track and framing them to through girders or to through trusses. The design details of these floors are essentially the same as for longitudinal troughs, the exception being the necessity for drain holes through the floors to avoid long, flat slopes for drainage, which in turn, requires the installation of a drainage system to dispose of the accumulated water.

Drainage

The primary requisites for bridge floors are economy, minimum weight and water tightness, together with strength and shallow depth. Comparisons of economy should include cost of materials, fabrication and erection. Bridge floors not only catch water but also retain it. As the track must be removed prior, replacement and maintenance of the bridge decks can be difficult and expensive. Every precaution should be taken to insure long life, which requires that all bridge floors be protected by waterproofing. Water falling on the track percolates through the ballast to the waterproofing where it remains, unless some suitable means for quick runoff has been provided. Quick runoff of precipitation is dependent upon clean ballast and a well-designed drainage system delivering water to outlets through the floor or to drain pipes located at the back of the abutments.

Open Deck vs. Ballast Deck

In addition to the obvious weight and construction costs, each of the span alternatives has its unique safety, environmental and maintenance concerns. In some instances, these intangible factors can carry more weight than the resulting cost implications. Only the governing railway can provide guidance as to the importance of these and related issues.
Concrete ballast decks are almost never cast in place in railway structures. They are typically precast or prestressed sections placed on the structure after steel erection. This means they cannot be considered part of a composite structure and offer no structural benefit, as would a similar concrete deck in the highway counterpart.

There are also operating disadvantages to the use of open deck bridges that may not be readily apparent. Bridge maintenance must often be performed under contractual agreements by bridge and building department forces. Thus, any operation involving an open bridge deck, e.g., renewal of plate cut ties, must be performed by bridge department personnel. As the adjacent track is also affected by anything affecting the elevation of the rails running across the bridge deck, track department forces must also be involved. Most railways have severely reduced their bridge gang rosters. Thus, it becomes a real logistics problem to have both groups present at the same time. On ballast deck structures, the ballasted trackage is considered track department work. Thus, surfacing operations and tie change-out can proceed unhindered.

**Anchorage of Bridge Ties**

Bridge ties on open-deck spans are held in position by bolts through the ties in line with the edge of supporting members (i.e., stringers). Usually two hook bolts are used on every third, fourth or fifth tie. The rail may or may not incorporate rail anchors. Anchoring rail on longer open deck structures can create alignment problems resulting from the thermal expansion of the rail. Most traditional mechanisms for fixing the bridge ties to the bridge cannot effectively transfer longitudinal forces. The servicing railroad guidelines pertaining to the anchorage of rail over both ballast and open deck structures should be consulted for guidance in this area.

**Guard Timbers**

Bridge ties are held to a uniform spacing by longitudinal timbers, called "guard timbers," placed outside of the track rails and fastened to the ties by bolts or lag screws. These guard timbers may be 4” x 8” or 6” x 8” in size. The 4” guard timbers are not dapped over the ties, while the 6” timbers generally are dapped (Check with individual railway standards).

**Inner Guard Rails**

In addition to the guard timbers, two lines of inner guard rails (Figure 8-10) are often used on each track on open and ballasted-deck bridges of such length as individual railways require. The two types commonly used are structural angles with a backing timber (found often on branch lines) and T-rails. On new installations, T-rails are generally used, even to the extent of replacing the angle guards when their renewals are necessary.
Scrap rails of a rail section smaller than the running rails are often used for inner guard rail. Each rail is placed on the inside of the running rail, often without the use of tie plates. Guard rails should be spiked to every tie and spliced at every joint. They should extend beyond the bridge ends in the direction of approaching traffic. The ends should terminate in a frog point or be joined and securely fastened so that a derailed truck will be straightened in direction and guided into the space between the running rail and the guard rail, thereby minimizing the damage that otherwise might result.

8.3 Bridge Types

8.3.1 Timber Trestles

The term, trestle, is commonly applied to a railway bridge with relatively short spans, constructed of timber, concrete or steel.

While the advent of economical steel construction has more or less eliminated timber from new mainline structures of any size, the lower initial cost and ease of construction still makes timber construction attractive for many light density lines. Additionally, because of the relative ease of repair, many significant older timber structures remain in service today.

In all of North America, timber trestles are the preponderant type of structure still found on branch lines, short lines and at temporary crossings. The timber used for timber trestles should be of a firm, close texture, which will afford strong structural members and offer maximum resistance to decay. A dense grade of Southern Pine, Douglas Fir or White Oak is suitable for this purpose. The timber selected should be sound, free from knots, pitch pockets and other imperfections that might impair its strength or durability. There is seldom justification for using untreated timber.

Terminology

The trestle supports are designated as "bents." (Figure 8-11) Each bent consists of posts (commonly four to six, but may be more or less depending on the design), transversely secured at the top with a member called a cap. The number of posts
utilized depends on the loading conditions, span length, soil and geology conditions and the height of bents (which affects the structure's lateral stability), as well as the individual railway's standards. When the lower ends of the supporting posts are driven into the ground, the structure is known as a "pile trestle." In other cases, the bottom of the bent as well as the top is secured by transverse framing and the bridge becomes a "frame bent trestle."

The center vertical posts used in each bent are known as "plumb posts," and take the vertical loads. The outside inclined posts, known as "batter posts," the tops being tilted toward the center of the bent and serving the purpose of giving increased stability, are installed adjacent to the plumb posts. The batter of these outside posts may vary between 1-1/2 and 3 inches per foot. Sway bracing provides additional lateral stability by the use of planks extending diagonally across the bent, through bolted to the ends of the cap and sill and also to the posts or piles. A similar brace, but placed with the opposite direction in slope, is installed on the opposite side of the bent such that the two braces cross in the middle. See Figure 8-11.

For trestles higher than 30 feet, a second bent is added to the top of the existing bent. Successive stories are added, not exceeding 20 to 30 feet in height, until the required elevation is reached. The bottom panel may be either pile or a frame bent; the upper stories are framed bents, each attached to the top of the lower panel. Each story has its own sway bracing. Shorter bents may utilize a transverse horizontal brace on each side of the bent in lieu of the diagonal bracing where sufficient height does not exist to install conventional sway bracing. Higher single story or multiple story bents often

Figure 8-11  Typical Timber Pile Bent
utilize horizontal bracing between bents called girts, which resist applied longitudinal loads.

For high timber trestles, the piles are often cut off at the ground line and the sill of the bottom story is framed on the pile tops. The transverse top members, or caps, are pinned to the bents by means of drift bolts, $\frac{3}{4}$ inches in diameter and 24 inches or longer in length. Attachment of the longitudinal girts and other bracing is done by through bolting the members.

**Caps**

Caps are typically 12-16 inches in section (width and thickness) and extend the width of the bent, commonly 13-16 feet for single tracks. Bent caps transfer the loads from the stringers to the pile or frame posts. False caps of varying thickness are used to shim up the height of the deck structure when required. Sills, the bottom transverse frame bent member atop the pile, are caps of the same dimensions, but may be longer in length.

**Stringers**

The stringers are structural members extending parallel to the rail and spanning the openings between the bents. (See Figure 8-12) Depending on individual railway standards, they will range in size from 7 to 10 inches wide by 14 to 18 inches deep and one or two spans in length depending on their location. The maximum span for the timber spans commonly in use today is 13 to 15 feet.

On open deck bridges, the stringers are chorded into a minimum of three and generally four or more beams with each adjacent stringer joint offset by one span length from its adjacent neighbor stringers for three span or longer structures. Each group of stringers is centered under the rail in order that load distribution is symmetrical.

On ballast decked bridges, spaced stringers with planking form the pan for a ballast deck. The spacing of stringers facilitates load distribution from the deck and inspection and stringer change-out. The longitudinal stringers should be spaced not less than 7 to 8 inches apart, as this will permit the insertion of suitable reinforcing timbers, if needed.
**Timber Connectors**

Timber connectors are used extensively in the construction of timber trestles. They consist of metal rings, plates or grids, which when embedded partly in the faces of overlapping members, transmits loads from one structural member to another. Certain types, such as the split ring and the flanged shear plate, fit into precut grooves or daps. Other types, such as the toothed ring and the spike grid, are embedded in the timbers by means of pressure.

The action of the connector in the joint is to increase the shear area, which actually carries the load. In timber joints, it is in the section of the timber nearest the contacting faces that the greatest shear stresses are developed. By embedding the connectors in this highly stressed shear area, the efficiency of the joint is strengthened significantly. For example, a 4-inch split ring with a 3/4-inch bolt will carry three times the load of a 3/4-inch bolt alone.

**8.3.2 Steel Bridges**

Typical steel construction covers the entire range from short simple beam spans on pile bents to large through trusses over major obstacles. In between, there is every possible combination of span and tower design. However, regardless of the specific span type, most steel structures are designed with simple spans. This facilitates ease of construction and maintenance under traffic. It also allows spans to be cascaded to different locations as needs arise. Simple spans are easier to analyze and for the most part, use simple, economical details.

**Girder Spans**

For short spans, rolled or welded sections are well suited for most applications. Spans up to seventy feet have been constructed using rolled steel beams. However, fifty feet is generally considered a practical maximum for rolled steel sections exclusive of special situations. Such structures are easy to fabricate and readily accept open and ballast decks. Additionally, they can be made more compact (top of rail to lowest member) by using multiple beams spaced with diaphragms.

For spans over fifty feet, rolled sections generally do not offer sufficient section modulus to control deflection. For these longer spans, a built up section (Figure 8-13) is more desirable as it produces a more efficient use of the material. Such built up sections are either welded or bolted plate girders and can achieve spans of 150 to 180 feet. These girder spans fall into two
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categories: Deck Plate Girders (DPG) and Through Plate Girders (TPG).

Deck plate girders (Figure 8-14) are typically the preferred design for locations where vertical clearance under the bridge is not critical, i.e., over streams or non-navigable rivers or on high fills. The top flange of the deck plate girder can be utilized to support the deck, thus no flooring system is required. (See Figure 8-15)

The elimination of the floor framing system and the need for girder bracing with knee braces required of through plate girders, makes the deck plate girder the more efficient and cost effective design. Deck plate girders are well suited for either open or ballast decks. However, the engineer must consider the presence of cover plates on the top flange for long spans and make the appropriate allowances in the deck structure. This may require specific dapping of the wood ties in open decks or different ballast pans in concrete ballast decks. The governing railway must be consulted for their standard details in this matter. Deck plate girders also require a greater total envelope beneath the track structure, thus limiting clearances below.

As indicated above, through plate girders are less efficient than deck plate girders of equal length. This is because the top cannot be directly supported and there is the added weight of the floor system (Figure 8-16). Knee braces are incorporated at each floor beam to girder connection to provide top of girder support. The floor system may consist of transverse
floor beams alone, as in the case of some ballast deck designs (Figure 8-17), or it may consist of floor beams and stringers in both ballast and open deck designs. The stringer and floor beam flooring system (Figure 8-18) drives the need for a deeper girder because of the greater depth required of the stringers to carry the imposed loads on the entire panel between floor beams rather than the distributed load spread out to each close-centered floor beam. Combined, these two factors make for a heavier span than a deck plate girder span of equal length. However, given the opportunity to decrease the depth of construction from the top of rail to lowest member, through plate girder spans are frequently employed in tight clearance situations such as over roadways.

The engineer must pay particular attention to side clearances since the track is effectively inside the structure. Special precautions must be taken when renewing bridge ties on through plate girder bridges utilizing an open deck in CWR – particularly in hot weather or in curves during cool weather. The girders typically do not provide sufficient clearance to remove/install the ties without jacking one rail up in each panel. Often the rail must be cut.

The pony through plate girder is a compromise between the through plate girder and the deck plate girder. In the pony girder, the floor beam connections to the longitudinal girders are made about half way up the girder. This minimizes the need for the knee brace system to support the girder, but it also reduces the vertical clearance under the structure as well, although not to the extent of the deck plate girder.

**Truss Spans**

Steel trusses (Figure 8-19) offer a practical solution for spans over 150 - 180 feet. Trusses are usually of open web design, consisting of top and bottom chord members connected by diagonal and vertical members called hangers. These members may be either of bolted or riveted construction. As with plate girders, trusses are either deck
trusses or through trusses. A bridge truss has two major structural advantages. The primary member forces are axial loads and the open web system permits the use of a greater overall depth than for an equivalent solid web girder. Both of these factors lead to economy in material and a reduced dead load. The increased depth also leads to reduced deflections, i.e., a more rigid structure. The advantages are achieved at the expense of increased fabrication, inspection and maintenance costs.

A truss is simply a framework for carrying a load. Like the top and bottom flanges of a girder span, the top chord members of a truss are in compression and the bottom chords are in tension. Formerly, trusses were pin connected, which freed the structure of imposed moments. Today connections are bolted, relieving the associated problem of pin wear at the expense of proportioning members for the moments created by a fixed connection. However, there are still significant numbers of pin-connected trusses in service. A brief description of the development stages of the truss leading up to today’s Warren truss is given in the following paragraphs. See Figure 8-20.

The Howe truss is the earliest type of simple truss, and was patented in the United States by William Howe in 1840. In this design, the diagonal web members are in compression; the vertical web members are in tension.

\[ \text{“Design of Bridge Superstructures,” Colin O’Conner, John Wiley & Sons, Inc., 1971.} \]
The Pratt truss is a modification of the Howe truss. In the Pratt truss, the vertical web members are in compression, and the diagonal members are in tension. The panel connections were pinned connected.

The Whipple truss in turn modified the Pratt truss. It uses a double system of web members, each diagonal extending over two panels. This permitted longer span lengths than achievable with the Pratt truss.

The Pennsylvania truss was another refinement of the Pratt truss. It uses sub-divided panels and curved top chords for through trusses and curved bottom chords for deck trusses. This type of truss is used for long spans, where simple Pratt or Warren trusses cannot obtain economical construction. The connections at the panel points were made by pins, but today are bolted.

In the original Warren truss, all of the web members were inclined, being alternately subject to compression and tension. This type was rarely used for pin-connected bridges. The loading and unloading of the panel (continual reversal of axial force in the web members) created pin wear. However, this truss, modified by the introduction of vertical members for the support of the panel load and with riveted or bolted connections at the panel points, is now the truss of choice for short spans. It is also widely used for longer spans by subdividing the panels.

In Figure 8-20, the dotted lines and in Figure 8-21, the light diagonal lines are called counters. With only the dead load of the structure, the adjacent diagonals act only as tension members. However, when a live load is introduced on the adjacent span, the formerly tensile load becomes compressive and the member may undergo critical buckling. Counters offset the applied reversal in loading. Designers today avoid the use of counters, thus limiting most truss designs to the Warren truss. Most through trusses include overhead bracing. Thus, interior vertical clearances must be considered in addition to side clearances.

Similar to plate girders, deck trusses (Figure 8-22) are typically more efficient than through trusses for all the same reasons specified earlier in the discussion regarding deck plate girders versus through plate girders.
Steel Trestles

Steel trestles (Figure 8-23) are similar in construction to a timber trestle, except that the various members are constructed of steel. They may be composed of bents supported by suitable foundations, e.g., H-pile with caps welded in place.

Viaducts

A viaduct (Figure 8-24) is any series of spans, whether arches or steel girders, that is supported on high steel towers.

Typically, railway viaducts are of steel construction and are distinguished by unusual height and significant length. The spans are often alternating long and short girders, usually deck plate girders. The short or tower spans are commonly 30 ft to 50 ft in length, while the long, or intermediate spans are 40 ft to 100 ft long. Keeping the short span over the tower top ensures that the spans will remain as simple spans. Sometimes a bent, instead of a tower, is used adjacent to the abutments. This bent supports the adjoining ends of two long spans, the second one terminating on the first tower.

The length of the spans is dependent upon the height and length of the structure, as well as on the loads to be carried. Consideration is given to the proper balance between the costs of the substructure and the superstructure. Generally, the longest spans are in the highest structures. At locations where the railway line is over a valley forming the bed of a sizable river (Figure 8-25), or where other physical characteristics make it necessary, a long deck truss span or a
series of arch truss spans are often utilized.

### 8.3.3 Concrete Bridges

Four general types of concrete/masonry bridges are commonly used for railway purposes: arches (Figure 8-26), rigid-frame bridges, slab bridges, and concrete girders. Although many large and costly stone arches are still in service, reinforced concrete is used exclusively for the erection of modern masonry bridges.

#### Arches

Stone masonry arches and boxes came into use early in the life of railways in North America. They were constructed in single and multiple spans, and a large number are still in service on important main lines after more than a century of continuous service. Structures of this character are built of stone masonry or of concrete.

#### Rigid-Frame Bridge

This type of structure is one in which vertical loads produce horizontal reactions, i.e., a structure in which the horizontal member is structurally integral with the upright supports. Bridges of this type are built either in single or multiple spans with the bearings for the upright supports either fixed or hinged, although hinged bearings are generally preferred. The construction material is typically concrete or steel, which may be formed for either curved or a flat soffit.

Such structures when built of concrete are slab bridges in which the horizontal member is solid; or ribbed bridges in which the horizontal member consists of ribs or girders supporting a slab floor. When built of steel, they consist of frames supporting a concrete slab floor. The frames are spaced to facilitate attachment of bracing between them. The outside frames should be encased in concrete integral with the slab floor.

Like arches, rigid-frame structures do not tolerate foundation settlement. Rigid-frame structures permit the use of quite long spans with relatively shallow construction depth. For this reason, they were frequently used in connection with grade-separation projects. They lend themselves very readily to pleasing designs and are sometimes found more economical than simple spans under certain conditions.
Slab Bridges

These structures are simple spans resting on abutments and piers. In some instances where construction depth is limited, the track rails are attached directly to the slabs by suitable bearing plates and fastenings (direct fixation). The length of span is limited by the construction depth available and the construction cost as compared to other types of construction. Slab bridges were very common at one time with a number still remaining in service today. There are much more economical ways of spanning small openings available to the designer today.

Slab bridges may be divided into three classes: Reinforced concrete, I-beam encased, and concrete and T-rail structures. The load bearing capacity of the span in reinforced concrete structures is a function of the compressive strength of the concrete and the tensile and shear strength of the steel reinforcement. The I-beam encased span and the concrete and T-rail span derive their load carrying capability from the I-beams and the T-rails. The concrete encasement acts merely as filler and a protective covering.

The I-beams used in the construction of slab bridges range in depth from 12 inches to 36 inches, according to the length of the opening with 24 inches being common.

The concrete and T-rail slab was used for very short spans only, i.e., 6 ft to 10 ft clear spans, such as for cattle runs and box culverts. The rails are placed near the bottom of the slab with wire mesh directly below the base of the rails and in the ballast stops.

The concrete used for slab construction should be dense and the upper surface should be crowned or sloped and waterproofed. On long or multiple spans, deck drainage should be provided with adequate outlets for the drains, so that the water will be carried off quickly in order to prevent seepage and consequent deterioration of the slab.

Concrete Trestles

Trestles of this type usually consist of concrete pile bents spaced from 14 to 20 feet apart. The height should not be greater than the span. The bents may also consist of narrow concrete piers or concrete columns footed on concrete pedestals. A ballast deck is almost invariably used on a concrete trestle.

Concrete trestles are more expensive in first cost than those of timber. However, concrete trestles may possess significant advantages over...
other types of trestle construction. The replacement of a timber trestle with one of concrete (Figure 8-27) may be accomplished with minimum delay to railway traffic. The concrete pile bents are driven and caps cast. After curing, the timber deck is removed and the concrete slab placed between trains. In some cases, precast caps may be utilized over the top of H-pile or even timber pile.

Caps are often prestressed units or cast-in-place with a high early strength concrete. The cap or transverse strut at the top of the piles or columns forming the bent must be designed as a reinforced concrete beam to transfer the load from the slab uniformly to the supporting piles or posts. The floor slab or span may be poured in place after the bents have been constructed, but the use of precast panels with the ballast pan integral is common.

Concrete Girders

These are sometimes adopted for the construction of bridges designed to span openings between approximately 25 ft and 60 ft in length. Through, half-through and deck types are used, although the latter is generally preferable.

Common beam sections are slabs, tees and voided single and double cell boxes (Figure 8-28). These shapes are well suited for spans up to approximately fifty feet. In most cases, box sections are the preferred section, since they provide a solid deck suitable for ballasted track with no additional construction.

Common AASHTO and DOT I-sections have also gained acceptance in new construction. In these bridges, the beam sections are used in conjunction with a cast in place, composite deck. This type construction can provide ballast deck spans up to 100 feet. However, given the time required to form and cure the cast in place concrete, this type of construction is only suited for new railway line, off-line or shoe-fly construction.

Precast, post-tensioned segmental concrete construction has also gained acceptance in new construction. This type of bridge allows construction of very long spans. However, given the time required to set and anchor each segment, this type of construction is also only suited for new, off-line or shoe fly construction.

Given the advances in precast concrete technology and acceptance, cast-in-place, reinforced concrete is seldom used in span construction. Rather, its primary use has
been relegated to below grade structures such as foundations, piers, abutments and headwalls. The time required to form and cure cast-in-place concrete renders it inappropriate for construction under traffic.

8.3.4 Moveable Spans

In locations where a fixed bridge cannot provide sufficient clearance over a specific obstacle such as a navigable waterway, a moveable span may be required. The common forms of moveable spans are Bascule, Lift and Swing. Variations of these structures are also found in shop environments where turntables and transfer tables are use to reposition cars and locomotives between various tracks.

Determination of whether a movable bridge to be utilized is dependent largely on the horizontal and vertical clearance requirements posed. Actual design requires additional considerations, since the structure is a precision machine that must maintain perfect alignment every time it is lowered to maintain track, signal and possibly electrical continuity.

Specific design elements that must be entertained beyond the structural characteristics of the bridge include:

- Lift machinery must be integrated within the structure.
- Bridge becomes a fixed span when closed and ready for traffic.
- Operating machinery should be of simple design, easily installed and maintained.
- Clearance between superstructure/pier elements critical in design and details.
- Specific loading conditions (structural and wind loading) are based on type of movement.

Selecting the type of movable bridge to be used is dependent on the width of the channel and the type of navigation using the channel. Appropriate foundations must be selected. Channel clearance will often dictate the location and/or configuration of piers (particularly center piers). Lastly, the duration and required frequency of bridge openings and closings must be considered. The potential impact to rail and other vehicles must be evaluated.

**Bascule Bridges**

Bascule bridges are single leaf spans of either plate girder or truss construction. They open vertically by pivoting at one end of the span to provide the navigable opening. They are suitable for small to medium span lengths and consist of one of three basic types:
Double leaf bascule bridges, although common in highway environments, are not used for railway design due to the lack of center support.

In the trunnion bascule bridge (Figure 8-29), the leaf rotates about a horizontal axis with the trunnion supporting the entire structure when raised.

In the rolling lift bascule bridge, the span rolls back and forth on curved tracks (See Figure 8-30) to open and close. The curved tracks, which are segmental girders with a tread plate, are attached to the tail end of the structure. Lugs or teeth (See Figure 8-31) are attached to the bottom of the curved track and they engage a matched track plate (girder) to prevent slippage. The entire weight of the bridge is supported by the curved track when the structure is opening. At the top pivot end of the structure is a counterweight, which offsets the weight of the span. A powered pinion gear (See Figure 8-32) engages a fixed horizontal rack gear attached to a frame on the adjoining span. As the pinion gear moves forward or backward on the rack, the curved track enables the horizontal motion of the rack to translate into vertical motion of the structure. Obviously, the opening angle secured is a function of the horizontal roll distance available for the curved track to move.
The third type of bascule bridge is the Heel-Trunnion Bascule Bridge (Figure 8-33). The bascule span rotates about the main trunnion. The counterweight is attached to a rotating framework. The operating strut is composed of a rack gear. The outer end of the operating strut is pinned to the top chord of the bascule truss and the counterweight link. The opposite end of the counterweight link is also pinned. The inner end of the operating strut is not fixed. The counterweight frame rotates about the counterweight trunnion. A fixed pinion gear moves the operating strut towards the pivoting end of the bridge. Because of the pinned connections, the counterweight frame rotates downward, thus raising the bascule span. The counterweight offsets the weight of the bascule span.

**Swing Span Bridges**

Swing spans are moveable spans (Figure 8-34) balanced on a center pivot pier and are rotated 90° horizontally to provide a navigable opening. Typically, these spans are truss constructed to accommodate negative bending over the center pier while in the open position.

Swing Spans posses unlimited vertical clearance, but the center pier reduces
channel clearance. This type of structure is suitable for short to medium span lengths on each side of the pivot pier, with the spans usually symmetric in length.

There are two types of swing span structures in common use. The center bearing swing span supports the weight of the structure by a center thrust bearing (Figure 8-35). The center of gravity of the structure is immediately over the bearing to ensure that the bridge is balanced when in the open position. Balance wheels stabilize the structure as it opens and closes.

The other type of swing span is termed as a rim bearing swing span (Figure 8-36). The span rides on tapered rollers, which carry the weight while opening and closing as well as providing stability during movement.

**Vertical Lift Bridges**

Vertical Lift Bridges are comprised of a rigid horizontal span supported between two towers. Cables raise vertical lift spans vertically. The span remains in a horizontal position when raised or lowered. Unlike the previous examples, the navigable opening provided by the vertical lift span remains limited by the height of the lift towers. These spans may be a rolled beam, plate girder or of truss construction.

The weight of the span is offset by two counterweights located at the top of each tower. The lift machinery is mounted above the deck. The towers can be of either braced or unbraced construction. Access to the tower is required to grease sheaves and service lifting machinery. A control house is provided for the bridge operator’s use, also typically, above the deck level where good sight visibility in all directions is present. A system of span guides and bridge locks ensures that the bridge is properly aligned when the span is lowered to operate trains. Any misalignment will not permit the bridge locks to engage and a proceed signal will not be displayed. Vertical lift bridges are suitable for medium to long spans where height clearance is required. Vertical lift bridges are categorized by the location of the drive machinery.
In the tower drive vertical lift (Figure 8-37), the drive machinery is located on top of each tower. Counterweight ropes attach to the span and to the counterweight. The machinery turns the sheave, thus raising the span and lowering the counterweight.

In the span drive vertical lift (Figure 8-38), the drive machinery is located on the movable span. Again, the counterweight ropes attach to the span and to the counterweight, but the lifting force is provided by operating ropes (cables) and drums, one at each corner. A drive shaft from the motor, located in the control house extends out to the drum located at each end of the span.

Although the weight of the span lifted is massive, the load imposed on the operating cables is relatively small, due to the offsetting weight of the counterweight.

In each case, a movable bridge requires balancing the structure. Although gravity plays only a small part in opening and closing bascule and vertical lift bridges, it plays no part at all in swing bridges. The critical balancing component is the counterweight. Typically, the counterweights are of concrete or steel-encased concrete construction. The AREMA Manual for Railway Engineering, Chapter 15, (15.6.5.4) requires ballast pockets for the addition or removal of offsetting blocks to maintain future balancing of the structure. Balance pockets are provided for not less than 3.5% underrun in weight of the span and not less than 5.0% overrun of the span weight. Balance blocks must represent at least 1% of the span weight and 0.5% of the weight of the counterweight for future adjustment. The configuration of the counterweight is important too, especially for bascule bridges. The lowered counterweight must maintain clearance between other structural members of the bridge.

For vertical lift bridges, the designer performing calculations dealing with balance must consider on the counterweight side of the tower:

- Ropes and sockets
- Counterweight frame
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- Concrete
- Balance locks

On the span side of the tower, consider:

- Load to lift
- Ropes and sockets

For bascule bridges, the designer must determine the center of gravity of the counterweight as well as the center of gravity of the span about the pivot point.

Other design considerations include electrical control equipment, including, but not limited to:

- Power Systems
  - Control Systems
  - Relay-Based Systems
  - Programmable Logic Controller (PLC) Based Systems
  - Automated Controls
  - Manual Controls
  - Maintenance/Reliability

- Drive Systems
  - AC and DC Controllers
  - Braking Systems

Maintenance plays an important issue in the design and operation of movable bridges.

Maintenance issues include:

- Rail Joints (Figure 8-39)
- Wire Ropes
- Lubrication (Figure 8-40)
- Seating and centering
Gears

Turntables are typically truss or plate girder type structures that can rotate 360° within a pit to turn locomotives. These spans are usually supported on a center pivot pier as well as a circular track at the end of the span. Conversely, a transfer table carries locomotives or cars in a lateral direction. These spans are usually supported on a track at each end of the span.

8.4 Other Structures

8.4.1 Drainage Structures

Culvert structures for railways do not differ in type or function from their highway counterparts. However, they tend to be significantly sturdier due to the higher live loads, which must be supported. Each railroad has different preferences relating to the types of materials installed. Many prefer metal pipes to concrete, as they tend to be less susceptible to failure due to settlement. Newer materials such as plastic have not generally gained wide acceptance for use under track. Box culverts (Figure 8-41) are almost exclusively concrete. The preference of cast-in-place versus pre-cast differs between railways. Headwalls are not commonly used.
unless space constraints require them.

Box culverts may be one cell, two cell or three cell, depending on the size of drainage stream.

French drains are constructed adjacent to and parallel to foundation structures to drain away ground water. They are typically corrugated metal pipe with perforations along the bottom invert to allow drainage of the surrounding soil.

Other types of French drains consist of granular backfill materials surrounded by porous fabric ("filter fabric").

Chapter 15, Part 16, of the AREMA Manual for Railway Engineering, provides detailed information for the design of concrete box culverts.

8.4.2 Retaining Walls

Gravity Retaining Walls

A retaining wall is designed to resist the lateral pressure exerted by material in its rear. This material may be an embankment for supporting track loads or natural earth along the edge of a cut and separated from the wall by a wedge of filled-in material. Normally, retaining walls usually do not carry vertical loads. However, bridge abutment walls are types of retaining walls that are required to carry bridge superstructure vertical loads in addition to large net overturning moments.

Ordinarily, gravity retaining walls are built of reinforced concrete, mass concrete and formerly of stone masonry. Overturning forces are resisted by the "gravity" weight alone of the masonry or concrete.

Failure of a retaining wall can occur by sliding along a horizontal plane, by overturning or rotating and by crushing of the masonry. The design of the wall, and especially the footing, should include such special features as indicated by the character of the supporting earth at each location.

Crib Walls

Crib walls, also known as bin walls (Figure 8-42), are composed of interlocking prefabricated members arranged to form a series of cells or "bins," that are then filled with compacted backfill. Crib walls are frequently used as an alternative to stone or concrete retaining walls. Crib
walls are made of timber, precast concrete or steel, and are designed following "gravity wall" theory. Note: Although a carefully constructed foundation forms the base of a solid retaining wall, crib walls are ordinarily supported directly on the particular material encountered at each location. Consequently, the use of crib walls should be confined to locations where the supporting material is reasonably firm and stable and is free of impounded water. Tensile forces within each cell resist overturning forces. The cells are anchored by "deadmen" in the back of the fill. Many old railway crib walls were often constructed using old railroad ties.

Since the width of a crib wall increases as the height of the wall increases, space limitations may impose restrictions upon crib wall use.

Three different types of ready-made cribbing are available: Creosoted timber, steel and reinforced concrete. Cribbing of these types are used for:

- Widening roadway where the right of way is limited.
- Wings on "breast" abutments for bridges.
- Extensions of existing wingwalls, abutments, etc.
- Revetment and retaining walls for bank protection along streams.
- Stabilization of embankments.
- Loading platforms.
- Foundations for signals and switch stands.
- Protecting waterfront property against wave action.
- Track elevation work.
- Retaining walls where adjoining tracks are established at different grades.

Creosoted timber cribbing is made up of two different types of units: a header, which is placed at right angles to the face of the wall, extending into the embankment and interlocking with the second or stretcher unit, which is laid parallel to the face of the wall. Each header and stretcher is dapped and bored prior to treatment. Drift bolts are driven during erection to give additional stability to the interlocking timbers.

Metal cribbing consists of box-like headers and stretchers. Each stretcher usually has lugs at both ends, which fit into corresponding slots in the header units. Every header is locked at opposite ends to the stretchers directly underneath by bolts.
Precast reinforced concrete units (sometimes called ecology blocks) (Figure 8-43) are available for the building of retaining walls comprising rectangular cells, which when filled, becomes a gravity-type retaining wall. The stretchers usually are of a plain square section while the headers are of rectangular section with T-shaped heads for tying the stretchers together. Both open and closed-face designs are used.

Sheet Piling

Sheet piling consists of a series of slabs of wood, metal or concrete, driven in close contact and forming a sheet or partition. As excavation proceeds on one side of the wall, horizontal sections known as walers are welded or fixed to the piling to provide additional support. Sheet pile walls are fairly expensive and require extensive information on buried utilities prior to driving. Wooden sheet piling includes a variety of proprietary designs intended to provide a tongue-and-groove effect at the mating surfaces. A variety known as Wakefield is formed of three planks fastened together to create a tongue and groove. Steel sheet pile retaining walls consist of individual sheet piles driven into the ground that are interlocked to each other to form a vertical steel wall.

Submerged timber piling has long life at points where it is left as a part of the permanent construction and may sometimes be salvaged, if desired. However, the relatively thin planks are readily split or broomed by contact with stones or other hard materials encountered in passage through the soil.

Metallic piles have great penetrating power under the impact of the hammer and are less susceptible to damage in driving than timber piles. These piles may be left as a permanent part of the under water construction, or they can be withdrawn and re-used.

Concrete sheet piling, when properly and thoroughly cured before use, and under normal conditions, is permanent in water and air, and is particularly applicable where the sheet piling is to remain as a part of the permanent structure. Due to the relatively brittle nature of the material, the salvaging of such piling is difficult. For temporary use, timber or metallic sheet piling is preferable.

Sheet piling may be driven to form either single or double partitions. For double partitions, the space between is filled with earth to keep out the water.
and provide stability for the unit. Steel sheet piling is frequently left in place as a part of the completed structure.

Soldier pile and lagging retaining walls are cheaper than sheet pile walls and are more appropriate in areas where buried utilities are expected. The soldier piles are usually steel rolled sections driven vertically into the ground at 5-foot to 10-foot center-to-center distances. As excavation proceeds, concrete or timber lagging is placed horizontally between the soldier piles. Horizontal steel walers are added as bracing is needed.

**Mechanically Stabilized Earth**

Mechanically Stabilized Earth (MSE) (Figure 8-44) is a simple retaining wall system that was developed approximately 40 years ago in France. MSE retaining walls represent a relatively new method of resolving earth retention problems. Instead of regarding soil as a mass to be contained by force, the earth itself is reinforced to become an integral part of the structure. MSE walls rely on increasing the strength and stability of earth embankments by placing corrosion resistant reinforcing straps, welded wire mesh, or geotechnical fabric within the earth embankment as it is constructed. The walls then behave as gravity structures in an integral unit and provide structural flexibility. Native soils at the site or from excavation are usually acceptable for backfill. The resulting structure is strong, yet resilient.

MSE walls generally include a fascia panel (typically precast concrete, but can also be welded wire mesh, cast-in-place concrete, or other materials). Precast panels or cast-in-place fascia allow for a wide variety of architectural treatments and finishes. An MSE constructed with a face of welded wire (Figure 8-45) can be covered with air-blown mortar, seeded with grass or plants, or filled with rock. Sometimes MSE walls are constructed without a “face” using wrapped around fabric. The outer edge to the “wrap” is not compacted to allow for growth of grass or plants, making the wall into a “green” wall (Figure 8-46).
The MSE wall is easily adaptable to curves, angles or steps, and the face may be cut to allow for the installation of culverts or to accommodate site-specific requirements. MSE walls perform extremely well in a multitude of conditions. It performs particularly well in seismic zones, due to the built-in flexibility of the system, which allows for some movement without distressing the structure or causing cracks. It can also tolerate a certain amount of settlement, making it a desirable solution even in relatively poor subsoil conditions.

The primary reason for the use of MSE walls is its inherent low cost. Installation is fast and efficient, using a simple, repetitive construction procedure. After placing the initial course of panels, the first lift of backfill is spread and compacted. The reinforcement (steel straps, welded wire or geotechnical fabric) is laid on the compacted lift and connected to the panels (if used). Next, a lift of backfill is spread and compacted over the reinforcing material. This procedure repeats until the design height is reached. Regardless of height or length, the structure is stable during construction. Equipment may operate on any layer of backfill.

MSE walls are well suited for restricted sites or close property lines since construction is performed behind the wall face without any forms or scaffolding. MSE structures should be considered for projects that have problems that may include costly right-of-way acquisition, lack of suitable borrow sources, topographic restraints or difficult subsurface conditions.

Although the use of MSE technology has been proven and accepted in standard practice in highway applications, the use of MSE walls in the railway industry is limited and should be approved by the impacted railway before design starts.

**Drainage of Retaining Walls**

Water under the foundation and behind the wall is the most frequent cause of failure of a retaining wall. Walls in cuts generally are more vulnerable than those
along fills. Effective weep-holes through the footings and the body of the wall ordinarily will prevent the impounding of water behind the wall. However, additional measures may be necessary, such as the installation of drainage pipes to collect and deliver the water to the weep-holes or other suitably located outlets, and also sub-drainage adjacent to the footings, to lower the water level in the cut.

### 8.4.3 Tunnels

Although expansive and difficult to construct, tunnels (Figure 8-47) offer an effective solution to extend rail lines through mountains or other obstructions, providing a more direct route while maintaining operational track gradients. Tunnels have also been constructed to carry rail lines underground, beneath cities, rivers and canals. The engineering associated with tunnel design and construction is not specific to railway engineering. However, there are aspects of tunnel design that railway engineers need to pay particular attention to including:

- **Overhead Clearance** – To accommodate double stack and other over height equipment, superelevation through curves, future grade raises, etc.

- **Side Clearance** – Particularly through curved tunnel sections to accommodate maximum railcar swingout (See next section for detailed calculation).

- **Ventilation and Pressure Equalization** – A train travelling into a tunnel compresses the air in the tunnel, increasing the resistance due to an increase in internal air pressure. Pressure relief through proper ventilation is required to release this pressure build up.

- **Drainage** – To prevent ballast saturation, icing, differential pressure head against the tunnel wall.

- **Ballast** – Free draining material to allow for adequate drainage to the side ditches. In tunnels, consideration should also be given to alternate track support structures, as the ballast may tend to break down faster due to the lack of flexibility in the sub-grade. Consider direct rail fixation or alternate methods.
Tunnel Construction Methods

Prior to any tunnel design, a geological and geotechnical investigation must be performed. Even with today’s technology, it is still difficult to identify all areas of loose gravel and saturated sand. Once the ground has been investigated, an excavation and support procedure must be chosen that can handle any unpredictable ground conditions without unnecessary interruptions or risk.

There are two major classifications of tunnels: Rock cut tunnels and soft ground tunnels. Both types of tunnels have had significant improvements in their construction over the last century, including faster construction rate, increased usable cross-sectional area per excavated volume, reduced support volume and improved construction safety.

Some major lessons that have been learned from tunneling in poor ground are:

- Effective use of shields ahead of excavation.
- Excavate in small headings and place supports immediately to prevent loose material from sloughing into the excavation.
- Provide good foundations for temporary supports or Tunnel Boring Machine (TBM) footings.
- Specialized construction methods are needed if the earth and water pressures are not symmetrical.
- Use a close arch as support if the lateral pressure is high.
- Provide adequate space for simultaneous excavation and support activities.
- Successful application of “Segmented Linings” is possible because of improved mechanization.

Development of the Tunnel Shield and Tunnel Boring Machines (TBM) began in 1825 when the first tunnel shield was used in the 18-year construction of the Thames Tunnel. The shield consisted of 12 sections that could be advanced independently by pushing against the brick liner. The first circular shield, propelled by screw jacks, came out fifty years later in 1870.

In 1897, the Price Rotary Digger Shield was developed. The Price Rotary Digger Shield was the first soil excavation machine. The first successful mechanized Tunnel Boring Machine, for rock drilling, took another sixty years to be developed. It was used in 1954 for the Oahe Dam project in South Dakota.

In 1910, the first immersed railway tunnel was built under the Detroit River. Since then there has been an increasing use of immersed tunnels for crossing water bodies.
The New Austrian Tunnelling Method (NATM) was developed between 1957 and 1965. NATM is a tunneling philosophy based on scientifically established principles and proven ideas and is not a construction (excavation and supporting) method.

The following excerpts are taken from Professor E.T. Brown’s article in the November 1981 issue of “Tunnels and Tunnelling”:

The inherent strength of the soil or rock surrounding the tunnel should be conserved and mobilized to the maximum extent possible.

Controlled deformation of the ground is required to develop its full strength safely. However, excessive deformation, which will result in loss of strength or in unacceptably high surface settlements, should be avoided.

These conditions may be achieved in a variety of ways, but generally a primary support system consisting of systematic rockbolting or anchoring and a thin semi-flexible shotcrete lining are used. Whatever support system is used, it is essential that it is placed and remains in intimate contact with the ground and deforms with it.

- The timing of the placement of the support and of closing the initial shotcrete ring is of vital importance in controlling deformations and will vary from case to case.

- The primary support will partly or completely represent the total support required. The dimensioning of the secondary support is based on an assessment of the results of systematic measurements of stresses in the primary support elements and deformations of the tunnel surface and the ground surrounding the tunnel.

- The length of tunnel left unsupported at any time during construction should be as short as possible. Where possible, the tunnel should be driven full face in minimum time with minimum disturbance of the ground by blasting.

- All parties involved in the design and execution of the project (design and supervisory engineers and the contractor’s engineers and foremen) must understand and accept the NATM approach and adopt a co-operative attitude to decision-making and the resolution of problems.

While in general, the above is a good guideline to follow in the construction of tunnels, special consideration should be given to the following difficult rock conditions:

- Rock of little resistance

- Rock of high plasticity and of intensive jointing

- Rocks subject to high stresses, particularly lateral stresses
CHAPTER 8 - STRUCTURES

- Rock in tunnels with very low or very high cover
- Rock mass destroyed by mining activities or dynamic geological processes.

For further information, please refer to the following articles from “Tunnels and Tunnelling”:

- Leopold Muller (October 1978)
- Johann Golser (March 1979)
- E.T. Brown (November 1981)

8.4.4 Sheds

Slide or rock sheds (Figure 8-48) are typically used in mountainous terrain areas where the track has been constructed along the side slope of a mountain or rock outcropping. The function of this type of shed is to deflect falling rock or debris from above the track, which might otherwise come in contact with the track or operating equipment.

The sheds are generally constructed from large timbers or cast in place concrete and incorporate a sloped roof over the track with sufficient clearance to allow trains and equipment to pass through the shed unimpeded. The roof of the shed is sloped, falling from a higher elevation on the uphill side to a lower elevation downhill, providing a barrier to deflect falling rock or debris over and away from the track, allowing it to accumulate or continue down the slope of the mountain.

Sheds of this type are often constructed at the portal or entrance to mountain tunnels.

Snow sheds follow a similar principle to deflect debris away from the track, in this case, specifically to deflect or prevent the accumulation of drifting snow that might otherwise make the track impassable.
8.5 Structural Design Considerations

8.5.1 Introduction

With the exception of larger bridges, most highway structures designed for a 50 to 75-year service life often begin to reach their practical service life at about 30 years of age. Though this is commonly a result of increases in traffic or higher safety standards, the ability to perform major repairs or upgrades of highway structures by temporary removal of the bridge from service is generally not a significant concern. Railway bridges, on the contrary, are designed to have a significantly longer life, and indeed, a considerable number of railway structures in service today are in the neighborhood of 100 years old.

Detour routes resulting from failure or significant repair/maintenance efforts are expensive and may not be viable. Though the design criteria within AREMA reflect this consideration, the operating impact and expense must be called to mind when considering the replacement of an existing structure. Often times a designer will have a proposed design solution rebuffed by a railway for this reason. Though the solution offered may be widely accepted in highway design, the permanence required by the railway environment may not have been yet proven to the railway.

Railway structures require a much greater consideration of longitudinal loading than a typical highway bridge. This is the result of two environmental variables. Vehicle and individual wheel loads of railway vehicles are many times greater than roadway vehicles. Likewise, unlike roadways, the vehicle running surface (the rail) is continuous between the bridge structure and the adjacent roadbed. The track structure by its very nature is moderately flexible, distributing loads in all directions over a length of track. The introduction of a fixed object (e.g. end of bridge) concentrates this loading to specific points of distribution.

When comparing railway bridges to roadway, pedestrian, and other sorts of bridges, the live loading relative to the dead load is much greater and more consistent. This consistent loading and unloading over a greater stress range results in fatigue considerations more prevalent in railway bridge design than other types.
8.5.2 Bridge Loading

In the design of any structure, the designer must consider several different load types, including, but not limited to, dead load, live load, wind, weather (snow, ice, etc.), earthquake or any combination there of. Like other governing codes and design organizations including ACI, AISC and AASHTO, AREMA sets forth guidelines for both allowable stress for steel (Chapter 15) and timber (Chapter 7) and load factor design guidelines for concrete (Chapter 8) to be used in the design of structures subject to railway loading. Many of these guidelines are consistent in character, if not identical to other codes. However, there are many distinctions, which are the result of the different service demands of railway structures as well as railway practice or preference developed over the past 150 years.

The designer must be cognizant of the fact that each chapter is effectively independent of the others, and not all handle similar design considerations in the same fashion. Where a single structure may incorporate several different types of materials (e.g., a composite structure with steel stringers and a concrete deck), both Chapters 8 and 15 must be referenced throughout the design process. Some other chapters of the AREMA Manual for Railway Engineering may reference one of the structural chapters when addressing structural issues.

The reader is also cautioned that the Manual for Railway Engineering is always under revision. The following material is current as of the date this text was published and is provided herein only for general informational understanding. Referencing the latest issue of the Manual for Railway Engineering is essential before undertaking any design activity.

Dead Load

The dead load consists of the estimated weight of the structural members, plus that of the tracks, ballast and any other railway appendages (signal, electrical, etc.) supported by the structure. The weight of track material (running rails, guard rails, tie plates, spikes and rail clips) is taken as 200 pounds per lineal track foot. Ballast is assumed to be 120 lbs per cubic foot. Treated timber is assumed to be 60 lbs per cubic foot. Waterproofing weight is the actual weight. The designer should allow for additional ballast depth for future grade or surfacing raises (generally 8” – 12”). On ballasted deck bridges, the roadbed section is assumed to be full of ballast to the top of tie with no reduction made for the volume that the tie would include.

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3 AREMA Manual For Railway Engineering, Volume 2, Chapters 7, 8 and 15

4 AREMA Bridge Loading Seminar 2000, Participant Guide
**Live Loads**

The one component singularly unique to railway structures is the vehicle loading to which it will be subject. Vehicle loading in railway design can be comprised of several parts, including the static load of the vehicle and the dynamic effect of the moving vehicle. The standard loading scheme incorporated by North American Railways and AREMA is the Cooper E-Series loading.

A prominent bridge engineer, Theodore Cooper, first proposed the Cooper E-Series load in the late 1800’s. Prior to this time, the live loads used in bridge design were subject to the judgment of the engineer and tended to vary to the extent that it was difficult to relate the relative strength of one structure to the next. This was a time when many prominent structures were being proposed and constructed on a contractual basis by many railways. It was difficult to objectively compare the proposals of different competing engineers for specific projects.

Although Cooper first published the basis of his proposal with the American Society of Civil Engineers 15 years prior to the establishment of the American Railway Engineering Association, the debate over the superiority of Cooper’s loading scheme versus other proposals remained unsettled past the turn of the 20th century. It would seem that the adoption of his loadings in the first edition of the AREA Manual for Railway Engineering in 1905 seemed to settle the debate. Though widely applied prior to and subsequent of this event, it would be nearly 20 years before all major North American railways incorporated E-series loadings in railway structure design.

Cooper E-series loading consists of two 4-driving axle steam locomotive and tenders followed by a uniform load. See Figure 8-49.

![Figure 8-49 Coopers E-80 Loading](image)

The E-series loading is scaleable with the number representing the driving axle load in kips. An E-80 loading is eight times heavier than an E-10 load.

Despite the fact that Cooper E-series loadings are not representative of today’s equipment, they are commonly applied when designing or evaluating railway structures. The continued application of these loadings is in part due to the legacy of the structures, which remain (i.e., were constructed long before the advent of modern railway equipment). Likewise, the specifics of the E-series loading are not entirely
unlike many of the loadings produced by modern equipment despite the fact that the vehicles are very different.

In load rating situations, loads are converted to E-series ratings for comparison. For example, a modern-day coal train on any specific bridge may equate to an E-58 load on a specific bridge, while a passenger train may equate to an E-41. That same bridge may have a service rating of E-62 and an ultimate rating of E-71. This practice has lead to a wide amount of confusion over the serviceability of existing bridges, which may have been constructed nearly 100 years ago (when the prevailing standard was E-50).

The first key to understanding is that the rating of the coal and passenger train of E-58 and E-41, respectively, is specific only to the bridge in the example. Because the actual loading was converted, the same trains will likely rate as something different on another bridge. An intermodal train will rate as something completely different than the other two trains considered.

Secondly, the bridge ratings provided represent the limiting structural member. During the rating process, each structural member is evaluated for strength and fatigue, where required, and assigned an E-series rating. This rating represents the bridge loading which would produce the maximum allowable load on the member under consideration. At the end of the process, each component is compared with the component rating the lowest representing the load rating of the bridge. A summarized example of bridge rating results for an open deck plate girder bridge is shown in Figure 8-50.

<table>
<thead>
<tr>
<th>Component</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing Stiffener</td>
<td>E-71</td>
</tr>
<tr>
<td>Bottom Flange Bending (includes fatigue)</td>
<td>E-62</td>
</tr>
<tr>
<td>Top Flange, Compression</td>
<td>E-71</td>
</tr>
<tr>
<td>Top Flange Rivet Shear</td>
<td>E-127</td>
</tr>
<tr>
<td>Web Shear</td>
<td>E-145</td>
</tr>
<tr>
<td>Web Splice</td>
<td>E-65</td>
</tr>
</tbody>
</table>

Figure 8-50  Bridge Rating Results for an Open Deck Plate Girder Bridge

There are two rating specifications for the bridge in question. The first given is the service rating. This is the maximum loading to which the bridge can be subject without limiting the life of the structure. In other words, the theoretical life of the structure handling this load would be infinite. This rating takes into account fatigue on specific bridge members, and is generally less than the ultimate loading and may or may not be
the same limiting component as the ultimate rating. The ultimate rating is normally used for occasional traffic or in special conditions where the structure replacement may occur in the near future.

It is unlikely that the bridge was actually designed as an E-62 structure. A number of things have occurred since Cooper Loadings were first recommended and used for design. AREMA rating guidelines specifically require that structures under consideration be rated with the current design guidance. As design methods and material behavior have been developed and better understood over the past 100 years since the bridge was erected, design practice may have changed in such a fashion that the original structure was over-designed. Likewise, the structure in the example was erected during a time when the load produced by a locomotive was considerably more significant than the loading produced by the cars that it was moving. Particular components of the live loading, such as the impact factor, are generally more severe for steam locomotives than diesel-electric power utilized today. This difference generally results in older bridges rating higher under today’s traffic environment than they did when first constructed. The AREMA bridge rating guidelines allow for an impact load reduction based upon speed, but no such allowance is made for the design of new structures.

AREMA design guidelines specify 80-90 miles per hour as a practical limit for the recommended design practices. Limited experience with how actual loading and deflection conditions at speeds above 90 mph relate to the current recommendations are left to the engineer’s judgement. Fortunately, freight train speeds approaching 80 mph are limited to a few select corridors, thus allowing for some additional strength allowance for bridges located on other routes.

Recent editions of the AREMA Manual for Railway Engineering (post-1968) recommend E-80 loadings be used for the design of steel, concrete and most other structures. Yet, the designer must verify the specific loading to be applied from the railway. The railway may require a design loading other than the E-80 AREMA guidance. Furthermore, there is no specific direction given for the E-series loading of timber structures other than ‘the live load shall consist of that Cooper loading which will produce a loading equivalent to that caused by the heaviest engine or train load expected to be moved over the structure during its expected life.’ One key element of any railway structural plan set is that the specific loading assumptions be included on the plan for record purposes.

**Alternate Live Loading – 4 Axle**

Though the Cooper E-series loading is generally incorporated throughout the AREMA Manual for Railway Engineering, the specific application will differ depending upon the chapter. Chapter 15, Steel Structures, only, incorporates an alternate loading, which is more representative of the heavy axle loading of modern intermodal equipment and unit trains (both in magnitude of loads and frequency of occurrence), particularly on short spans.
Heavy double stack cars with axle loads of 78,750 lb per axle and 125-ton capacity 4-axle cars (315,000 lb gross weight) are operated on an ever-increasing basis. These cars produce nearly the equivalent of E-80 on shorter spans. The alternate 4-axle load, introduced in 1995, addresses the fatigue problems associated with short span steel members. Heavy unit trains, similar to the ones mentioned above, increase the rate of fatigue life consumption in short members by inducing very high stress levels and increasing the number of stress cycles experienced by the member.

The alternate load has four axle loads of 100,000 lb (25% greater than E-80), and axle spacing similar to those found in typical 4-axle coupled cars. (See Figure 8-51) The alternate live load induces higher moments and shears than an E-80 load on shorter spans. The resulting higher design stresses lead to bigger sections, which are expected to offer more fatigue life under regular operating conditions. For shorter span lengths, the alternate load produces approximately a 25% higher bending moment than the Cooper's E-80 loading. This will control the design of short spans, stringers and floorbeams. For spans greater than 54 ft (approximately), Cooper's E-80 loading governs.

This alternate loading is frequently and incorrectly applied to other structures, including concrete and timber. This makes the design of composite structures complicated, as not only are the material properties between structural components different, but suddenly the loads are different as well.

The loading of structures with multiple tracks also varies slightly between chapters. In general, reduction in live loading is allowed for members receiving live load simultaneously from three or more tracks to model the reduced probability of occurrence. For open deck structures, the live load is assumed to be distributed equally to beams equally-spaced under the rails and no longitudinal distribution of the live load is assumed. For ballasted deck structures, the lateral and longitudinal live load distribution are a function of the distance from bottom of tie to top of supporting structure and the length of the tie, with longitudinal distribution not exceeding the axle spacing.

**Impact**

Impact is an occurrence of dynamic increment and impulsive loads. Train characteristics (speed, structural stiffness of cars and trucks, wheel conditions, make-up
of train consist, etc.), track geometry (line, surface (crosslevel)), tie, rail and ballast condition and structure (component) displacement under load contribute to the impact produced on a structure by moving locomotives and cars. The impact produced can be severe, as much as 200% of the axle load for out of round or flat spot wheels.

Although each of the latter components of impact can be quantified on a one time individual basis, the designer does not have control over their imposition. AREMA has developed empirical relationships based on experimental observations to evaluate design impact values (percentage of live load) for various bridge types. The impact produced is represented as a vertical load applied at the top of the rail at the same location as the Cooper axle loadings, expressed as a percentage of the live load. The impact on a ballasted deck structure can sometimes be reduced compared to that for an open deck structure because of the absorbing effect of the ballasted track.

Steel Structures

For steel bridge design, the percentage of live load attributed to impact is a function of the spacing of the structure supporting elements (girder or stringer spacing) relative to the spacing of the rails (rocking effect) and the distance between supports for the member being designed (span length). Chapter 15, Section 1.3.5 provides the specifics to the calculation for the recommended percentage of live load that is to be attributed to impact. Recognize that the resultant impact percentage will vary for different components of the same bridge.

The AREMA formulae account for the higher impact produced by steam-powered locomotives with their attenuate hammer blows than that produced by diesel and electric equipment. Chapter 15 also provides impact load values for multiple track structures. For spans less than 175 feet in length, each track of a two-track structure will assume the full impact design value. The impact design value may be reduced for a second track for spans greater than 175 feet in length. For more than three tracks, the full value of impact on any two tracks is used for all span lengths. (See Chapter 15, Section 1.3.5c.)

As of the published date of this text, for purposes of bridge rating only, AREMA provides a reduction in impact design values for speeds less than 60 mph. (See Chapter 15, Section 1.3.5 and Section 9.1.3.5.) Impact is also considered when performing fatigue analysis and design. When checking fatigue stresses, impact forces may be reduced for members over 30 feet in length. Check with the latest manual revision to ensure that this information is current.

Concrete Structures

Although the conditions that produce impact are the same for both a steel structure and a concrete structure, the methodology for estimating design impact values varies. AREMA utilizes the live load and dead load to develop a modified ratio for reinforced concrete bridges and the span length of concrete members for evaluating the impact percentage of prestressed members. Reductions in impact may be allowed for
members receiving live load from more than one track in the same manner as that
done for steel structures. Chapter 8 specifies the derivation of the impact percentage
of the live load for reinforced cast-in-place concrete, which differs slightly with the
derivation of the impact percentage to be used for prestressed concrete.

As of the published text date, for concrete structures, AREMA recommends a
reduction in impact percentages for rating purposes only, based on speed. For speeds
less than 40 mph, the impact value may be reduced in a linear fashion from full effect
at 40 mph to one-half the full effect at 10 mph. Again, reduction in impact based on
speed reductions is not applicable to design. Check with the latest manual revision to
ensure that this information is current.

Timber Structures
The dynamic increment of load due to impact related conditions is not well established
for timber structures. The effect of impact is estimated to be less than the increased
strength of timber for the short cumulative duration of loading that railway bridges
experience, and is taken into consideration in the derivation of allowable working
stresses for design. Thus, Chapter 7 (Timber Structures) does not include an impact
factor in structural design due to the material properties of timber.

Culverts and Retaining Wall Structures
Impact is applied on a sliding scale in Chapter 1 of the AREMA Manual for Railway
Engineering when considering culverts and other structures, which have fill or
embankment physically separating the structure being loaded and the track structure.

Centrifugal Load
Centrifugal force is the force a train moving along a curve exerts on a constraining
object (track and supporting structure) and acts outwardly away from the center of
rotation. In the process, both a horizontal force and an overturning moment are
produced.

Both must be considered in design or evaluation of a structure. The horizontal force
tends to bend the structure laterally. For steel structures (deck girders, for example), it
loads laterals and cross frames. For concrete structures (box girders, for example), the
structure is typically stiff enough in the transverse direction that the horizontal force is
not significant. For all 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' above the top of rail, at a point perpendicular to the center of a line connecting the rail tops. It is expressed as a percentage of the live load. The effect of track superelevation compensates somewhat for centrifugal force. (See Chapter 6, Railway Track Design) Note: Impact is not added to centrifugal force.

Rather than applying the centrifugal force at each axle location, most railways simply multiply the calculated live load force by the centrifugal force percentage, factor in the effect of the force location above the top of the rail, and use the resulting value in their calculations.

**Effects of Centrifugal Force**

- **Timber Pile Trestle Bridge**
  - Increases forces in outside stringers and piling
  - Increases forces in pile (sway) bracing

- **Precast Concrete Trestle Bridge**
  - Increases force in outside box girder and piling
  - Increases force in pile bracing
  - Horizontal bending effect on superstructure typically not significant due to lateral stiffness of the structure

- **Rolled or Welded Beam Bridge**
  - Increases force in outside beams
  - Induces lateral bending in beam top flanges

- **Deck Plate Girder Bridge**
  - Increases force in outside girder
  - Increases force in bracing system

- **Through Plate Girder Bridge**
  - Increases forces in outside girder, stringer and outer section of floorbeams
  - Increases forces in stringer and girder bracing

- **Through Truss Bridge**
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- Increases force in outside stringer and truss
- Increases force in bracing system

Lateral Loads

Lateral loads are applied to the structure as a result of routine train passage, excluding centrifugal force. This is largely due to the nosing (the tendency of the train to bear laterally against the rails as it travels down the track) and hunting action of the train as it traverses the bridge. Lateral force manifests itself as horizontal forces on specified bridge members including lateral bracing members, flanges of longitudinal girders or stringers without a bracing system, and to the chords of truss spans. The magnitude and application point of these loads vary among Chapters 7, 8 and 15. As of the published date of this text, 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. Check with the latest manual revision to ensure that this information is current. Lateral loads from equipment are not included in the design of concrete bridges. However, if concrete girders are supported on steel or timber substructures, lateral loads should be applied to the design of those members.

Experience has shown that very high lateral forces may be applied to structures due to lurching of certain types of cars. Wheel hunting is another phenomenon, which applies lateral force to the track and structure. Damaged rolling stock (slewed trucks, binding center plate, etc.) can also create large lateral forces. Although there is not extensive research background supporting the lateral forces developed in the AREMA Manual for Railway Engineering, they have historically worked well when combined with wind loads to produce adequate lateral resistance in structures.

Effects of Lateral Forces from Equipment

- Timber Trestle Bridge
  - Not applied to stringers
  - Apply to bents (resulting vertical forces can be neglected)
- Precast Concrete Trestle Bridge
  - Effects not applicable
- Rolled or Welded Beam Bridge
  - Lateral bending of flanges
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- Deck Plate Girder Bridge
  - Axial forces in girder spacing
- Through Plate Girder Bridge
  - Axial stresses in stringer bracing or lateral bending of stringers without bracing
  - Axial stresses in flange bracing
- Through Truss Bridge
  - Axial stresses in truss chords
  - Axial stresses in cross-frames of truss
  - Axial stresses in stringer bracing or lateral bending of stringers without bracing

**Longitudinal Loading**

Longitudinal force is induced on the structure along its length from train movement (braking and accelerating) on the bridge. Longitudinal force manifests itself as a horizontal force parallel to the rail and distributed into the supporting structure.

Generally, the design load is the maximum of either the braking or accelerating force of the train. Each of these is figured independently of the other. The chapters differ in their consideration of the acceleration (traction) aspect of the force. In each chapter, the braking and traction forces are compared, and the larger value used in design.

As of the published date of this text, Chapter 7 of the AREMA Manual for Railway Engineering takes the longitudinal force due to braking to be 15% of the live load, without impact, while Chapters 8 and 15 specify the longitudinal load due to braking to be $45 + 1.2 \times (L)$, where $L$ is the span length in feet and the load is specified in kips. For the acceleration (traction) aspect of the force, Chapter 7 uses 25% of the weight on the drive axles for traction, while Chapters 8 and 15 use $25 \times (L)^{1/2}$, where again $L$ is the length of span in feet and the load is designated in kips. Check with the latest revision of the manual to ensure that this information is current.

With the advent of the high-adhesion AC locomotives, there is much concern in the industry that the AREMA-based design load percentage for longitudinal loading for timber structures, as recommended, may be significantly understated. Check with the specific railway for their current policy on longitudinal force application.

Chapters 7 (Timber Structures), 8 (Concrete Structures) and 15 (Steel Structures) differ in the point of application of the longitudinal force. Chapter 7 applies it 6 feet above
the top of rail. Chapters 8 and 15 apply the braking force at 8 feet above the top of rail, and the traction force 3 feet 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, less than 200 feet long, ballasted deck bridges with short spans less than 50 feet in length, 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.

Effects of Longitudinal Force

- Timber Trestle Bridge
  - If much or all of force is not carried directly to embankments, will apply longitudinal force to the bents.

- Precast Concrete Trestle Bridge
  - Applies longitudinal force to the substructure.

- Rolled or Welded Beam Bridge
  - Applies longitudinal force to the substructure.

- Deck Plate Girder Bridge
  - Applies longitudinal force to the substructure.

- Through Plate Girder Bridge
  - Applies longitudinal force to the substructure.
• Through Truss Bridge
  - Applies longitudinal force to the substructure.
  - Introduces force in truss bottom chords and laterals.

**Wind Loading**

Wind loading is the force produced on the structure due to wind action on both the bridge and the train. Wind loading produces a horizontal force and an overturning moment. Both must be considered in the design or evaluation of a structure. Chapters 7, 8 and 15 treat wind on the structure slightly differently. Though all chapters treat it as a moving load perpendicular to the track, the magnitude varies between timber, steel and concrete.

For example, the 30-lbs/ft² wind force on a loaded structure and 50-lbs/ft² force on an unloaded structure used in Chapter 15 reflects assumptions on train operations. It is assumed that the maximum wind velocity, under which train operations would be attempted, would produce a force of 30 lbs/ft². Hurricane winds, under which train operations would not be attempted, would produce a wind force of 50 lbs/ft².

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

**Effects of Wind Loading on Bridge (Loaded or Unloaded)**

- Timber Trestle Pile Bridge
  - Increased force in leeward piling
  - Increased moment in leeward stringers (overturning effect)
  - Increased force in pile bracing

- Precast Concrete Trestle Bridge
  - Increased force in leeward piling
  - Increased moment in leeward girders (overturning effect)
Increased force in pile bracing

Lateral bending can be neglected in superstructure due to high lateral stiffness

- Rolled or Welded Beam Bridge
  - Lateral bending in flanges
  - Increased bending moment in leeward beams (overturning effect)
  - Lateral load transmitted to substructure

- Deck Plate Girder Bridge
  - Lateral bending in flanges
  - Increased forces in bracing
  - Increased bending moment in leeward girder (overturning effect)
  - Lateral load transmitted to substructure

- Through Plate Girder Bridge
  - Lateral bending in flanges
  - Increased forces in bracing
  - Increased bending moment in leeward girder (overturning effect)
  - Lateral load transmitted to substructure

- Through Truss Bridge
  - Increased force in leeward truss (overturning effect)
  - Increased force in truss bracing and chords (lateral bending)
  - Lateral load transmitted to substructure

Stream Flow, Ice and Buoyancy

Stream flow, ice and buoyancy loads are applied to the structure because of its location in a body of water. These loads provide horizontal and/or vertical forces as well as overturning moments.
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, scour and ice pressure are to be applied to any portion of the structure that is exposed therewith. This typically includes piers and other elements of the substructure. While the AREMA Manual for Railway Engineering does not address design forces for stream flow and ice pressure, the AASHTO LRFD and other bridge design specifications do include procedures for calculating them.

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 before designing a bridge across a navigable waterway.

The second factor to be addressed 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 watersheds upstream of a bridge change.

Seismic Loads

Seismic loads are induced by horizontal and vertical forces in the structure, resulting from earthquake ground motion. Seismic design for railways is covered in Chapter 9 of the AREMA Manual for Railway Engineering. In general, the provisions are similar to AASHTO.

The philosophical background of Chapter 9 recognizes that railway 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 dampening agent) against bridge movement.

2. Railway bridges are typically simple in their design and construction.

3. Trains operate in a controlled environment, which makes types of damage permissible for railway bridges that might not be acceptable for structures in general use by the public.

The third item above is related to the post-seismic event operation guidelines given in Section 1.2 of Chapter 9. This section gives 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 railway policies may vary from this.)
Three levels of ground motion are defined:

- **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 railway 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 railway 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 book. Guidelines are given in Chapter 9. Base acceleration coefficient maps for various return periods are included in the chapter. It should be noted that Section 1.4.2.d. (4) states that no seismic analysis is necessary for locations where a base acceleration of 0.1g or less is expected with a 475-year return period. For most locations in North America, therefore, a seismic analysis would not be needed. (Canadian and Mexican requirements may differ from this. Check with the affected railway for specific guidelines.)

Important structures should be designed for higher seismic loads than non-important 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. The provisions of adequate bearing area and designing for ductility are examples of inexpensive seismic detailing. Typically, seismic loading is not considered in combination with live load, although engineering judgment should be used in cases where trains may be parked on a bridge (e.g., a bridge in a yard).

Chapter 9 also includes seismic provisions for items other than bridges. These other items include buildings, culverts, tunnels, cuts, fills and signal equipment. The chapter is frequently updated to reflect current research into seismic performance of railway structures. The designer should be aware of the limits of their expertise in this area. Consult with engineers experienced in seismic design of railway bridges as necessary.
Combined Loads

Several types of loads may be applied to a structure simultaneously. These loads are combined in a prescribed manner to produce design loads for the bridge. For example, a bridge may experience dead load, live load, impact, centrifugal force, wind and stream flow simultaneously. The AREMA Manual for Railway Engineering 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.

Like the individual load calculations, the combined load effects recommended by each chapter vary from one another. 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. Likewise, the combinations are treated slightly different within each chapter between allowable stress and load factor design. 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.

8.5.3 Other Structure Design Criteria

Fatigue

Cracking or fracture due to repetitive loading is the result of fatigue. The repetitive loading that causes fatigue fracture produces stresses in the material below its yield stress. Due to the magnitude of live loads, fatigue is generally more dominant for railway bridges than highway structures.

An example of fatigue failure that is easy to replicate may be performed with a paper clip. Using a small paper clip, bend it between your thumbs and count how many bends it takes until the paper clip breaks. If you were to repeat this exercise for numerous paper clips, recorded the results, and then found an average number of bends, you could establish a ‘fatigue limit’ for your particular brand of paper clips. The stress you induce in the paper clip for each bend is far less than the stress required to break the paper clip in a single bend or pull, but the repetitive loading produced small cracks in the paper clip, eventually causing it to fracture.

The small-scale scenario demonstrated above can also be used to predict the fatigue limit for steel bridge members. Research has provided allowable stress ranges for various fabrication details. In Chapter 15 (Section 1.3.13) of the AREMA Manual for Railway Engineering, various joint and connection details are enumerated (1-23) then assigned a stress range category (A-F). This allows the engineer to design and detail the structure, avoiding fatigue prone details in areas of high stress range. The stress range refers to the difference between the maximum and minimum stress. In the case of simple spans, which are most common in railway construction, this difference is the
stress due to Live Load and Impact (LL + I). In some cases, a reduction in impact is permitted when calculating stress range. This is referred to as the Mean Impact Load and is based on the span length.

The three major factors affecting the fatigue strength of connections are:

- The number of cycles, N, causing tension loading.
- The magnitude of the stress range.
- The type and location of the construction detail.

Fatigue loads will frequently require the designer to use bolted connections in locations, such as intermediate stiffeners, that would usually be welded on highway bridges. It is also important to note that the allowable stress range for Fracture Critical Members is different than Non-Fracture Critical Members. When designing steel structures, it is the responsibility of the engineer to select the proper materials and provide adequate details that meet the requirements of AREMA design specifications as well as the requirements of the governing railway.

**Fracture Critical Members (FCM)**

The AREMA Manual for Railway Engineering defines and describes Fracture Critical Members (FCM’s) in Chapter 15 (Section 1.14). AREMA defines a fracture critical member (FCM) as:

> “Those tension members or tension components of members whose failure would be expected to result in collapse of the bridge or inability of the bridge to perform its design function.”

Strict interpretation of this definition would suggest that nearly any tension member would be classified as fracture critical, as it is specifically required for the bridge to remain serviceable. This is somewhat impractical in that if a particular bridge member were not required for the bridge to remain serviceable, then speculation would question its overall usefulness. As applied to North American railways, the definition has been curtailed such that it applies to those members whose failures would result in a catastrophic bridge failure.

AREMA and AASHTO both consider FCM’s as non-redundant, meaning an alternate load path does not exist. FCM’s are held to more stringent design criteria, as well as welding and testing procedures, therefore it is imperative to identify all FCM’s on the bridge drawings. These requirements are also known as a Fracture Control Plan, the details of which are outlined in Section 12 of AWS D1.5. It should be noted that some railways, which perform their own steel fabrication, may not be qualified or equipped to perform welding or testing for FCM’s.
It is the responsibility of the engineer to first identify Fracture Critical Members, then to perform design in accordance with the AREMA Manual for Railway Engineering. There are certain limits for FCM’s that do not exist for other members, the most notable being the allowable fatigue stress range requirements established in Table 15-1-15.

Some common Fracture Critical Members are listed below:

- The web and bottom flange of girders in a two-girder, simple span structure.
- Certain tension members of trusses, such as bottom chord and some diagonals.
- The hanger components of pin and hanger assemblies.

**Structure Serviceability**

**Deflection**

Excessive deflection in a railway structure can provide severe serviceability problems that may impinge on the owner’s ability to safely use the structure even though allowable stresses are not exceeded. Deflection in the structure will exacerbate line and surface problems, thereby increasing the impact forces exerted on the structure.

Because of the various adverse effects of high deflection levels, especially from the serviceability standpoint, it is in the designers’ interest to control the levels of deflection in a structure. This is particularly important with today’s high strength steels, which permit the use of smaller and lighter members, based on allowable stresses.

Any structure resists applied loads by deflecting under load application to appropriate levels. The extent of deflection of various structural components depends on the applied loads and the stiffness of the various structural components. The stiffness of the member in turn is dependent on its type (axial, flexural, etc.), length, cross-section and elastic modulus of the material. Traditionally, the design of structures has been governed more by strength concerns than by deflection concerns. However, with the increasing use of long span structures and higher strength materials, deflection levels might control the design.

Generally, a designer can control deflection levels in a structure either by varying the configuration of the structure or the cross-sections of members. Controlling deflections by varying the section properties is usually a more practical approach.

Various codes and recommended practices have generally limited deflection levels in a structure by:

- Specifying limiting span-to-depth ratios for structural members,
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- Specifying maximum deflection levels as a ratio of the span length, or
- An appropriate combination of the above two criteria.

For Steel Structures, Chapter 15, Section 1.2.5 states, “The deflection of the structure shall be computed for the live loading plus impact loading condition producing the maximum bending moment at mid-span for simple spans. In this computation, gross moment of inertia shall be used for flexural members, and gross area of members for trusses; except that for members with perforated cover plates, the effective area shall be used. The effective area shall be the gross area reduced by the area determined by dividing the volume of a perforation by the distance center to center of perforations.” Further, “The structure shall be so designed that the computed deflection shall not exceed 1/640 of the span length center-to-center of bearings for simple spans.”

Chapter 15, Section 1.2.10 specifies criteria requiring camber in trusses and plate girders. For trusses, the camber required is equal to the deflection produced by the dead load plus a live load of 3,000 lbs/lf of track. For plate girders longer than 90 feet in length, the required camber is equal to the deflection produced by the dead load alone. Shorter plate girders and rolled beams do not require camber.

In concrete structures, AREMA limits deflections by specifying minimum thickness of members as a function of the span length as found in Chapter 8, Section 2.40.2, Table 8-2-8. Lesser thickness is permitted if the deflection calculations indicate no adverse effects. The deflections calculated are based on the entire superstructure section or any element placed monolithically with the superstructure before falsework or shoring removal.

Deflections of concrete portions of composite members shall take into account shoring during erection, differential shrinkage of the elements and the magnitude and duration of load prior to the beginning of effective composite action. Computation of live load deflection may be based on the assumption that the superstructure flexural members act together and have equal deflection. The live load shall be considered uniformly distributed to all longitudinal flexural members.

Computation for deflections occurring immediately on application of load as well as for long-time deflections must be performed and are covered in Chapter 8, Section 2.23.7.

Deflection of ties and stringers is the primary concern in timber structures. Stipulating depth and width ratio controls stringer deflection. Per Chapter 7, Section 2.4.3b, stringer depth should not be less than 1/12 of the span and the width not less than 1/3 of the depth. Deflection equations for load distribution for ties, stringers and pile bents are provided in Figures 7-2-3 and 7-2-4 of Chapter 7.
**Corrosion**

New spans are usually not painted considering the initial and maintenance costs. Several weathering grades of steel, including A588 or A709, Grade 50W and A852 or A709, Grade 70W, have atmospheric corrosion resistance in most environments better than that of carbon steels, with or without copper addition, and can often be used unpainted. Do not use weathering steels in environments not suited to weathering steels, or in projects in highly visible locations paid for and maintained by agencies other than the railway company.

**Welding**

The fatigue concerns related to railway loading, coupled with the overall expected structure life, have resulted in some limits in the use of welded connections, requiring bolted connections in their stead. This aspect of bridge design and construction is subject to great variation between individual railways, pertaining to the type and specific applications of welds. The designer is strongly encouraged to discuss the railway’s preferences towards the application of all welded connections prior to the commencement of the design.

Table 15-1-9 and Figure 15-1-4 of Chapter 15 specify construction details for various groove and fillet weld connections, as well as examples of potential weld produced fatigue conditions.

**Bearings and Volumetric Changes**

Chapter 19, Bridge Bearings is currently under development in the AREMA Manual for Railway Engineering. Here the writers have outlined and defined the basic components of bearings for non-movable bridge spans. The fundamental concepts of bridge bearings are simple but their design is somewhat more complicated. The basic principle is to transmit all loads from the superstructure to the substructure while still permitting rotation and translation of the attached superstructure element. In the simplest form, bearings may be merely plates or pads placed beneath the ends of a beam/girder with little or no positive attachments. On the other end of the spectrum are multi-rotational (pot) bearings or seismic isolation bearings. These types of bearings should only be detailed under special conditions that warrant their use.

Selecting the type of bearing can often times be difficult. The bearing type selection is based primarily on the magnitude of loads and the required movement range. Many times this selection will be empirical based on the superstructure type, span length, and above all, owner preference. Schematic diagrams of some bearing types can be found in Chapter 19 of the AREMA Manual for Railway Engineering

Volume changes in structures can be caused by thermal expansion or contraction or by properties of the structural materials, such as creep or shrinkage. Restrained volume changes can produce significant forces in the structure. Volume changes in themselves, if unrestrained, have relatively little effect on the forces on 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 length change due to temperature per every 100' of span length.

Chapter 8 provides a table for design temperature rise and fall values based on concrete temperature. Provisions also 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.

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 for Railway Engineering contains procedures for calculating the shear force transmitted through bearing pads. For steel bearings, a rule of thumb is to apply 25% of the dead load reaction as the maximum force that can be transmitted through an expansion bearing before it moves. 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.

A specific railway may have different requirements for thermal movement. Expansion bearings are the main design feature typically used to accommodate volume changes. Common bearing types included:

- Sliding steel plates
- Rocker bearings
- Roller bearings (cylindrical and segmental)
- Elastomeric bearing pads

Provision should be made for span length change due to live load. For spans longer than 300 feet, provision must be made for expansion and contraction of the bridge floor system. It should be noted that movement of bridge bearings affect 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 rail). Maintenance requirements are also important when selecting bearings, since unintended fixity due to freezing of bearings can cause significant structural damage.
Composite Design

The design and use of composite steel and concrete spans for railway bridges is addressed in Section 5.1 of Chapter 15 of the AREMA Manual for Railway Engineering. This type of superstructure comprises a steel beam or girder and a concrete deck slab. The connection between the two materials is designed and constructed to transfer adequate shear force, such that the two materials behave as a single, integral unit under load.

The theory of composite design, governing the recommendations in the AREMA Manual for Railway Engineering, is very similar to that found in the working stress method in AASHTO and allowable stress methods in various building codes. Some of the important issues include:

- Selection of the effective flange width of the concrete as a function of slab thickness, steel beam spacing or span length;
- Proportioning of the cross-section by the moment-of-inertia method;
- Application of the dead load forces to the non-composite or composite section, depending on construction sequencing and methods;
- Considering the effect of creep due to long term dead loads acting on the composite section.

Shear connectors may be either steel channels or headed studs welded to the top flange of the steel and embedded in the concrete deck. Reference is made to the AREMA Manual for Railway Engineering or engineering textbooks for specifics on the preceding items.

Additional consideration is warranted for railway bridges in other aspects of design, however. One issue to address in composite design is the magnitude of live load to be resisted. Although not specifically addressed in the AREMA Manual for Railway Engineering, railway companies generally require that the steel beams or girders be proportioned to carry without contribution from the concrete deck slab, a Cooper’s live load of only a slightly reduced magnitude than that of the entire structure. For example, a bridge with a composite design load of E-80 is often required to have the steel section alone provide support for an E-60 or higher, and maybe as much as E-80 as well, depending upon the railroad and the type of structure considered. This ensures that if the concrete deck is damaged during a derailment, the steel section will be sufficient to carry rail traffic, even if the concrete must be torn out and an open deck installed.

If the steel alone is sized for the design load, the cost savings through efficient use of materials is somewhat less for railway structures than it is for highway and building structures that make full use of the composite section to resist live load and impact.
The limitation on deflection due to live load plus impact also can usually be waived when considering the steel only for railway bridge design. The full composite section should be designed as being sufficiently stiff to meet the deflection limitations.

Even if the steel section is adequate to carry the final design loading without contribution from the concrete slab, composite action still must be investigated. The neutral axis of the composite section will be higher on the cross-section than that for the non-composite section. This will increase the stress range in the material below the neutral axis, and fatigue details should be checked for this increased range.

While composite steel and concrete spans provide a stiff design with the benefits of a ballasted-deck bridge, they are unlikely to be used to replace existing structures on existing alignments. Compared to precast concrete deck panels, the additional time required to form, place and cure the cast-in-place concrete deck of a composite span requires off-line construction to minimize impact to rail operations. Parallel construction of a composite span with a lateral roll-in during a train free window is one way to work around this problem. Additionally, since the deck concrete is not under compression from prestressing or post-tensioning, the use of a waterproofing system to protect the deck may be warranted.

Where structure depth is limited by vertical clearances below the structure, a steel plate may be used instead of a concrete deck. The steel plate may or may not be included in the beam design, depending on the connection to the beam.

**Bridge Design Assumptions and Constructibility Issues**

When planning railway structures, it is imperative to be mindful of the factors that frequently control design and construction. Many in the railway industry would agree, that the driving factor of design and construction is track time. Operations are key, and with greater traffic demands on an ever-aging infrastructure, track time is at a premium. It is important for railways to balance time for operations, maintenance/repair and new construction. The designer is challenged with producing plans and specifications that will yield the best structure in the shortest amount of time.

Many times the design efficiency is sacrificed for a shorter construction period. Let’s briefly examine one simple scenario: the superstructure replacement of a short, single-span bridge. In it’s nearly 100-year life span, the steel superstructure had been raised while being converted from a ballasted deck to an open deck. This conversion included the use of what is known as a grillage. A grillage (also known as cribbing) is a temporary steel support, usually in the form of short sections of steel H-piles, welded together side-by-side to form a shallow (1-2 foot) bearing seat.

The steel is subsequently encased in concrete. This technique is most common to rehabilitation projects. When replacing this superstructure, the most efficient design might include cutting the backwall, removing the grillage (thus lowering the beam seat)...
and/or using a shallow, multi-girder, ballasted concrete deck system. This design would take weeks of construction, if not months to complete. However, the most efficient design (from a track time perspective) might include the use of deep girders, oversized bearings, and a timber deck. Using this type of superstructure, the span may be replaced in a matter of hours, thus minimizing interruptions to train movements.

As simple as it may seem, the scenario demonstrated above exemplifies the decision process of engineers in the railway industry. Techniques that may be considered antiquated or over-conservative to engineers in the highway industry are common to railway structure design. For instance, simple spans and bolted construction (i.e. stiffener bolted to the web plate) are still widely used for railway bridges, whereas continuous spans with welded stiffeners are standard practice in highway bridge design.

The use of bolted construction reduces fatigue requirements, and simple spans allow for the replacement of each individual span, thus minimizing traffic interruptions. When constructing railway bridges, it has become common practice to erect spans in a nearly complete form in order to expedite span change-out. Steel spans and precast concrete box beams as well as other superstructure types may be shipped to a construction site fully assembled, sometimes including the track. These spans are lifted into their permanent location and traffic is restored quickly.

Such practices may have added construction costs in the form of shipping larger sections or the need for larger cranes required to lift heavier segments. The engineer is charged with the task of producing a design that is both an economical use of materials and labor as well as reducing interference with train operations. Often the outcome of a design is counter-intuitive to the standard practice of producing highly efficient structural systems that use a minimum amount of material. This break from the common practice, in the long-term, proves more beneficial to railway companies due to the savings yielded from a design that lasts many years, requires minimal maintenance and provides a construction period that keeps trains moving.

**Recommended Construction Considerations**

- **Safety**
- **Labor Issues**
  - Flagging
  - Work Trains
  - Railway Collective Bargaining Agreements in Force
  - Contractor/Labor Agreements
- **Operational Impacts**
• Complying with the design requirements
  o Sequence of Work
  o Schedule
  o Coordination of Work
• Impact on:
  o Adjoining Land Owners
  o Other Contractors
  o Railway Departments
• Availability of Material (Delivery)
• Regulatory Permits
• Local Ordinances
• Disposal & Salvage
• Site Security
  o Construction Access
• Staging Area
• Environmental Controls
  o Seasonal Issues
  o Weather
  o Best Management Practices
  o Storm Water Retention
  o Construction Entrances
  o Erosion Control
  o Silt Fences
8.5.4 Retaining Wall Loads

Railways frequently incorporate retaining walls within the right-of-way. There are countless systems available to the designer when considering a retaining wall solution. Some systems, which include some mechanically-stabilized-earth (MSE) systems and some interlocking block systems are not favored by railroads due to concerns over the long-term (upwards of 100 years) serviceability of the structure, or over concerns relating to the vibratory loads on structures within close proximity to the track.

Of primary concern is the relative location of the wall to the live-loading envelope of the track structure. The location of all or part of the wall within this envelope can significantly increase the size and strength of the wall with only a slight change in the relative distance of the wall to the track.

Part 5 of Chapter 8 in the AREMA Manual for Railway Engineering addresses Retaining Walls and Abutments. It should be noted that these provisions apply to permanent construction. Other parts of Chapter 8 address specific types of retaining structures, such as crib walls (Part 6), and flexible sheet pile bulkheads (Part 20).

For abutments and structures approximately perpendicular to the track center, the track load is distributed uniformly on the ballast over the width of the tie. The distribution width spreads on a 1 horizontal to 2 vertical slope with increasing depth to a maximum of 14’.

For track loading approximately parallel to a wall, the load is distributed uniformly over the width of the tie. The AREMA Manual for Railway Engineering requires calculations of stability of the retaining structure. No specific check of stability against overturning is required if the resultant force on the base falls within the middle third for structures founded on soil, and the middle half for structures founded on rock, masonry or piles. A safety factor of 1.5 is required for sliding.
Frequently, temporary retaining structures (such as shoring) are required for construction adjacent to railway tracks. It is important that the guidelines of the specific railway for construction near their tracks be followed.

References:

2. “AREMA Bridge Loading Seminar - Participants Manual.”