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38.1 Introduction

The principles of designing railroad structures are similar to those for structures carrying highways. However, structures carrying railways have much heavier loadings than those subject to highway loadings due to increased dead load, live load and impact required for railways.

The general features of design, loadings, allowable stresses, etc., for railway structures are controlled by the specifications of the American Railway Engineering and Maintenance-of-Way Association (AREMA). The different railroad companies vary somewhat in their interpretation and application of these specifications as stated in the *AREMA Manual for Railway Engineering* (hereafter referred to as *AREMA Manual).* Requirements for railroad structures vary with the railroad company whose tracks are carried by the structure, and are sometimes varied by the same company in different locations. The *AREMA Manual* provides for design of railroad structures using Allowable Stress Design (ASD) and Load Factor Design (LFD) methods. The Load and Resistance Factor Design (LRFD) method is currently not used. Designers should bear in mind that specifications were developed for more or less typical conditions. If a structure is unusual in some respects, designers should use their best engineering judgment in selection of proper design criteria. Most railroad companies permit and prefer high strength bolted or shop welded steel plate girders, reinforced concrete or prestressed concrete members in bridge construction.

Safety of the traveler on the highway under the structure and uniformity of track surface dictates that the full ballast section of the railway be carried on the structure. The relatively heavy loadings and high impact factor together with the span and clearance requirements usually found in underpass structures, practically limit the choice of materials for the superstructure to structural steel. The floor under the ballast may be steel plate or reinforced concrete and the substructures could be reinforced concrete or structural steel as conditions warrant.

The *AREMA Manual* covers all phases of railway design, construction, maintenance and operation. It is divided into sections and chapters. Chapter 8, Concrete Structures and Foundations (Volume 2), governs the design and construction of plain and reinforced concrete members, rigid concrete structures, retaining walls, pile foundations, substructures of railway structures, etc. Chapter 15 - Steel Structures (Volume 2), governs the design and construction of steel railroad structures.

In this chapter, reference will be made to specific articles of the *AREMA Manual* as required.

The AREMA specifications are revised annually and it is essential that the latest revisions be used. The *AREMA Manual* is a guideline only and should be followed as a starting point in design.

Railroad companies are essentially conservative as their primary interest is the safety of their trains and human lives. Their requirements are usually based upon their operating experience and are set up with that interest in view.

38.2 Design Specifications and Design Aids

38.2.1 Specifications

Allowable stresses are provided in different chapters and sections of the *AREMA Manual*.

Refer to the design, construction, maintenance and operation related materials as presented in the stated sections of the following specifications:

American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering

- Chapter 8 - Concrete Structures and Foundations (Volume 2)
- Chapter 15 - Steel Structures – (Design, Fabrication and Construction) (Volume 2)
- Chapter 28 – Clearances - (Horizontal and Vertical) (Volume 4)

AASHTO Standard Specifications for Highway Bridges, 17th Edition

Wisconsin Standard Specifications for Highway and Structure Construction

38.2.2 Design Aids

In the design of railroad structures the only short cut available is a method of computing Live Load Moments, Shears and Reactions by the use of tables which can be found in Section 1.15 of Chapter 15, part 1 of the *AREMA Manual*. This table reflects Cooper E80 Live Loading shown in Figure 38.3-5. All the moment, shear and reaction values are for one rail (one-half track load) only and all the values can be prorated (directly proportional) for smaller or larger Cooper's E live loadings.

Floor beam spacings in through plate-girder railroad structures may be determined by a number of things, but consideration should be given to the transverse stiffener spacings of the girders. It is very convenient to have the floor beam spacing in multiples of stiffener spacings.

For ballasted structures, all lateral forces will be carried by the steel ballast plate which is extremely rigid and lateral bracing will not be required.

38.2.3 Horizontally Curved Structures

The latest AREMA specifications as well as individual railroad company's interpretation and application of the *AREMA Manual* should be followed in designing and detailing curved structures. There is considerable information available on designing curved steel girders. Most of the methods require computer programs that may be difficult to use. The Approximate Method of Design developed by USS Corporation is an accepted approach for horizontally curved girders.

38.2.4 Railroad Approval of Plans

There is a need to get the individual railroad company's unique design requirements. Smaller companies such as Wisconsin & Southern may rely on AREMA requirements and DOT experience.

Prior to starting the preliminary design, the Bureau of Structures (BOS) should receive the railroad company's current standards and design policy guidelines.

Before the preliminary plan is prepared, the Regional Project Manager, BOS and Bureau of Rails and Harbors (BRH) should review the particular railroad company's design standards for compliance with 23 CFR (Code of Federal Regulations) and DOT policy, and for compatibility and practicality with unique project features.

The preliminary structure plan should be prepared and submitted to the railroad company for approval after the above steps have been completed.

Detailed structure design should not begin until the railroad company has approved the preliminary plan.

The bridge designer should work directly with the railroad's bridge engineering office where interpretation of requirements or clarification of design details is needed.

The final structure plan and special provisions need to be sent to and approved by the railroad company before the project is authorized for letting.

38.3 Design Considerations

38.3.1 Superstructure

38.3.1.1 Methods of Design, Selection Type and Superstructure General

The preferred types of railroad structures are as follows:

- Rolled or welded girders for spans of 50 feet or less
- Bolted or welded plate girders for spans over 50 to 150 feet
- Bolted or welded trusses for spans over 150 feet

The superstructures of grade separations carrying railroad traffic are usually of beam and girder construction. The spans are generally too short for economical use of trusses and other factors, such as appearance, maintenance, etc., discourage their use.

Floor systems in beam and girder construction, for moderate spans, may be divided into two general classes:

- One-way Floor System
- Two-way Floor System

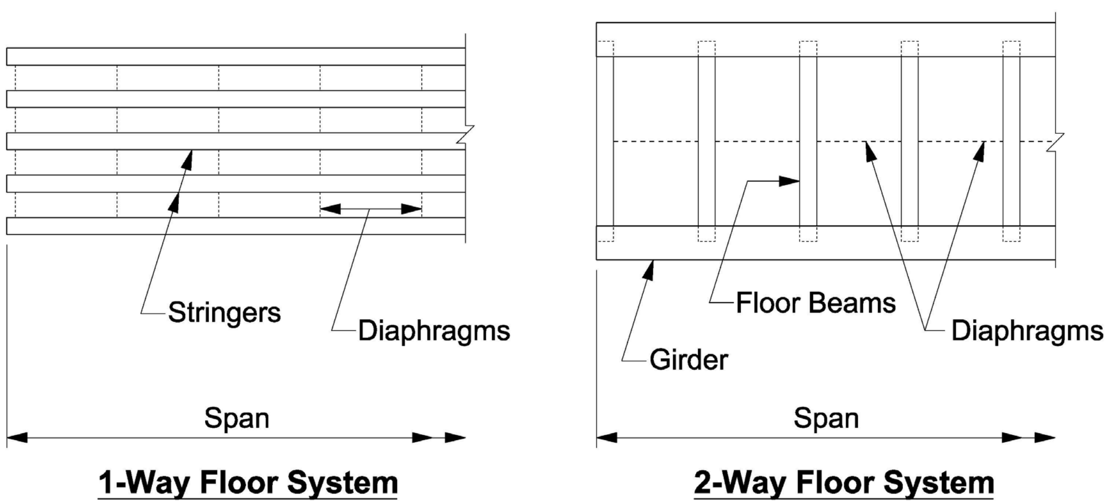

Figure 38.3-1
Types of Floor Systems

The One-way floor system is always a deck structure and is particularly adaptable for structures carrying several tracks or subject to future widening or other controls which make a deck structure desirable. The Two-way floor system may be either a through plate-girder or deck structure depending upon whether the floor beams are placed near the bottom or the top flange of the girders. It is usually desirable to keep the depth of structure (base of rail to low steel) at a minimum. Therefore, most underpasses with two-way floor systems are through structures as shown in Figure 38.3-2.

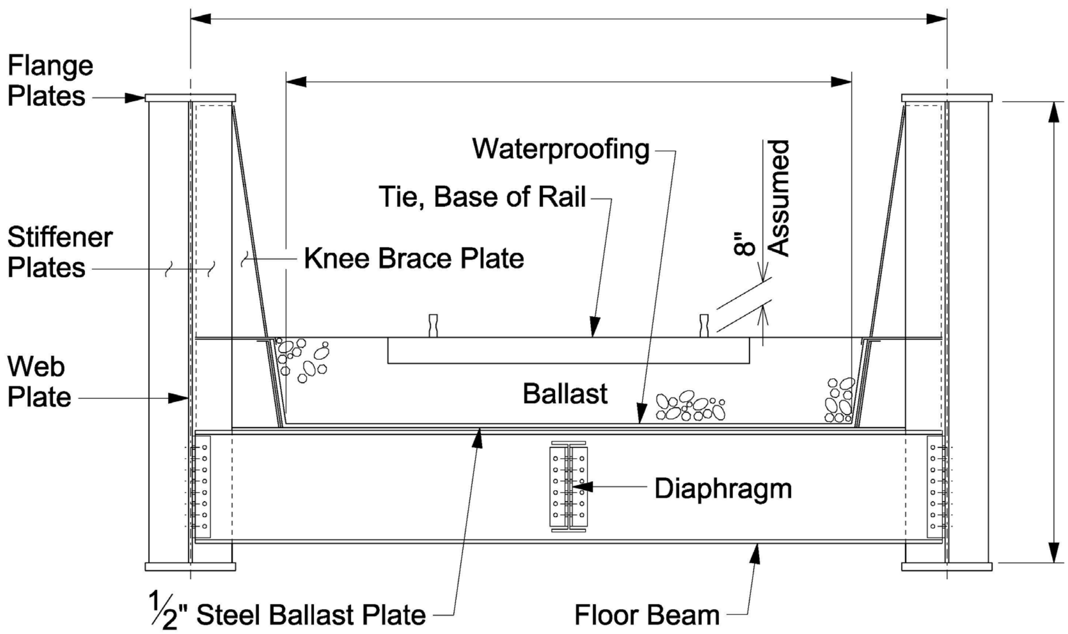

Figure 38.3-2
Typical Section of Through-Girder Bridge
(Two-Way Floor System)

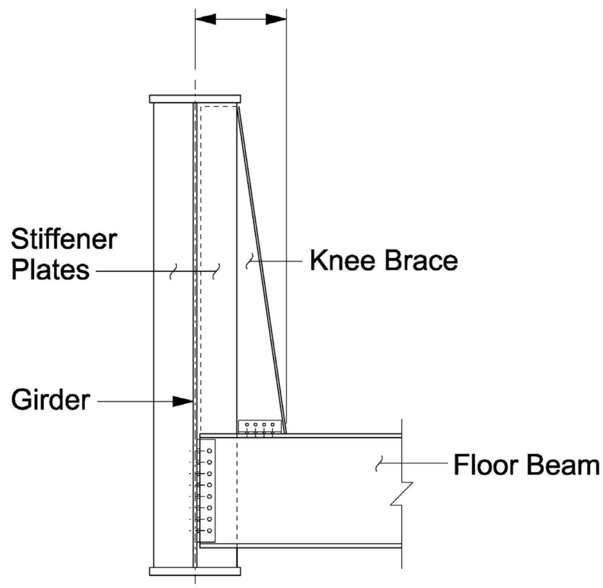

Figure 38.3-3
Knee Brace for Through-Girder Bridge

Through girders should be laterally braced with gusset plates or knee braces with solid webs connected to the stiffeners as shown in Figure 38.3-3. The *AREMA Manual* limits the spacing of knee braces to 12 feet maximum. They also dictate that the type of braces are to be web plates with flanges. Since knee braces support the top flanges against buckling, smaller values of L/b (L = unsupported distance between the nearest lines of fasteners or welds, or between the roots of rolled flanges)/ (b= flange width) produce higher allowable fiber stresses in the top flanges.

Almost all railroad structures are usually simple spans for the following reason:

Usually the maximum negative moment over the support is nearly equal to the positive moment of the simple beam. In some combinations, the continuous beam negative moments may be greater than the simple beam positive moment because of the unfavorable Live Load placement in the spans. Continuity introduces complications and it is questionable if any real saving is realized by its use.

In railroad structures, spacing of the through girders is governed by AREMA specifications for Steel Railway Structures. The spacing should be at least 1/20 of the span or should be adequate to insure that the girders and other structural components provide required clearances for trains, whichever is greater. The requirement of lateral clearance each side of track centerline for curved alignment should be as per latest AREMA specifications. A typical girder inside elevation view is shown in Figure 38.3-4.

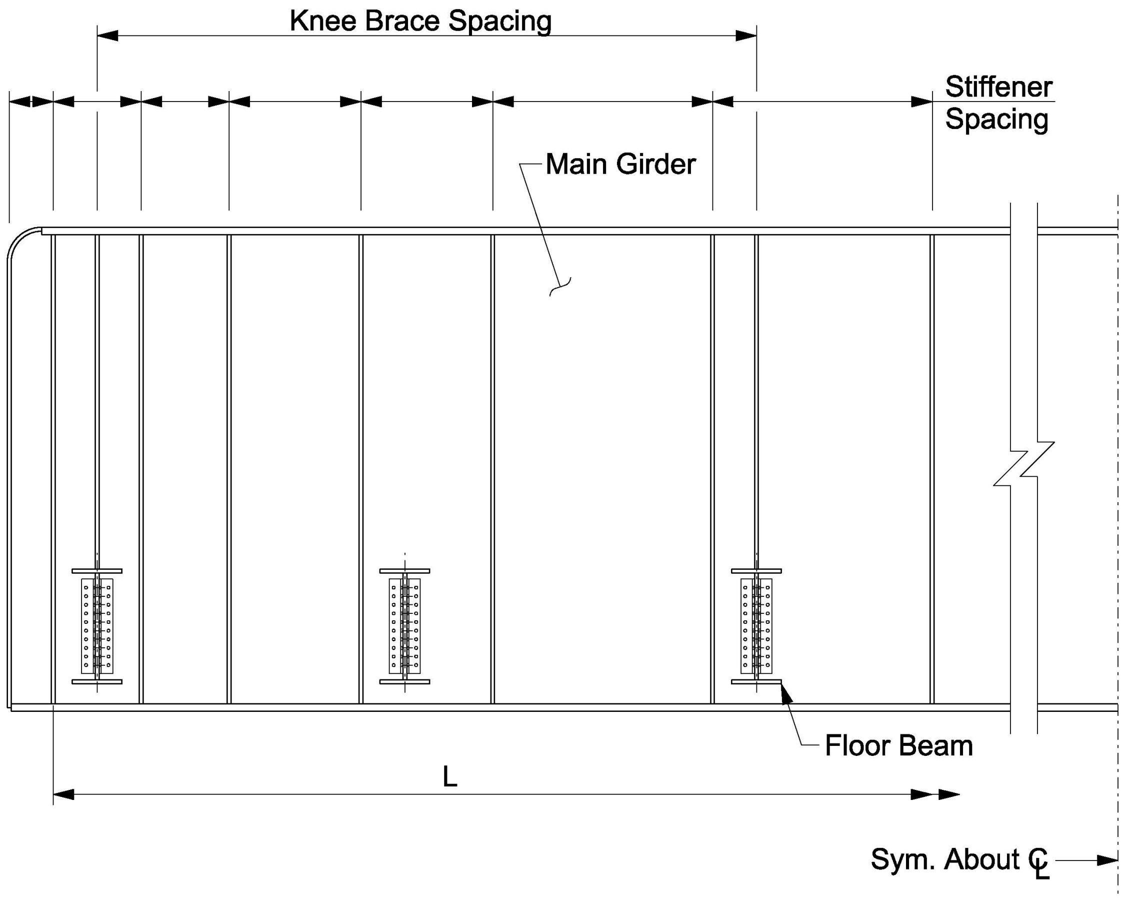

Figure 38.3-4
Typical Through-Girder Inside Elevation

38.3.1.2 Ballast Floor

The superstructure includes the ballast floor, girders and girder bearings to the top of the masonry. The thickness of the ballast floor shall not be less than ½ inches for steel plate or 6 inches for reinforced or prestressed concrete. For concrete floor, thickness is measured from top of bars or cover plate and the reinforcement is usually #4 bars at 6 inches both at top and bottom.

38.3.1.3 Dead Load

Dead Load consists of weight of track rails and fastenings, ballast and ties, weight of waterproofing, ballast plate, floor beams, etc. Most of the load carried by each girder is transmitted to it by the floor beams as concentrated loads. Computations are simpler, however, if the floor beam spacings are ignored and the girder is treated as if it received load from the ballast plate. Moments and shears computed with this assumption are sufficiently accurate for design purposes because of the relatively close spacing of the floor beams. Thus, the dead load on the girder may be considered uniformly distributed.

38.3.1.4 Live Load

The *AREMA Manual* recommends that design be based on Cooper E80 Live Loading as shown in Figure 38.3-5. Heavier Cooper E loadings will result in directly proportional increases in the concentrated and uniform live loadings shown in Figure 38.3-5.

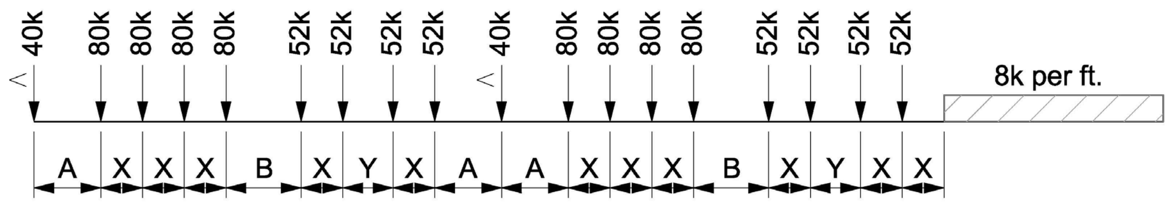

Figure 38.3-5
Cooper E80 Live Loading

X = 5 ft A = 8 ft
Y = 6 ft B = 9 ft

To account for the effect of multiple tracks on a structure, the portions of full live load on the tracks may be taken as:

Number of Tracks	Loading
Two tracks	Full live load
Three tracks	Full live load on two tracks, one-half full live load on the third track
Four tracks	Full live load on two tracks, one-half on one track, and one quarter on the remaining track
More than four tracks	As specified by the Engineer

Table 38.3-1
Live Load vs. Number of Tracks

The selection of the tracks for these loads shall be such as will produce the greatest live load stress in the member.

38.3.1.5 Live Load Distribution

On open-deck structures, ties within a length of 4 feet but not more than three ties may be assumed to support a wheel load. The live load should be considered a series of concentrated loads, however, for the design of beams and girders. No longitudinal distribution of wheel loads shall be assumed.

When two or more longitudinal beams per rail are properly diaphragmed in accordance with *AREMA Manual* Chapter 15, and symmetrically spaced under the rail, they shall be considered as equally loaded.

For ballasted-deck structures, live load distribution is based on the assumption of standard cross ties at least 8 feet long, about 8 inches wide, and spaced not more than 2 feet on centers, with at least 6 inches of ballast under the ties. For deck design, each axle load should be uniformly distributed over a length of 3 feet plus the minimum distance from bottom of tie to top of beams or girders, but not more than 5 feet or the minimum axle spacing of the loading. In the lateral direction, the axle load should be uniformly distributed over a width equal to the length of tie plus the minimum distance from bottom of tie to top of beams or girders.

Transverse steel beams without stringers

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

$$P = \frac{1.15\,AD}{S}$$

Where:

P	=	Load on a beam from one track
A	=	Axle Load
S	=	Axle spacing (ft)
D	=	Effective beam spacing (ft)

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

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

Where:

a	=	Beam span (ft)
H	=	$\frac{nI_b}{ah^3}$

n = Ratio of modulus of elasticity of steel to that of concrete

I_b = Moment of inertia of beam (in^4)

h = Thickness of concrete deck (in)

d = Beam spacing (ft)

For end shear, D = d

The load P shall be applied as two equal concentrated loads on each beam at each rail, equal to P/2. Lateral distribution of such loads shall not be assumed.

The value for “D” should be taken equal to “d” for structures without a concrete deck or for structures where the concrete slab extends over less than 75% of the floor beam.

Where “d” exceeds S, P should be the maximum reaction of the axle loads with the deck between beams acting as a simple span.

For longitudinal steel beams or girders

For ballasted decks supported on longitudinal girders, axle loads should be distributed equally to all girders whose centroids lie within a lateral width equal to length of tie plus twice the minimum distance from bottom of tie to top of girders. Distribution of loads for other conditions shall be determined by a recognized method of analysis.

38.3.1.6 Stability

For spans and towers, stability should be investigated with live load on only one track, the leeward one for structures with more than one track. The live load should be 1200 plf, without impact.

38.3.1.7 Live Load Impact

AREMA Manual Chapter 15 specifies the impact forces to be used and how they are to be applied. Impact forces should be applied vertically and equally at top of each rail. Impact, I, expressed as a percentage of axle loads, is given for open-deck structures by the following equations and modified by a factor determined by the number of tracks to be supported. For ballasted deck structures the percentage to be used shall be 90% of that specified for open deck structures.

For rolling equipment without hammer blow (diesels, electric locomotives, tenders alone, etc.)

For L less than 80 feet

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

For L = 80 feet or more

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

For steam locomotives with hammer blow:

For beam spans, stringers, girders, floorbeams, posts of deck truss spans carrying load from floorbeams only, and floorbeam hangers:

For L less than 100 feet

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

For L = 100 feet or more

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

For truss spans

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

Where:

RE	=	Either 10% of axle load or 20% of the wheel load.
L	=	Length in feet, center to center of supports for stringers, transverse floorbeams without stringers, longitudinal girders and trusses (main members), or length in feet, of the longer adjacent supported stringers, longitudinal beam, girder or truss for impact in floor beams, floor beam hangers, subdiagonals of trusses, transverse girders and viaduct columns.

For members receiving load from more than one track, the impact percentage shall be applied to the live load on the number of tracks designated below.

Load received from two tracks	
For L less than 175 ft	Full impact on two tracks
For L from 175 to 225 ft	Full impact on one track and a percentage of full impact on the other as given by the formula, 450 - 2L
For L greater than 225 ft	Full impact on one track and none on the other
Load received from more than two tracks	
For all values of L	Full impact on any two tracks

Table 38.3-2
Live Load Impact

38.3.1.8 Centrifugal Forces on Railroad Structures

On curves, a centrifugal force corresponding to each axle load should be applied horizontally through a point 6 feet above the top of rail. This distance should be measured in a vertical plane along a line that is perpendicular to and at the midpoint of a radial line joining the tops of the rails. This force should be taken as a percentage, C, of the specified axle load without impact.

$$C = 0.00117 S^2 D$$

Where:

S = Speed (mph)

D = Degree of curve = 5729.65/R

R = Radius of curve (ft)

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

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

38.3.1.9 Lateral Forces From Equipment

For bracing systems or for longitudinal members entirely without a bracing system, the lateral force to provide for the effect of the nosing of equipment, such as locomotives, (in addition to the other lateral forces specified) should be a single moving force equal to 25% of the heaviest axle load. It should be applied at top of rail. This force may act in either lateral direction at any point of the span. On spans supporting multiple tracks, the lateral force from only one track should be used.

The resulting stresses to be considered are axial stresses in members bracing the flanges of stringer, beam and girder spans, axial stresses in the chords of truss spans and in members of cross frames of such spans, and stresses from lateral bending of flanges of longitudinal flexural members having no bracing system. The effects of lateral bending between braced points of flanges, axial forces in flanges, vertical forces and forces transmitted to bearings shall be disregarded.

38.3.1.10 Longitudinal Forces on Railroad Structures

The longitudinal force from trains should be taken as 15% of the live load without impact.

Where the rails are continuous (either welded or bolted joints) across the entire structure from embankment to embankment, the effective longitudinal load shall be taken as L/1200 (where L is the length of the structure in feet) times the load specified above (15% of live load), but the value of L/1200 used shall not exceed 0.80.

Where rails are not continuous, but are interrupted by a moveable span, sliding rail expansion joints or other devices, across the entire structure from embankment to embankment, the effective longitudinal force should be taken as 15% of live load.

The effective longitudinal force should be taken on one track only. It should be distributed to the various components of the supporting structure, taking into account their relative stiffnesses, where appropriate, and the type of bearings.

The effective longitudinal force should be assumed to be applied at base of rail.

38.3.1.11 Wind Loading on Railroad Structures

AREMA Manual Chapter 15 provides the details of wind loading on railroad structures.

The wind load shall be considered as a moving load acting in any horizontal direction. On the train it shall be taken at 300 plf on the one track, applied 8 feet above the top of rail. On the structure it shall be taken at 30 psf on the following surfaces:

- For girder spans, 1.5 times the vertical projection of the span.
- For truss spans, the vertical projection of the span plus any portion of the leeward trusses not shielded by the floor system.
- For viaduct towers and bents, the vertical projections of all columns and tower bracing.

The wind load on girder spans and truss spans, however, shall not be taken at less than 200 plf for the loaded chord or flange, and 150 plf for the unloaded chord or flange.

The wind load on the unloaded structure shall be assumed at 50 psf of surface as defined in the bulleted items above.

38.3.1.12 Loads from Continuous Welded Rails

Section 8.3 of Chapter 15 *AREMA Manual* describes the details of the effect of continuous welded rails. Forces in continuous welded rail may be computed from the following equations:

$$\text{I.F.} = 38WT$$

$$\text{R.F.} = \frac{WDT}{150}$$

Where:

I.F.	=	Internal force in two rails (lb); compression for temperature rise, tension for temperature fall.
R.F.	=	Radial force in two rails, (lb/ft of bridge); acting toward outside of curve for temperature rise, toward inside for temperature fall.
W	=	Weight of one rail (lb/yd)
T	=	Temperature change (°F)
D	=	Degree of curvature

38.3.1.13 Fatigue Stresses on Structures

The major factors governing fatigue strength are the number of stress cycles, the magnitude of the stress range, and the type and location of constructional detail. The number of stress cycles, N, to be considered shall be selected from Table 15-1-7 of Chapter 15 *AREMA Manual*, unless traffic surveys or other considerations indicate otherwise. The selection depends on the span length in the case of longitudinal members, and on the number of tracks in the case of floor beams and hangers.

Formulas for allowable fatigue stresses on structures recommended by AREMA are dependent primarily on the strength of the material, the stress range, number of stress cycles and a stress ratio R.

The stress range, S_R, is defined as the algebraic difference between the maximum and minimum calculated stress due to dead load, live load, impact load and centrifugal force. If live load, impact load and centrifugal force result in compressive stresses and the dead load stress is compression, fatigue need not be considered.

The type and location of the various constructional details are categorized in Table 15-1-9 and illustrated in Figure 15-1-5 *AREMA Manual*. The stress range for other than Fracture Critical Members shall not exceed the allowable fatigue stress range, $S_R$fat, listed in Table 15-1-10.

The stress range for Fracture Critical Members shall not exceed the allowable fatigue stress range $S_R$fat, listed in Table 15-1-10 (see Note 2) *AREMA Manual*.

38.3.1.14 Live Load Deflection

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. For members with perforated cover plates, the effective area shall be used.

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.

38.3.1.15 Loading Combinations on Railroad Structures

Every component of superstructure and substructure should be proportioned to resist all combinations of forces applicable to the type of structure and its site. Members subject to stresses resulting from dead load, live load, impact load and centrifugal force shall be designed so that the maximum stresses do not exceed the basic allowable stresses of Section 1.4, and the stress range does not exceed the allowable fatigue stress range allowed by AREMA specifications.

The basic allowable stresses of Section 1.4 shall be used in the proportioning of members subject to stresses resulting from wind loads only, as specified in *AREMA Manual*, Article 1.3.8.

With the exception of floorbeam hangers, members subject to stresses from other lateral or longitudinal forces, as well as to the dead load, live load, impact and centrifugal forces may be proportioned for 125% of the basic allowable unit stresses, without regard for fatigue. But the section should not be smaller than required with basic unit stresses or allowable fatigue stresses when those lateral or longitudinal forces are not present.

Increase in allowable stress permitted by the previous paragraph shall not be applied to allowable stress in high strength bolts.

38.3.1.16 Basic Allowable Stresses for Structures

Design of steel railroad structures usually is based on a working stress level that is some fraction of the minimum yield strength of the material. This value commonly is 0.55, allowing a safety factor of 1.82 against yield of the steel. The basic allowable stresses for structural steel, rivets, bolts and pins to be used in proportioning the parts of a structure are furnished in Table 15-1-12 in the *AREMA Manual* Chapter 15.

38.3.1.17 Length of Cover Plates and Moment Diagram

The dead load moment diagram is a parabola with mid-ordinate showing the maximum dead load moment. Determination of the exact shape of the envelope for the live load moment involves long and tedious calculations. The procedure consists of dividing the span into parts and finding the maximum moment at each section. The smaller the divisions, the more accurate the shape of the curve and the more involved and tedious the calculations.

Fortunately, a parabola with mid-ordinate equal to the tabular value for maximum moment, Section 1.15, *AREMA Manual* Chapter 15, very nearly encloses the envelope. Therefore the shape of the moment diagram of DL + LL + I is parabolic for all practical purposes. Knowing the maximum ordinate, the designer can compute the other values and draw the moment curve.

The resisting moment diagram can be superimposed upon actual moment diagram described above. The theoretical end of cover plates can be determined from these moment envelopes.

The AREMA specifications require that flange plates shall extend far enough to develop the capacity of the plate beyond the theoretical end. This method of determining the theoretical end of cover plates, on girders proportioned for deflection is not exact, but is acceptable for design purposes.

38.3.1.18 Charpy V-Notch Impact Requirements

Recognizing the need to prevent brittle fracture failures of main load carrying structural components, AREMA specifications have provisions for Charpy V-Notch impact testing and the values for steel other than fracture critical members are tabulated in Table 15-1-2 in *AREMA Manual*.

The design requirements for materials of Fracture Critical Members shall further comply with the Fracture Control Plan specified in *AREMA Manual* Chapter 15, Section 1.14. The Engineer shall designate on the plans which members or member components fall in the category of Fracture Critical Members.

38.3.1.19 Fracture Control Plan for Fracture Critical Members

For purposes of the Fracture Control Plan, Fracture Critical Members or member components (FCM's) are defined as those tension members or tension components of members whose failure would be expected to result in collapse of the structure or inability of the structure to perform its design function.

AREMA specifications have elaborate descriptions of the Fracture Control Plan which has special requirements for the materials, fabrication, welding, inspection and testing of Fracture Critical Members and member components in steel railway structures. The provisions of this plan are to:

- Assign responsibility for designating which steel railway structure members or member components, if any, fall in the category of "Fracture Critical".

- Require that fabrication of FCM or member components be done in plants having personnel, organization, experience, procedures, knowledge and equipment capable of producing quality workmanship.
- Require that all welding inspectors demonstrate their competency to assure that welds in FCM or member components are in compliance with this plan.
- Require that all non-destructive testing personnel demonstrate their competency to assure that tested elements of FCM or member components are in compliance with this plan.
- Specify material toughness values for FCM or member components.
- Supplement recommendations for welding contained elsewhere in AREMA specifications.

Charpy V-Notch (CVN) impact test requirements for steels in FCM's shall be always followed as given in *AREMA Manual* Table 15-1-15. Impact tests shall be in accordance with the CVN tests as governed by ASTM Designation A673 for frequency of testing P (impact). Impact tests shall be required on a set of specimens taken from each end of each plate. Wisconsin currently specifies its steel to Zone 3 when impacts are required on railroad structures. Since Wisconsin Standard Specifications say Zone 2, Zone 3 must be stated on the plans.

38.3.1.20 Waterproofing Railroad Structures

AREMA specifications on waterproofing railroad structures apply to materials and construction methods for an impervious membrane and auxiliary components to protect structures from harmful effects of water. Railroad structures which require waterproofing shall be designed so that they can be waterproofed by the methods and with the materials specified in AREMA specifications. The materials for waterproofing and the methods of application should be such as to insure that the waterproofing will be retained by bond, anchorage or other adequate means, in its original position as applied to the surface to be waterproofed.

The membrane shall consist of one of the following types, as described below.

- Minimum 3/32 inch thick butyl rubber sheeting secured with an approved adhesive.
- Heavy Duty Bituthene or Protecto Wrap M400 may be used.
- Rubberized asphalt with plastic film or 4 feet x 8 feet sheets of preformed board membrane with maximum thickness of ½ inch.

The butyl rubber sheeting, rubber membrane splicing cement and the butyl gum splicing tape shall be in accordance with the requirements for membrane waterproofing as specified in part 29 of Chapter 8 of the *AREMA Manual*. Cement for splicing rubber membrane shall be a self-vulcanizing butyl rubber compound and shall be applied at a minimum rate of 2 gallons/100 square feet.

38.3.2 Substructure

38.3.2.1 Abutments and Retaining Walls

The abutments for railroad structures are essentially bearing walls subject to lateral pressure. The design procedure is similar to that required for a retaining wall. The typical section is shown in Figure 38.3-6.

AREMA Manual Chapter 8, Part 5, governs the requirements for retaining walls. They are essentially the same as AASHTO requirements providing the backfill is of sandy material.

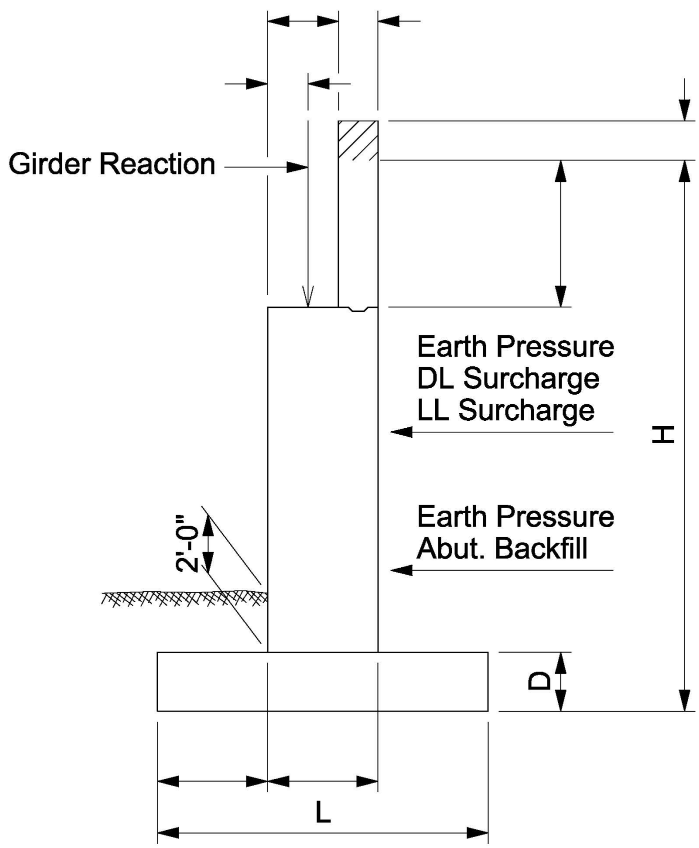

Figure 38.3-6
Typical Abutment

1. Field Survey

   Sufficient information shall be furnished, in the form of a profile and cross sections or a topographic map, to determine the structural requirements. Present grades and alignments of tracks and roads shall be indicated, together with the records of high water, low water and depth of scour, the location of underground utilities, and information concerning any structures that may affect or be affected by the construction.

2. Subsurface exploration

   Specifications provided by *AREMA Manual* Chapter 8 and Part 22 should be followed.

3. Character of backfill

   Backfill is defined as all material behind the wall, whether undisturbed ground or fill, that contributes to the pressure against the wall.

   AREMA Manual Chapter 8, Table 8-5-1 classifies the type of backfill materials for retaining walls.

Type	Description
Type 1	Coarse-grained soil without admixtures of fine soil particles, very free draining (clean sand, gravel or broken stone)
Type 2	Coarse-grained, soil of low permeability due to admixtures of particles of silt size
Type 3	Fine silty sand; granular materials with conspicuous clay content; or residual soil with stones
Type 4	Soft or very soft clay; organic silt; or soft silty clay
Type 5	Medium or stiff clay that may be placed in such a way that a negligible amount of water will enter the spaces between the chunks during floods or heavy rains

Table 38.3-3
Classification of Backfill Material

4. Computation of Earth Pressure

   Values of the unit weight, cohesion and angle of internal friction of the backfill material shall be determined directly by means of soil tests or, if the expense of such tests is not justifiable, refer to Table 8-5-2 in *AREMA Manual* for the soil types defined above. Unless the minimum cohesive strength of backfill material can be evaluated reliably the cohesion shall be neglected and only the internal friction considered.

   When the backfill is assumed to be cohesionless; when the surface of the backfill is or can be assumed to be plane; when there is no surcharge load on the surface of the backfill; or when the surcharge can be converted into an equivalent uniform earth surcharge, Rankine's or Coulomb's formulas may be used under the conditions to which each applies. Formulas and charts given in *AREMA Manual* Chapter 8, Part 5 Commentary and the trial wedge method also presented in this Commentary are both applicable.

5. Computation of Loads Exclusive of Earth Pressure

In the analysis of retaining walls and abutments, due account shall be taken of all superimposed loads carried directly on them, such as building walls, columns, or bridge structures; and of all loads from surcharges caused by railroad tracks, highways, building foundations or other loads supported on the fill behind the walls.

In calculating the surcharge due to track loading, the entire load shall be taken as distributed uniformly over a width equal to the length of the tie. Impact shall not be considered unless the bearings are supported by a structural beam, as in a spill-through abutment.

6. Stability Computation

The resultant force on the base of a wall or abutment shall fall within the middle third if the structure is founded on soil, and within the middle half if founded on rock, masonry or piles. The resultant force on any horizontal section above the base of a solid gravity wall should intersect this section within its middle half. If these requirements are satisfied, safety against overturning need not be investigated.

The factor of safety against sliding at the base of the structure is defined as the sum of the forces at or above base level available to resist horizontal movement of the structure divided by the sum of the forces at or above the same level tending to produce horizontal movement. The numerical value of this factor of safety shall be at least 1.5. If the factor of safety is inadequate, it shall be increased by increasing the width of the base, by the use of a key, by sloping the base upward from heel to toe or by the use of battered piles.

In computing the resistance against sliding, the passive earth pressure of the soil in contact with the face of the wall shall be neglected. The frictional resistance between the wall and a non-cohesive subsoil may be taken as the normal pressure on the base times the coefficient of friction of masonry on soil. For coarse-grained soil without silt, the coefficient of friction may be taken as 0.55; for coarse-grained soil with silt, as 0.45; and for silt as 0.35. For concrete on sound rock the coefficient of friction may be taken as 0.60.

The factor of safety against sliding on other horizontal surfaces below the base shall be investigated and shall not be less than 1.5.

38.3.2.2 Piers

A pier is a structural member of steel, concrete or masonry that supports the vertical loads from the superstructure, as well as the horizontal loads not resisted by the abutments. Also, piers must be capable of resisting forces they may receive directly such as wind loads, floating ice and debris, expanding ice, hydrokinetic pressures and vehicle impact.

The connection between pier and superstructure is usually a fixed or expansion bearing which allows rotation in the longitudinal direction of the superstructure.

The types of piers most frequently used in railroad structures can be classified in one of the following categories:

- Pile Bents
- Solid Single Shaft
- Multi-Column Frames
- Individual Column in Line
- Steel Section

38.3.2.3 Loads on Piers

38.3.2.3.1 Dead Load and Live Loading

Dead load and live load comes from the superstructure on to the pier as girder reactions.

38.3.2.3.2 Longitudinal Force

The longitudinal force from trains shall be taken as 15 percent of the live load without impact. Where the rails are continuous (either welded or bolted joints) across the entire structure from embankment to embankment, the effective longitudinal force shall be taken as L/1200 (where L is the length of the structure in feet) times the force specified above (follow AREMA specifications), but the value of L/1200 shall not exceed 0.80.

The effective longitudinal force shall be assumed to be applied at the top of the supporting structure.

38.3.2.3.3 Stream Flow Pressure

All piers and other portions of structures which are subject to the force of flowing water or drift shall be designed to resist the maximum stresses induced thereby.

38.3.2.3.4 Ice Pressure

The effects of ice pressure, both static and dynamic, shall be accounted for in the design of piers and other portions of the structure where, in the judgment of the engineer, conditions so warrant. The values of effective ice pressure furnished in AASHTO specifications may be used as a guide.

38.3.2.3.5 Buoyancy

Buoyancy shall be considered as it affects the design of the substructure including piling.

38.3.2.3.6 Wind Load on Structure

The wind load acting on the structure shall be assumed as 45 psf on the vertical projection of the structure, applied at the center of gravity of the vertical projection. The wind load shall be assumed to act horizontally, in a direction perpendicular to the centerline of the track.

38.3.2.3.7 Wind Load on Live Load

A moving load of 300 plf on the train shall be applied 8 feet above the top of the rail horizontally in a direction perpendicular to the centerline of the track.

38.3.2.3.8 Centrifugal Force

On curves, a centrifugal force corresponding to each axle load shall be applied horizontally through a point 6 feet above the top of rail measured along a line perpendicular to the line joining the tops of the rails and equidistant from them. This force shall be the percentage of the live load computed from the formulas in 38.3.1.8.

38.3.2.3.9 Rib Shortening, Shrinkage, Temperature and Settlement of Supports

The structure shall be designed to resist the forces caused by rib shortening, shrinkage, temperature rise and/or drop and the anticipated settlement of supports. The following values for range of temperature and coefficient of thermal expansion apply to Wisconsin steel structures.

Temperature range	90°F
Coefficient of thermal expansion	0.0000065/°F

38.3.2.3.10 Loading Combinations

The following groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned for the group of loads that produce the most critical design condition.

<u>Service Load Design</u>

The group loading combinations for Service Load Design are as follows:

Load Case	Load Combinations	Allowable Percentage of Basic Unit Stress
Group I	D + L + I + CF + E + B + SF	100
Group II	D + E + B + SF + W	125
Group III	Group I + 0.5W + WL + LF + F	125
Group IV	Group I + OF	125
Group V	Group II + OF	140
Group VI	Group III + OF	140
Group VII	Group I + ICE	140
Group VIII	Group II + ICE	150
No increase in allowable unit stresses shall be permitted for members or connections carrying wind load only.		

Table 38.3-4
Service Load Design

Load Factor Design

The group loading combinations for Load Factor Design are as follows:

Group I	1.4 (D + 5/3 (L+I) + CF + E + B + SF)
Group IA	1.8 (D + L + I + CF + E + B + SF)
Group II	1.4 (D + E + B + SF + W)
Group III	1.4 (D + L + I + CF + E + B + SF + 0.5W + WL + LF + F)
Group IV	1.4 (D + L + I + CF + E + B + SF + OF)
Group V	Group II + 1.4 (OF)
Group VI	Group III + 1.4 (OF)
Group VII	1.0 (D + E + B + EQ)
Group VIII	1.4 (D + L + I + E + B + SF + ICE)
Group IX	1.2 (D + E + B + SF + W + ICE)
The load factors given are only intended for designing structural members by the load factor concept. The actual loads should not be increased by these factors when designing for foundations (soil pressure, pile loads, etc.). The load factors are not intended to be used when checking for foundation stability (safety factors against overturning, sliding, etc.) of a structure.	

Table 38.3-5
Load Factor Design

38.3.2.4 Pier Protection for Overpass Structures

Pier protection should be placed according to the railroad company involved, as they each have different requirements. For minimum requirements, refer to Standard for Highway Over Railroad Design Requirements. Check with the railroad company to determine if they want to extend crash wall beyond columns. Usually they do not.

Crash walls are not required on team tracks and spur tracks as these are for storage or loading and unloading on secondary lines.

Temporary sheet piling may be required by the railroad company during pier and footing construction. All sheet pilings have to be removed after completion of overpass structures. Refer to Standard Highway Over Railroad Design Requirements.

On rehabilitated or widened structures, past practice is to extend the existing protection. If the structure does not have any crashwall protection, past practice is to widen the pier in line with the existing as-built pier provided there is no reduction in horizontal clearance; even though it does not meet current standard clearance criteria.

38.3.2.5 Pier Protection Systems at Spans Over Navigable Streams

38.3.2.5.1 General

AREMA Manual Chapter 8, Part 23, covers the design, construction, maintenance and inspection of protective systems for railway piers located in and adjacent to channels of navigable waterways.

The purpose of the protective systems is to protect supporting piers of railway structures from damage caused by accidental collision from floating vessels. Such protection should be designed to eliminate or reduce the impact energy transmitted to the pier from the vessel, either by redirection of the force or by absorption, or dissipation of the energy, to non-destructive levels.

The size and type of vessel to be chosen as a basis for design of the pier protection should reflect the maximum vessel tonnage, type of cargo and velocity reasonably to be expected for the specific facility involved.

38.3.2.5.2 Types of Construction

The type of construction to be chosen for the protective system should be based on the physical site conditions and the amount of energy to be absorbed or deflected, as well as the size and ability of the pier itself to absorb or resist the impact.

Some of the more common types of construction are as follows:

- Integral piers - Where the pier is considered to be stable enough to absorb the impact of floating vessels, it may be necessary to attach cushioning devices to the surfaces of the pier in the areas of expected impact to reduce localized damage such as spalling of concrete surfaces and exposure of reinforcing steel, or disintegration of masonry jointing.

- Dolphins - Where depth of water and other conditions are suitable, the driving of pile clusters may be considered. Such clusters have the piles lashed together with cable to promote integral action. The clusters should be flexible to be effective in absorbing impact through deflection.

- Cellular dolphins - May be filled with concrete, loose materials or materials suitable for grouting.

- Floating shear booms - Where the depth of water or other conditions precludes the consideration of dolphins or integral pier protection, floating sheer booms may be used. These are suitably shaped and positioned to protect the pier and are anchored to allow deflection and absorption of energy. Anchorage systems should allow for fluctuations in water level due to stream flow or tidal action.

- Hydraulic devices - Such as suspended cylinders engaging a mass of water to absorb or deflect the impact energy may be used under certain conditions of water depth or intensity of impact. Such cylinders may be suspended from independent caissons, booms projecting from the pier or other supports. Such devices are customarily most effective in locations subject to little fluctuations of water levels.

- Fender systems - Constructed using piling with horizontal wales, is a common means of protection where water depth is not excessive and severe impacts are not anticipated.

- Other types of various protective systems have been successfully used and may be considered by the Engineer. Criteria for the design of protective systems cannot be specified to be applicable to all situations. Investigation of local conditions is required in each case, the results of which may then be used to apply engineering judgment to arrive at a reasonable solution.

38.4 Overpass Structures

Highway overpass structures are placed when the incidences of train and vehicle crossings exceeds certain values specified in the *Facilities Development Manual* (*FDM*). The separation provides a safer environment for both trains and vehicles.

In preparing the preliminary plan which will be sent to the railroad company for review and approval several items of data must be determined.

- Track Profile - In order to maintain clearances under existing structures when the track was upgraded with new ballast, the railroad company did not change the track elevation under the structure causing a sag in the gradeline. The track profile would be raised with a new structure and the vertical clearance for the structure should consider this.

- Drainage - Hydraulic analysis is required if any excess drainage will occur along the rail line or into existing drainage structures. Deck drains shall not discharge onto railroad track beds.

- Horizontal Clearances - The railroad system is expanding just as the highway system. Contact the railroad company for information about adding another track or adding a switching yard under the proposed structure.

- Safety Barrier – The Commissioner of Railroads has determined that the Transportation Agency has authority to determine safety barriers according to their standards. The railroad overpass parapets should be designed the same as highway grade separation structures using solid parapets (Type "SS" or appropriate) and chain link fencing when sidewalks are present.

38.4.1 Preliminary Plan Preparation

Standard for Highway over Railroad Design Requirements shows the minimum dimensions for clearances and footing depths. These should be shown on the Preliminary Plan along with the following data.

- Milepost and Direction - Show the railroad milepost and the increasing direction.

- Structure Location - Show location of structure relative to railroad right of way. (Alternative is to submit Roadway Plan).

- Footings - Show all footing depths. Minimum footing depth requirements are shown on the Standard for Highway over Railroad Design Requirements.

- Drainage Ditches - Show ditches and direction of flow.

- Utilities - Show all utilities that are near structure footings and proposed relocation is required.

- Crash Protection – See Standard for Highway over Railroad Design Requirements for crash protection requirements. On a structure widening a crashwall shall be added if the multi-columned pier is equal to or less than 25 feet from centerline of track.
- Shoring – If shoring is required, use a General Note to indicate the location and limit.
- Limits of Railroad Right-of-Way – The locations are for reference only and need not be dimensioned.

38.4.2 Final Plans

The Final Plans must show all the approved Preliminary Plan data and be signed and/or sealed by a Registered Engineer.

38.4.3 Shoring

Railroad companies are particularly concerned about their track elevations. It is therefore very important that shoring is used where required and that it maintains track integrity.

38.4.4 Horizontal and Vertical Clearances

38.4.4.1 Horizontal Clearance

The distance from the centerline of track to the face of back slopes at the top of rail must not be greater than 20’-0” since federal funds are not eligible to participate in costs for providing greater distances unless required by site conditions. Minimum clearances to substructure units are determined based on site conditions and the character of the railroad line. Consideration must be given to the need for future tracks. Site specific track drainage requirements and possible need for an off-track roadway must also be considered.

38.4.4.2 Vertical Clearance

Section 192.31, Wisconsin Statutes requires 23’-0” vertical clearance above top of rail (ATR) for new construction or reconstruction, unless the Office of the Commissioner of Railroads approves less clearance. As a result, early coordination with the Railroads and Harbors Section is required.

Double stack containers at 20’-2” ATR are the highest equipment moving in restricted interchange on rail lines which have granted specific approval for their use. Allowing for tolerance, this equipment would not require more than 21’-0” ATR clearance. Railroad companies desire greater clearance for maintenance purposes and to provide allowance for possible future increases in equipment height.

38.4.4.3 Compensation for Curvature

Where a horizontal clearance obstruction is within 80 feet of curved track AREMA specifications call for lateral clearance increases as stated in *AREMA Manual* Chapter 28, Table 28-1-1.

38.4.4.4 Constructability

The minimum clearances discussed are to finished permanent work. Most railroad companies desire minimum temporary construction clearances to forms, falsework or track protection of 12’- 0” horizontal and 21’-0” vertical. The horizontal clearance provides room for a worker to walk along the side of a train and more than ample room for a train worker who may be required to ride on the side of a 10’-8” wide railroad car. Where piers are to be located close to tracks the type of footing to be used must be given careful consideration for constructability. The depth of falsework and forms for slab decks may also be limited by temporary vertical clearance requirements.