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# 16

# Highway Truss Bridges

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# **16.1 Truss Configurations**

# 16.1.1 Historical

During the 1800s, truss geometries proliferated. *The Historic American Engineering Record* illustrates 32 separate bridge truss geometries in its 1976 print shown in Figure 16.1 [1]. These range from the very short King Post and Queen Post Trusses and Waddell "A" trusses to very complex indeterminate systems, including the Town Lattice and Burr Arch truss. Over a period of years following Squire Whipple's breakthrough treatise on the analysis of trusses as pin-connected assemblies, i.e., two force members, a number of the more complex and less functional truss types gradually disappeared and the well-known Pratt, Howe, Baltimore, Pennsylvania, K truss and Warren configurations came into dominance. By the mid-20th century, the Warren truss with verticals was a dominant form of truss configuration for highway bridges, and the Warren and K trusses were dominant in railroad bridges. *The Historic American Engineering Record* indicates that the Warren truss with verticals, as this provided a very convenient way to brace compression chords, reduce stringer lengths, and frame sway frames into the relatively simple geometry of the vertical members.



FIGURE 16.1 Historic trusses.

#### 16.1.2 Modern

Few single span trusses are used as highway bridges today, although they are still used for railroad bridges. Modern highway trusses are usually either continuous or cantilever bridges and are typically Warren trusses with or without verticals. Some typical configurations are shown in Figure 16.2.

Throughout the 1980s and 1990s, the Warren truss without verticals has resurfaced as a more aesthetically pleasing truss configuration, especially in the parallel chord configuration, and this has led to a significant simplification in truss detailing, because sway frames are typically omitted in this form of truss, except for portals. The Warren truss without verticals received a great deal of use on the Japanese Railroad System and, more recently, in U.S. highway practice as exemplified in the Cooper River Bridge near Charleston, South Carolina, and the Kanawha River Bridge near Charleston, West Virginia; such a bridge configuration is shown in Figure 16.3 as it was considered one option for the Second Blue Water Bridge (281 m main span) between Port Huron, Michigan, and Point Edward, Ontario, Canada.

The truss bridge behaves much like a closed box structure when it has four planes capable of resisting shear and end portals sufficient to transmit shear back into vertical loads at the bearings. Given the need for a box configuration to resist vertical and lateral loads, it is possible that the configuration could be either rectangular, i.e., four-sided, or triangular, if that geometry is able to





FIGURE 16.2 Typical modern highway truss configuration.



FIGURE 16.3 Second Blue Water Bridge — parallel chord truss study option.

accommodate the roadway clearances. Issues of redundancy should be addressed, either by supplementary load paths, e.g., prestressing, or by sufficiently improved material properties, primarily toughness, to make a triangular configuration acceptable to owners, but it is certainly within the technical realm of reason.

# 16.2 Typical Components, Nomenclature, and Materials

#### Components and Nomenclature

The truss bridge is usually characterized by a plethora of bracing and wind-carrying members in addition to those members seen in front elevation. Typical members of a simple single span through truss are identified in Figure 16.4, taken from Ref. [2].



FIGURE 16.4 Typical truss members.

The lateral members in the planes of the top and bottom chords resist wind loads and brace the compression chords. Sway frames are thought to square the truss and increase its torsional rigidity. End portals carry torsional loads resulting from uneven vertical loads and wind loads into the bearings.

It is the visual impact of the various members, especially bracing members, which contribute to aesthetic opposition to many truss designs. However, if unforeseen events cause damage to a main truss member, these bracing members can serve as additional load paths to carry member load around a damaged area.

#### Truss Members

Some of the cross sections used as modern truss members are shown in Figure 16.5. Truss members have evolved from rods, bars, and eyebars to box and H-shaped members. Generally speaking, the box members are more structurally efficient and resist the tendency for wind-induced vibration better than H-shapes, whereas H-shapes are perceived as being more economical in terms of fabrication for a given tonnage of steel, generally easier to connect to the gusset plates because of open access to bolts, and easier to maintain because all surfaces are accessible for painting. The use of weathering steel offsets these advantages.

Even in the late 1990s, box members are widely used and, in some cases, the apparent efficiency of the H-shape is offset by the need to make the members aerodynamically stable. The choice is clearly project specific, although the H-shaped sections have a relatively clear advantage in the case of tension members because they are easier to connect to gusset plates and easier to paint as indicated above, without the stability design requirements needed for compression members. They are, however, more susceptible to wind-induced vibrations than box shapes. Box shapes have an advantage in the case of compression members because they usually have lower slenderness ratios about the weak axis than a corresponding H-shaped member.

The sealing of box shapes to prevent corrosion on the inside of the members has been approached from many directions. In some cases, box shapes may be fully welded, except at access locations at



FIGURE 16.5 Cross sections of modern truss members.

the ends used to facilitate connection to gusset plates. Sealing of box members has met with mixed success. In some instances, even box members which have been welded on all four sides and have had welded internal squaring and sealing diaphragms have been observed to collect moisture. The issue is that the member need not be simply watertight to prevent the infiltration of water in the liquid form, it must also be airtight to prevent the natural tendency for the member to "breath" when subjected to temperature fluctuations, which tends to draw air into the member through even the smallest cracks or pinholes in the sealing system. This air invariably contains moisture and can be a recurring source of condensation, leading to a collection of water within the member. In some cases, box members have been equipped with drainage holes, even though nominally sealed, in order to allow this condensate to escape. In some instances, box members have been sealed and pressurized with an inert gas, typically nitrogen, in order to establish that adequate seals have been developed, as well as to eliminate oxygen from the inside of the member, thus discouraging corrosion. Box members have been built with valve stems in order to monitor the internal pressure, as well as to purge and refill the inert gas corrosion protection. Various types of caulking have been used to try to seal bolted joints with mixed success.

Due to the increased interest in redundancy of truss bridges, stitch-bolted members have been used in some cases. Because a bolt does not completely fill a hole, this does leave a path for water ingress, making adequate ventilation and drainage of the member important.

The future of truss member configurations is somewhat dependent on the evolution of new materials as discussed below.

#### Materials

Early trusses were made of timber. Over a period of time, the combination of timber compression members and wrought iron tension members was evolved and eventually the timber components were replaced by cast iron. After the construction of the Eads Bridge in St. Louis, steel became more widely used and remains today the predominant, and almost exclusive, material for truss construction.

As truss bridges reached longer and longer spans, they became the natural platform for the introduction of new steels, including the silicon steels in the earlier part of this century, weathering steels, copper-bearing steels, low-alloy steels, and even the high-yield-strength quenched and tempered alloy steels, such as ASTM A514/A517. An earlier version of weathering-type steels was used in trusses during the first quarter of the 20th century and was little recognized for their weathering capabilities until relatively recently.

By the current era (1998), 345 MPa yield steel dominated, with some use of higher strength materials, especially for very long-span bridges. The recent high-performance steel (HPS) initiative on the part of the Federal Highway Administration, the steel industry, and the U.S. Navy has led to the development of steels which hold the promise for relatively high strength, e.g., 485 MPa yield and higher, relatively favorable yield-to-tensile ratios on the order of 0.85, extraordinary toughness which could eliminate fracture as a consideration and with it fracture-critical members designation, reduced interpass temperature controls, and reduced preheat. This new material not only holds the promise for increased efficiency and cost-effectiveness using conventional box and I-shaped members in truss design, but may also lead to the use of tubular members and cast joints. This sounds like a return to the old Phoenix bridge system, but the increased efficiency of new materials, as well as the advances in the casting industry, may make field welding of tubular shapes to nodes possible and efficient. The use of high-performance concrete (HPC) truss members may evolve in which concrete serves as a composite stiffener to relatively thin steel plates in compression members.

Advanced composite materials may lead to further truss efficiencies. A pultruded shape as a compression member, butting into a metallic, concrete, or composite node may be a near-term possibility. Composite tension members may require a bonding agent, e.g., glue, before it becomes possible to make a fatigue-resistant joint. The problems associated with mechanical fastener-type connection will probably be solved sooner or later facilitating the use of advanced composites.

## 16.3 Methods of Analysis

#### 16.3.1 Two-Force Member Methods — Pin-Connected Truss

In the 1840s, a method of analyzing trusses as pin-connected assemblages was developed and is still in wide use today. This method is based on assuming that the truss joints are frictionless pins. This assumption means that as long as loads are applied to the joints and not along the member length, the only bending is caused by self-weight. Thus, the major force in the member is assumed to act along its length. This is often called a "two-force member." The two forces are the axial load at each end of the member.

Throughout the 19th century and even into the early part of the 20th century, it was common to use physical pins in truss joints in order to facilitate the interconnection of components of members, and also to replicate the mathematical assumptions. As a truss deflects under loads, the joints rotate through what are typically very small angles. If the pins truly were frictionless, the truss members would rotate relative to each other and no end moments would be developed on the members. The physical pins never really were friction-free, so some moments developed at the ends in truss members and these were typically regarded as secondary forces. When pin-ended construction gave way to riveted joints and then to bolted or welded joints, the truss joints were detailed so that the working lines of the members intersected either at a common point, so as to reduce eccentricities or to utilize eccentricities to compensate for the bending caused by the dead weight of the members. In either event, it was widely regarded that the pin-connected analysis model was applicable. As will be discussed later, as long as a bridge is properly cambered, it often is an accurate analysis tool.

Two variations of the pin-connected truss model are in common usage; the method of joints and the method of sections. Each of these are illustrated below.

#### 16.3.1.1 Method of Joints

As the name implies, the method of joints is based on analysis of free-body diagrams of each of the truss joints. As long as the truss is determinate, there will be enough joints and equations of equilibrium to find the force in all the members. Consider the simple example shown in Figure 16.6. This six-panel truss supports a load P at Joint L3. By taking the summation of the moments about each end of the bridge, it is possible to determine that the left-hand reaction is  $\frac{2}{3}P$  and the right-hand reaction is  $\frac{1}{3}P$ .

Isolating Joint L0, it can be seen that there are two unknowns, the force in Member L0-U1 and the force in Member L0-L1. For this small truss, and as typically illustrated in most textbooks, the truss is assumed to be in a horizontal position, so that it is convenient to take one reference axis through Member L0-L1 and establish an orthagonal axis through L0. These are commonly called the horizontal and vertical axes. The forces parallel to each must be in equilibrium. In this case, that means that the vertical component of the force in Member L0-U1 is equal the reaction RL. By considering the forces in the horizontal direction, the force in Member L0-L1 is equal to the horizontal component of the force in Member L0-U1. Thus, all of the member forces at L0 can be determined.

If we proceed to Joint L1, at which there is no applied load, it is clear that vertical equilibrium of the joint requires that the force in L1-U1 be equal to 0, and that the force in L1-L2 be equal to L0-L1.

Proceeding to Joint U1, it can be seen that, although four members frame into that joint, the force in two of the members are now known, and the force in the other two members can be found from the equation of equilibrium of forces along the two axes.

The analysis continues in this way from joint to joint.

#### 16.3.1.2 Method of Sections

The method of sections proceeds by identifying free-body diagrams which contain only two unknowns, so that equilibrium of the sum of the moment about one joint and the equilibrium of the sum of the shears through a panel are sufficient to determine the two unknown truss forces. Consider Section AA in Figure 16.7 which shows a portion of the same truss shown in Figure 16.6. If we consider the free-body diagram to the left of Section AA, it is clear that the shear in the panel is equal to the reaction  $R_L$  and that this can be reacted only by the force L0-U1. Similarly, since the section and hence the free-body diagram is taken just to the left of the Joints L1 and U1, summing moments about Joint U1 or, more accurately, the end of Member L0-U1 an infinitesimally small distance to the left of the section line enables us to compute the force in L0-U1.

If we consider Section BB, it can be seen that the sum of the moments about the lower chord joint enables us to find the force in the top chord, and the shear in this panel enables us to find the force in the diagonal directly.

The analysis then proceeds from section to section along the truss. As a practical matter, a combination of the method of section and the method of joints usually results in the most expeditious calculations.

#### 16.3.1.3 Influence Lines for a Truss

An influence line is a graphical presentation of the force in a truss member as the load moves along the length of the structure.

Influence lines for forces in the members are usually found by applying a unit load at each of the affected chord joints. This information is then shown pictorially, as indicated in Figure 16.8, which shows the influence line for a Chord Force U1-U2 (or U2-U3) and Diagonal Force L2-U3. If the truss is statically determinate, the influence line is a series of straight line segments. Since panel point loading is usually used in a truss, the influence lines for diagonals typically pass through a truss panel, as shown in Figure 16.8. If the truss is statically indeterminate, then the influence lines will be a series of chords to a curve, not a straight line.



Summing Moments About Right End:

4PL = 6R, L







U2

U1L2 = LOU1 = 0.868 P

<u>From  $\Sigma V = 0$ </u>:

 $\frac{1.2 LOU1}{1.562} = R_L = \frac{2}{3}P$ 

LOU1 = 0.868 P

From  $\Sigma V = 0$ :

L1U1 = O

<u>From  $\Sigma V = 0$ </u>:

1.2 LOU1 1.562 = 1.2 U1L2 1.562

 $\frac{\text{From }\Sigma V = 0}{L2U2 - 0}$ 

Determined So Far: L2U2 - 0

U1L2 = 0.858 P

L1L2 = 0.555 P

L2

From 
$$\Sigma V = 0$$

 $\frac{1.2\,U1L2}{1.562} + \frac{1.2\,U3L2}{1.562} = P$ 

U3L2 = 0.434 P

FIGURE 16.6 Method of joints.

Summing Moments About Left End:

2PL = 6R\_L

 $R_R = \frac{1}{3}P$ 

<u>From  $\Sigma H = O$ </u>:

LOU1 1.562 = LOL1

LOL1 = 0.555 P

 $\frac{\text{From }\Sigma H = 0}{\text{L1L2} - \text{L0L1} - 0.555 P}$ 

<u>From  $\Sigma H = 0$ </u>:

 $U1U2 = \frac{LOU1}{1.562} + \frac{U1L2}{1.562} = \frac{2X0.968 P}{1.562}$ 

U1U2 = 1.111 P

<u>From  $\Sigma H = 0$ </u>: *U2U3 - U1U2 - 1.111 P* 

*L2L3 + L1L2 + U1L2 - U3L2* 1.562 - 1.562

From  $\Sigma H = 0$ 

L2L3 - 0.833 P



FIGURE 16.7 Method of sections.



FIGURE 16.8 Influence lines for forces in one chord and one diagonal.

#### 16.3.2 Computer Methods

The method of joints and the method of sections identified above appear to be very simple as long as the geometry of the truss is also simple and the structure is statically determinate. This is particularly true if one or both chords are horizontal. On most modern trusses, the span is sufficiently long that the change in vertical geometry can be significant. In fact, most larger trusses are on vertical curves if they cross a waterway. The chord joints are usually parallel to the deck profile. Thus, in many practical truss bridges, one or both truss chords is a series of chord segments representing a parabolic curve over at least part of the length of the bridge. This significantly complicates the geometry with respect to the use of either the method of joints or the method of sections. It does not negate the use of either of these methods, but certainly makes them less attractive.

There are many software packages for computers which permit the analysis of trusses, as can be done typically as either the pin-connected assemblage or as a frame with moment-resisting joints.

If the bridge is determinate in the plane of a truss, and if the truss is analyzed with two force members, then the cross-sectional area of the members does not affect the analysis. Assuming unit area for all members will give the proper forces, but not necessarily the proper displacements. If the truss is indeterminate in a plane, then it will be necessary to use realistic areas for the truss members and may be important to include the camber of the members in order to get realistic results in some cases. This will be true of the so-called "geometric case" which is usually taken as the state of the bridge under all dead load, at which time it is supposed to have the proper grades and profile. An analysis for a subsequent load, such as unit loads for the assembly of influence lines, or a transient load, does not require inclusion of the member camber. In fact, inclusion of the camber for other than the loads acting in the geometric condition would yield erroneous results for the indeterminate truss.

Where software contains the ability to put in a unit length change within a member and an analysis similar to this will be required to account properly for camber of the members, then it is often found efficient to calculate the influence lines for truss members using the Mueller–Bresslau principle as found in any text on structural mechanics.

When a truss is analyzed as a three-dimensional assemblage with moment-resisting joints, then the method of camber becomes even more important. It is common practice for some of the members to be cambered to a "no-load position" and in order for these members to have no load in them as analyzed, all the other members in the truss will have to be properly cambered in the computer model. With the usual fabrication techniques and adequate care for camber in both primary and secondary member (which may have no camber), a three-dimensional computer analysis of a roadway truss will typically result in determining truss member forces which are very close to those obtained by the pin-connected truss analogy. The secondary stresses from joint rotation resulting from transient loads will be determined directly from the computer analysis.

# 16.4 Floor Systems and Framing Details

#### 16.4.1 Conventional Deck Systems Not Integral with Truss Chords

Initially, floor system framing was intended to be as structurally simple as possible. In the past, floor beams were often hung from truss pins with yokes, the simple-span stringers framing between floor beams often supported by saddle brackets on floor beam webs. As time went on, the advantages of continuous stringers, particularly in highway bridges, became very evident, and framing involving stringers-over-floor-beams developed, as did improved details for framing simply supported stringers between floor beams. Composite design of stringers and/or stringers and floor beams continued to add strength, stiffness, and robustness to trusses, while simultaneously eliminating many of the







sources of uncontrolled drainage, and hence, corrosion, which have been the perceived source of excessive maintenance in trusses. Currently, floor beams are either vertical or set normal to roadway grade, and stringers are usually normal to crown and parallel to grade. If they are vertical, some sort of beveled fill will be necessary between the floor beam and the stringers. A typical through-truss cross-section is shown in Figure 16.9.

Most modern truss designs continue to use concrete decks, as well as filled grid, or grid and concrete composite systems as efficient durable decks. Relatively little use has been made of orthotropic decks in conjunction with original design (as opposed to rehabilitation) of trusses in the United States but this is certainly a feasible alternative. The use of newer lightweight deck systems, such as the proprietary Aluma-Deck or possibly advanced composite orthotropic deck systems can lead to further reduction in weight and, hence, savings in a competitive environment, as well as holding the potential for significantly reduced maintenance in future trusses.

#### 16.4.2 Decks Integral with Truss Chords

So far, deck systems have almost always been designed to be structurally separate from the main supporting truss systems. As the need for efficiency and reduced cost, as well as increased redundancy, continues, a possible merging of the deck and truss system is a technical possibility. An orthotropic deck has been used as part of the bottom or top chord on some foreign bridges. Redundancy issues should be thoroughly considered as more traditional load paths are reduced. The available computer capabilities allow modeling of damage scenarios and the emerging knowledge on the computation of reliability indexes for damaged structures can provide designs with high levels of confidence, but such sophisticated calculations will have to be justified by cost savings and/or other benefits. Merging chords and deck has the potential to eliminate more joints within the deck system, perhaps at the expense of accommodating certain differential temperature features. Generally, as in all types of bridge structures, the elimination of joints is perceived as a favorable development. If the use of orthotropic decks as part of the chord system, and the lateral system for that matter, were to evolve, designers would have to consider the possibility of using either reinforced or prestressed concrete in a similar manner. This would, of course, tend to lead toward loading the chord at other than the panel points, but this situation has been handled in the past where, in some situations, deck chord members directly supported the roadway deck over their full length, not simply at the panel points.

### 16.5 Special Details

#### 16.5.1 Hangers and Dummy Chords for Cantilever Bridges

The cantilevered truss has been used effectively on long-span structures since the Firth of Forth Bridge was built in Scotland in the late 1800s. This structural system was developed to provide most of the economy of continuous construction, as well as the longer spans possible with continuity, while simultaneously providing the simplicity of a statically determinate structural system. Consider the system shown in Figure 16.10. Figure 16.10a shows what appears to be a Warren truss configuration for a three-span continuous unit. The parallel diagonal configuration shown in the detail in Figure 16.10a is an indication that the framing system for the standard Warren truss has been interrupted. The statical system for the cantilever truss, indicated by the parallel diagonals, is shown in Figure 16.10b. The continuity has been interrupted by providing two points along the structure where the chords carry no axial force, resulting in a "shear only" connection. This is, by definition, a structural hinge. The unit between the two hinges is commonly referred to as a "suspended span." The remaining portions of the structure are called the "cantilever arms" and the "anchor spans," as indicated in Figure 16.10a. The mechanism for supporting suspended span is shown in concept in Figure 16.10c, which indicates that two chords are missing and hinges have been placed in the strap, or hanger, carrying the load of the suspended span into the anchor arm. The configuration with the link and two hinges allows the portions of the structure to expand and contract relative to each other.

In practice, the unnecessary top and bottom chords are added to the structure to allay public concerns, and are articulated in a manner which prevents them from carrying any axial load. These elements are typically called "false chords, or "dummy chords." A typical top chord joint at the hanger point is shown in Figure 16.11. Figure 16.11a is a plan view of the top chord element, and the corresponding elevation view is shown in Figure 16.11b. The false chord in this case is supported by the anchor arm, utilizing a pin. The false chord is slotted, so that it may move back and forth with expansion and contraction without carrying any load. It simply moves back and forth relative to the pin in the slot provided. The pin carries the vertical weight of the member to the top chord joint. Also shown in Figure 16.11b, and extending further into Figure 16.11c, are details of the hanger assembly and the top pin of the pins in the hanger used to allow it to swing back and forth. Hangers are potentially fracture-critical members, as a failure of this member would almost certainly



FIGURE 16.10 Cantilever suspension.

result in a collapse of at least the suspended span portion of the structure. These members are usually built of multiple components to add redundancy. The particular assembly shown has multiple plates bolted together to compensate for the hole occupied by the pin. In recent years, many truss bridges have been retrofitted with redundancy-adding assemblies usually consisting of rods or cables parallel to the hanger and attaching to the top and bottom chord. These assemblies are intended to pick up the load if the hanger or pin were to fail. Some of the details for the hanger pin are also shown in Figure 16.11c.

The corresponding portions of the structure at the bottom chord are shown in Figure 16.12. An elevation view of the lower chord joint is shown in Figure 16.12a, and a partial plan view is shown in Figure 16.12b. The concepts are very similar to those utilized in the upper joints, in that there are pins and slots to allow the false chord to move without picking up the axial load and pins and gusset plates to transfer the load from the suspended span into the hanger.

After completion of erection of the anchor span and the cantilever arm, the suspended span may be erected component by component, often referred to as "stick erection," or the entire suspended span may be assembled off site and hoisted into position until it can be brought into bearing at the hanger pins. If stick erection is used, the bridge will sag toward the middle as the cantilevers reach midspan. The bridge will be in the sag position because, once assembled to midspan, the cantilevers will be much shorter than they are at the midspan closure. It will thus be necessary to raise or lower portions of the bridge in order to get the closure members in and to transfer loads to the intended statical system. Also during this time, the false chords have to carry loads to support the cantilevering. With this type of erection, the false chords may be temporarily fixed, and one or both of the chords may have mechanical or hydraulic jacks for transferring load and for repositioning the two cantilevers for closure. Provisions for this type of assembly are often made in the false chord, at least to the point of being certain that the required space is available for jacks of sufficient capacity and that bearing plates to transmit the load are either in place or can be added by the contractor. A



FIGURE 16.11 (a) Top false chord details — plan view; (b) top false chord details — elevation view; (c) hanger details.

typical detail for providing for jack assemblies is shown in Figure 16.13, which indicates how jacks would fit in the bottom false chord shown in Figure 16.12, and bear against the rest of the structure, so as to swing the cantilevered portion of the suspended span upward to facilitate closure.

#### 16.5.2 Bearings

From the viewpoint of bearings, the cantilever form of erection offers several other advantages. The bearings used on the main piers can be fixed to the pier tops, and the chords framing into that point can be pinned into gusset plates. This provides a very simple and relatively maintenance-free connection to carry the major reaction of the bridge. The bearings on the end piers at the ends of the anchor spans are sometimes unique. Depending upon the requirements of the site or to reduce costs, the end spans are sometimes quite short, so that even under dead load, the reaction on the end piers is negative, which is to say an uplift condition exists under the dead load. Under some patterns of live load, this uplift will increase. When this condition exists, hanger assemblies similar to those described in Section 16.5.1 may be used to connect to a bearing fixed to the pier, and connected to an embedded steel grillage or similar device used to engage the weight of the pier to hold down the superstructure. Such a bearing is shown in Figure 16.14. The link accommodates



FIGURE 16.11 (continued)

the movement of the superstructure relative to the substructure required by expansion and contraction. The length of the arch swing of the link is designed so that the vertical displacement associated with the swing of the link can be accounted for and accommodated.

Where positive reactions are possible under all loadings, the bearing on the back span pier may be a rocker, roller nest, such as that shown in Figure 16.15, roller and gear assembly, or low-profile modern bearings, such as a pot bearing shown in Figure 16.16 as applied to a girder bridge or the disk bearing shown in Figure 16.17. It will usually be necessary for this bearing to provide for expansion and contraction while minimizing the forces put on the piers and to allow for rotation about the major bending axis of the bridge. Depending upon the designer's preferences, these bearings may or may not also carry the horizontal forces on the structure, such as wind loads, into the piers in the transverse direction. In some instances, the chord bearings serve this function through guide bars or pintles and, in some cases, a separate wind bearing, such as that shown in Figure 16.18, is provided to carry the transverse loads into piers separate from the main chord bearings.

Where structures are continuous, as opposed to cantilevered, it will usually be necessary for three of the four span bearings supporting the typical three-span truss to move. Individual movement will be greater in these bearings than they would be for a comparable-length cantilevered truss, and additional requirements will be placed on the bearing at one of the two main piers which moves, because of the large vertical reaction that will be transmitted at that point. Additionally, the continuous bridge will have two expansion joints, instead of four on the cantilever bridge, which is both an advantage and a disadvantage. These joints will have to be larger for the continuous bridge than for the cantilevered bridge and, therefore, more expensive. On the other hand, the tendency toward minimizing the number of joints in structures in order to reduce damage from deck drainage favors the continuous structure. Generally speaking, the extra points of expansion and contraction, associated deck joints and articulation hardware in the cantilevered bridge have required aboveaverage maintenance.



FIGURE 16.12 (a) Bottom false chord details — elevation view; (b) bottom false chord details — plan view.

#### 16.5.3 Wind Tongues and Bearings for Transverse Forces

Wind loads carried by the suspended span in the cantilevered bridge have to be carried to the bearings on the piers. Thus, it is necessary for the wind loads to be carried through the panels framed with the false chords. Typically, in a through-truss, all of the wind loads on the suspended span are reacted by the hangers and by a special-purpose mechanism used to transmit horizontal forces at the lower chord level from the suspended span into the anchor arms. Wind load tributary to the upper lateral truss system in the plane of the top chord joints is carried into the anchor arms as a shear at the lower chord joint and the torque necessary to react to the transfer of loads from the top chord to the lower chord is carried as equal and opposite vertical reactions on the hangers. The horizontal forces at the lower chord level are then transmitted from the suspended span to the cantilever arm by a device called a "wind tongue," shown schematically in Figure 16.19. Because of the offset in chord joints at the suspended span, the horizontal force creates a torque in the plane of the bottom lateral system as the shear is transmitted across the expansion joint. Additionally, because expansion and contraction movements are accommodated at this point, allowance has to be made for some of the lateral members to swing along with that expansion and contraction. Thus, in Figure 16.19, there are four pin assemblies shown in the detail. The two horizontal links thus swing back and forth to accommodate the relative movement occurring at the open joint. The torque caused by the offset shear is reacted by the members framing from the open joint back into



FIGURE 16.12 (continued)



FIGURE 16.13 False chord jacking details.

the next panel point of the suspended span. These members form a lever to react to torque and prevent significant rotation of the wind tongue.

The typical details for accomplishing this wind transfer are shown in Figure 16.20a and b. Figure 16.20a shows the assembly that spans the open joint between the suspended span and the anchor arm. Also shown is one of the horizontal link members. The reacting members that form the lever to react the torque are also shown in this view, and are shown again in Figure 16.20b as they converge back to a common work point in the lateral truss of the suspended span. A typical pin assembly is shown in Figure 16.21.

In the case of the continuous truss, since the open joint does not exist, no assembly similar to the wind tongue, described above, is necessary. As can be seen in a plan view, the colinear force system can be developed to transmit wind and other transverse forces into the piers without creating torque in the bottom lateral system. Despite this, designers will often support the bearing point in order to accommodate accidental eccentricies that might exist. As seen in a vertical plane, there will almost certainly be an eccentricity between the center of transverse forces and the bearing. This will also be typically framed into a triangular system to carry this eccentricity through truss action,



FIGURE 16.14 Link-type tie-down bearing.

rather than bending. These details are usually much simpler than the wind tongue at the suspended span, because it is not necessary to account simultaneously for expansion and contraction in the lateral truss system. This is usually handled by allowing the reaction points to move relative to the bearing while they are stationary relative to the lateral truss.

# 16.6 Camber and Erection

#### 16.6.1 Camber for Vertical Geometry

It is obvious that all bridges have a theoretical geometric location as determined by the final design drawings. Every member has a theoretical length and location in space. One goal of the designer, fabricator, and erector is to produce a bridge as close to the theoretical position as possible, thereby ensuring actual stresses similar to design stresses.

To accomplish this, main members are usually cambered. Tensioned members that stretch under load are fabricated, such that their unstressed length is shorter than their length under the effect of dead load of the structure. The opposite is true for compression members. The cambered lengths are then accounted for during the erection stress and geometry studies. The state of the bridge when the camber "comes out" is called the geometric position. In this state the loads on the bridge are sufficient to return all of the members to their theoretical length. At any other state, the bridge will be out of shape and additional forces may result from the difference between its shape at any time and the final shape. This is relatively easy to see in the case of a continuous truss because it is clearly statically indeterminant and the shears and moments producing member forces are dependent on



FIGURE 16.15 Roller bearing.



FIGURE 16.16 (a) Typical pot bearing; (b) typical pot bearing details.



FIGURE 16.17 Typical disc bearing.

its shape. However, this is also true for a simple span truss because the joints are not frictionless pins as may have been assumed in the analysis. Because of this, even the simplest form of truss can have significant temporary member forces and moments until it reaches the geometric position. As will be discussed in the next section, there may be secondary moments in the geometric position, depending on the positioning of connector patterns on member ends in the shop.

Secondary members such as laterals and sway frame members are usually not cambered. They are usually intended to be stress free in the geometric position. Thus, at intermediate stages of erection they may also be subject to temporary forces.



FIGURE 16.18 Truss wind bearing.



FIGURE 16.19 Schematic of wind tongue.

The importance of camber in achieving the designer's intent for the structure will be shown in the following discussion. In a determinant structure, the forces in components are uniquely determined by the geometry, loading, and support condition through the equations of equilibrium. The designer cannot alter the structural actions.

In a redundant structure, the designer can alter the forces associated with any one loading case. After this, the distribution of forces is again uniquely determined by all the conditions above, plus the relative stiffness of the various structural components.

Consider a simple case. If a two-span simply supported beam supporting a uniform load is intended to be horizontal under that load, the natural order is to have the reactions and negative moment at the pier equal to the value shown below at Piers 1, 2, and 3, respectively.

$$R_1 = \frac{3w\ell}{8} = R_3 \tag{16.1}$$

$$R_2 = \frac{10w\ell}{8} \tag{16.2}$$

$$M_2 = \frac{w\ell^2}{8}$$
(16.3)

Given the reactions, the load and length of the beam, the deflection at any point of the beam can be determined. Given that the example started with the condition that the beam is to be horizontal, calculation of deflections would reveal that the deflection is zero at all three support points and that the beam deflects downward between the support points. In order to put the beam on a horizontal position under a load, the web plates must be cut to the reverse of the deflected shape.

An alternative set of reactions and a corresponding moment diagram can be determined for this beam by cambering it so that not all three support points are on a straight line. For example, suppose it was desired to reduce the negative moment at the center pier. This could be achieved by cutting the web plate such that, in the horizontal layout position, the girder would rest such that a straight line between the two external reactions would be below the middle of the beam (with gravity acting downward). Thus, when the beam is assembled in the field, a certain amount of bending will have to take place as a simply supported beam of span equal to 2L until such time as the girder touches the middle bearing. After that, the remainder of the loads will be carried as a two-span unit. Unless a loading is sufficient in magnitude and opposite in sense so as to create uplift at the center bearing, all loads will then be handled as a two-span continuous unit. In this way, one might determine that an optimum condition is to have equal dead-load moments in the two spans and over the support. All that is necessary to achieve this is to determine the offset distance off the bearing point in the laydown position. Then cut the beam to that configuration using the cambered shape and erect it.

In truss construction, this adjustment of natural forces is seldom actually done. On occasion, camber to produce a determinant structure at steel closure has been used to facilitate erection. Camber for force control is very common in other types of bridges.

#### 16.6.2 Camber of Joints

When the principal operations on a main member, such as punching, drilling, and cutting are completed, and when the detail pieces connecting to it are fabricated, all the components are brought together to be fitted up, i.e., temporarily assembled with fit-up bolts, clamps, or tack welds. At this time, the member is inspected for dimensional accuracy, squareness, and, in general, conformance with shop detail drawings. Misalignment in holes in mating parts should be detected then and holes reamed, if necessary, for insertion of bolts. When fit-up is completed, the member is bolted or welded with final shop connections.

The foregoing type of shop preassembly or fit-up is an ordinary shop practice, routinely performed on virtually all work. There is another class of fit-up, however, mainly associated with highway and railroad bridges that may be required by project specifications. These may specify that the holes in bolted field connections and splices be reamed while the members are assembled in the shop. Such requirements should be reviewed carefully before they are specified. The steps of subpunching (or subdrilling), shop assembly and reaming for field connections add significant costs. Modern computer-controlled drilling equipment can provide full-size holes located with a high degree of accuracy. AASHTO Specifications, for example, include provisions for reduced shop assembly procedures when computer-controlled drilling operations are used.

#### 16.6.3 Common Erection Methods

The most common construction methods for trusses include cantilever construction, falsework, float-ins, and tiebacks. It is common for more than one method to be used in the construction of any single bridge. The methods selected to erect a bridge may depend on several factors, including the type of bridge, bridge length and height, type and amount of river traffic, water depth, adjacent geographic conditions, cost, and weight, availability, and cost of erection equipment.

Regardless of the method of erection that is used, an erection schedule should be prepared prior to starting the erection of any long-span bridge. The study should include bridge geometry ,member stress, and stability at all stages of erection. Bridges under construction often work completely differently than they do in their finished or final condition, and the character of the stress is changed, as from tension to compression. Stresses induced by erection equipment must also be checked, and, it goes without saying, large bridges under construction must be checked for wind stresses and sometimes wind-induced vibrations. An erection schedule should generally include a fit-up schedule for bolting major joints and a closing procedure to join portions of a bridge coming from opposite directions. Occasionally, permanent bridge members must be strengthened to withstand temporary erection loads. Prior to the erection of any bridge, proper controls for bridge line and elevation must be established and then maintained for the duration of the construction period.

Most long-span bridge construction projects have a formal closing procedure prepared by bridge engineers. The bridge member or assembled section erected to complete a span must fit the longitudinal



FIGURE 16.20 (a) Details of wind bearing; (b) details of wind tongue.

opening for it, and it must properly align with both adjoining sections of the bridge. Proper alignment of the closing piece is generally obtained by vertical jacking of falsework, lifting the existing bridge with a tieback system, or horizontal jacking of truss chords. At the Greater New Orleans Bridge No. 2, a scissors jack was inserted in place of a dummy top chord member, and it was used to pivot the main span truss for proper alignment.











FIGURE 16.22 Cantilever erection.



FIGURE 16.23 Early stage.

#### 16.6.3.1 Cantilever Construction

In balanced cantilever construction, a bridge cantilevers in both directions from a single pier, as shown in Figure 16.22. The loads on each side of a pier must be kept reasonably in balance. In this type of construction, the first horizontal member erected in each direction may have to be temporarily supported by a brace, as shown in Figure 16.23.

In other types of cantilever construction, one span of a truss may be complete, and the bridge then cantilevers into an adjacent span, as shown in Figure 16.24. This type of construction is quite common.

#### 16.6.3.2 Falsework

Falsework is commonly used when building medium-span trusses, especially for bridge approaches over land, as shown in Figures 16.25 and 16.26. It is also used in long-span bridge construction, but heavy river traffic, deep water, or poor foundation material may restrict its use. Falsework is sometimes used in the anchor or side span of cantilever trusses.

#### 16.6.3.3 Float-In

Float-in is another commonly used construction method for long-span bridges. In this method, a portion of the bridge is generally assembled on a barge. The barge is then moved to the construction site, and the bridge is then either set into place off the barge or pulled vertically into place from the barge and connected to the part of the bridge already constructed. This method was used on the second Newburgh–Beacon Bridge across the Hudson River as discussed below.

First, a short section of the main span truss was erected on a barge; then sections of the approach spans over the river were assembled on top of the main span truss, floated into position, and erected, as shown in Figures 16.27 and 16.28. The section of bridge floated in is generally higher than its final position, and it is lowered into position with jacks. The two anchor spans were then assembled on a barge and erected the same way, as shown in Figure 16.29.



FIGURE 16.24 Advanced stage of cantilevering.



FIGRE 16.25 Use of falsework — 1.

The cantilever portion of the bridge was then erected member by member by the cantilever erection method out into the main span to the pin hangers, as shown in Figure 16.30, which also shows the arrival of the suspended span of the main span. The suspended span was then barged into position and hoisted in place as shown in Figure 16.31.



FIGURE 16.26 Use of falsework — 2.



**FIGURE 16.27** Float-in — 1.

The forces obtained during various stages of construction may be entirely different from those applicable to the final condition, and, in fact, the entire mechanism for resisting forces within the structure may change. During the erection of a truss, falsework bents may be used to support a portion of the structure. The gravity and lateral loads still act in the sense that they do on the final



FIGURE 16.28 Float-in — 2.



**FIGURE 16.29** Float-in — 3.

condition, and the basic internal mechanism of resisting forces, as outlined in Chapter 10 of the Standard Specifications or Section 6 of the AASHTO-LRFD Specifications, remains unchanged, i.e., the primary load path involves axial load in the chords and diagonals, with the vertical components of those forces adding up to equal the applied shear within the panel, and the bending moment accounted for by the horizontal component of those forces. The camber of the truss will not pertain



FIGURE 16.30 Float-in of suspended span.



FIGURE 16.31 Lift of suspended span.

to an intermediate construction case, and, therefore, there is apt to be more joint rotation and, hence, more secondary bending moments within the truss members. Nonetheless, the primary loadcarrying mechanism is that of axial forces in members. As the truss is erected, it is entirely possible that members that are in tension in the permanent condition will be under compression during erection, and vice versa. In fact, the state of stress may reverse several times during the erection of the bridge. Clearly, this has to be taken into account, not only in the design of the members, but also in the design of the connections. Compression members are apt to have been designed to transmit part of the forces in bearing. This will not be applicable when the member is in tension during erection. Similarly, a compression member that has tension during erection has to be reviewed for net section provisions and shear lag provisions.

# 16.7 Summary

Truss bridges have been an effective and efficient force of long-span bridges for over 150 years. As plate girder bridges have been utilized for spans of about 550 ft, box girders for spans of up to 750 ft, segmental concrete box girders for spans of up to about 800 ft, and cable-stayed bridges for spans of about 500 feet to 2000 ft, the use of trusses has declined over the last 25 years. Nonetheless, they remain a cost-effective bridge form, one with which many fabricators and erectors are experienced. Emerging materials and the use of computer analysis to treat the bridge as a three-dimensional structure will keep the truss form viable for the foreseeable future.

#### References

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