# SECTION 13
# TRUSS BRIDGES*

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A truss is a structure that acts like a beam but with major components, or members, subjected primarily to axial stresses. The members are arranged in triangular patterns. Ideally, the end of each member at a joint is free to rotate independently of the other members at the joint. If this does not occur, secondary stresses are induced in the members. Also if loads occur other than at panel points, or joints, bending stresses are produced in the members.

Though trusses were used by the ancient Romans, the modern truss concept seems to have been originated by Andrea Palladio, a sixteenth century Italian architect. From his time to the present, truss bridges have taken many forms.

Early trusses might be considered variations of an arch. They applied horizontal thrusts at the abutments, as well as vertical reactions, In 1820, Ithiel Town patented a truss that can be considered the forerunner of the modern truss. Under vertical loading, the Town truss exerted only vertical forces at the abutments. But unlike modern trusses, the diagonals, or web systems, were of wood lattice construction and chords were composed of two or more timber planks.

In 1830, Colonel Long of the U.S. Corps of Engineers patented a wood truss with a simpler web system. In each panel, the diagonals formed an X. The next major step came in 1840, when William Howe patented a truss in which he used wrought-iron tie rods for vertical web members, with X wood diagonals. This was followed by the patenting in 1844 of the Pratt truss with wrought-iron X diagonals and timber verticals.

The Howe and Pratt trusses were the immediate forerunners of numerous iron bridges. In a book published in 1847, Squire Whipple pointed out the logic of using cast iron in compression and wrought iron in tension. He constructed bowstring trusses with cast-iron verticals and wrought-iron X diagonals.

*Revised and updated from Sec. 12, "Truss Bridges," by Jack P. Shedd, in the first edition.

These trusses were statically indeterminate. Stress analysis was difficult. Latter, simpler web systems were adopted, thus eliminating the need for tedious and exacting design procedures.

To eliminate secondary stresses due to rigid joints, early American engineers constructed pin-connected trusses. European engineers primarily used rigid joints. Properly proportioned, the rigid trusses gave satisfactory service and eliminated the possibility of frozen pins, which induce stresses not usually considered in design. Experience indicated that rigid and pin-connected trusses were nearly equal in cost, except for long spans. Hence, modern design favors rigid joints.

Many early truss designs were entirely functional, with little consideration given to appearance. Truss members and other components seemed to lie in all possible directions and to have a variety of sizes, thus giving the impression of complete disorder. Yet, appearance of a bridge often can be improved with very little increase in construction cost. By the 1970s, many speculated that the cable-stayed bridge would entirely supplant the truss, except on railroads. But improved design techniques, including load-factor design, and streamlined detailing have kept the truss viable. For example, some designs utilize Warren trusses without verticals. In some cases, sway frames are eliminated and truss-type portals are replaced with beam portals, resulting in an open appearance.

Because of the large number of older trusses still in the transportation system, some historical information in this section applies to those older bridges in an evaluation or rehabilitation context.

(H. J. Hopkins, ‘‘A Span of Bridges,’’ Praeger Publishers, New York; S. P. Timoshenko, ‘‘History of Strength of Materials,’’ McGraw-Hill Book Company, New York).

## 13.1 SPECIFICATIONS

The design of truss bridges usually follows the specifications of the American Association of State Highway and Transportation Officials (AASHTO) or the Manual of the American Railway Engineering and Maintenance of Way Association (AREMA) (Sec. 10). A transition in AASHTO specifications is currently being made from the ‘‘Standard Specifications for Highway Bridges,’’ Sixteenth Edition, to the ‘‘LRFD Specifications for Highway Bridges,’’ Second Edition. The ‘‘Standard Specification’’ covers service load design of truss bridges, and in addition, the ‘‘Guide Specification for the Strength Design of Truss Bridges,’’ covers extension of the load factor design process permitted for girder bridges in the ‘‘Standard Specifications’’ to truss bridges. Where the ‘‘Guide Specification’’ is silent, applicable provisions of the ‘‘Standard Specification’’ apply.

To clearly identify which of the three AASHTO specifications are being referred to in this section, the following system will be adopted. If the provision under discussion applies to all the specifications, reference will simply be made to the ‘‘AASHTO Specifications’’. Otherwise, the following notation will be observed:

‘‘AASHTO SLD Specifications’’ refers to the service load provisions of ‘‘Standard Specifications for Highway Bridges’’

‘‘AASHTO LFD Specifications’’ refers to ‘‘Guide Specification for the Strength Design of Truss Bridges’’

‘‘AASHTO LRFD Specifications’’ refers to ‘‘LRFD Specifications for Highway Bridges.’’

## 13.2 TRUSS COMPONENTS

Principal parts of a highway truss bridge are indicated in Fig. 13.1; those of a railroad truss are shown in Fig. 13.2.

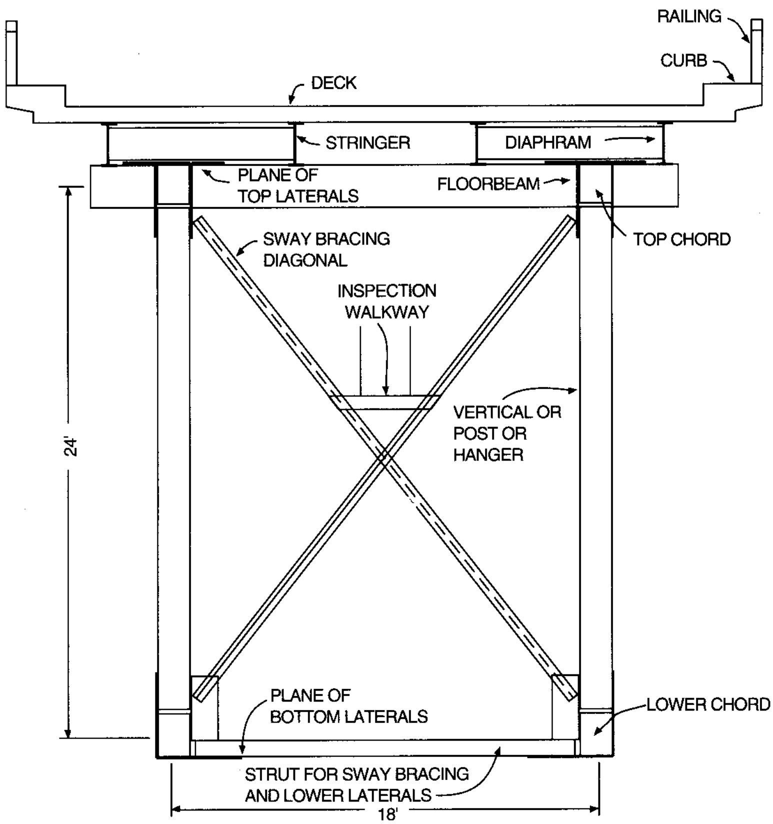



**FIGURE 13.1** Cross section shows principal parts of a deck-truss highway bridge.

**Joints** are intersections of truss members. Joints along upper and lower chords often are referred to as panel points. To minimize bending stresses in truss members, live loads generally are transmitted through floor framing to the panel points of either chord in older, shorter-span trusses. Bending stresses in members due to their own weight was often ignored in the past. In modern trusses, bending due to the weight of the members should be considered.

**Chords** are top and bottom members that act like the flanges of a beam. They resist the tensile and compressive forces induced by bending. In a constant-depth truss, chords are essentially parallel. They may, however, range in profile from nearly horizontal in a moderately variable-depth truss to nearly parabolic in a bowstring truss. Variable depth often improves economy by reducing stresses where chords are more highly loaded, around midspan in simple-span trusses and in the vicinity of the supports in continuous trusses.

**Web members** consist of diagonals and also often of verticals. Where the chords are essentially parallel, diagonals provide the required shear capacity. Verticals carry shear, provide additional panel points for introduction of loads, and reduce the span of the chords under dead-load bending. When subjected to compression, verticals often are called posts, and when subjected to tension, hangers. Usually, deck loads are transmitted to the trusses through end connections of floorbeams to the verticals.

**Counters,** which are found on many older truss bridges still in service, are a pair of diagonals placed in a truss panel, in the form of an X, where a single diagonal would be

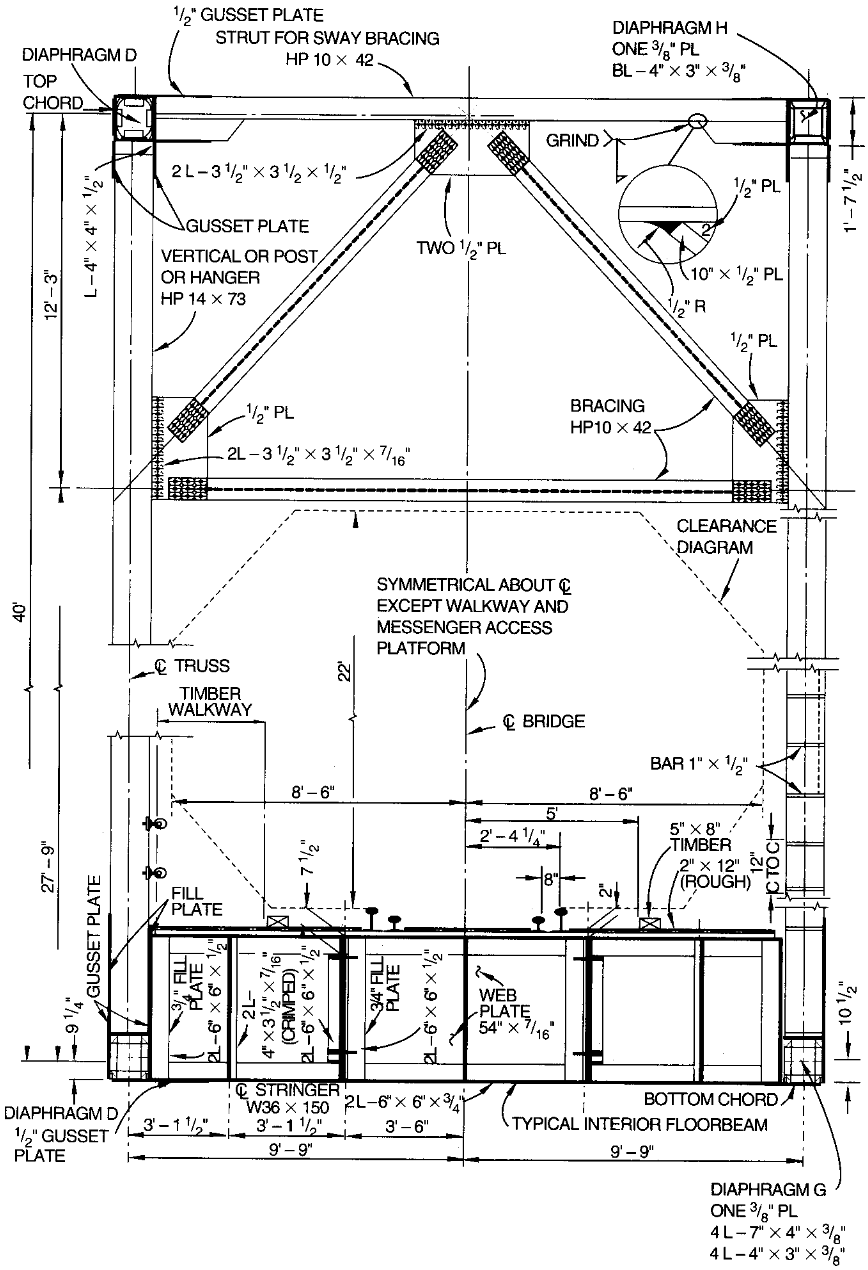



**FIGURE 13.2** Cross section shows principal parts of a through-truss railway bridge.

subjected to stress reversals. Counters were common in the past in short-span trusses. Such short-span trusses are no longer economical and have been virtually totally supplanted by beam and girder spans. X pairs are still used in lateral trusses, sway frames and portals, but are seldom designed to act as true counters, on the assumption that only one counter acts at a time and carries the maximum panel shear in tension. This implies that the companion counter takes little load because it buckles. In modern design, counters are seldom used in the primary trusses. Even in lateral trusses, sway frames, and portals, X-shaped trusses are usually comprised of rigid members, that is, members that will not buckle. If adjustable counters are used, only one may be placed in each truss panel, and it should have open turnbuckles. AASHTO LRFD specifies that counters should be avoided. The commentary to that provision contains reference to the historical initial force requirement of 10 kips. Design of such members by AASHTO SLD or LFD Specifications should include an allowance of 10 kips for initial stress. Sleeve nuts and loop bars should not be used.

**End posts** are compression members at supports of simple-span tusses. Wherever practical, trusses should have inclined end posts. Laterally unsupported hip joints should not be used.

**Working lines** are straight lines between intersections of truss members. To avoid bending stresses due to eccentricity, the gravity axes of truss members should lie on working lines. Some eccentricity may be permitted, however, to counteract dead-load bending stresses. Furthermore, at joints, gravity axes should intersect at a point. If an eccentric connection is unavoidable, the additional bending caused by the eccentricity should be included in the design of the members utilizing appropriate interaction equations.

*AASHTO Specifications require that members be symmetrical about the central plane of a truss. They should be proportioned so that the gravity axis of each section lies as nearly as practicable in its center.*

Connections may be made with welds or high-strength bolts. AREMA practice, however, excludes field welding, except for minor connections that do not support live load.

**The deck** is the structural element providing direct support for vehicular loads. Where the deck is located near the bottom chords (through spans), it should be supported by only two trusses.

**Floorbeams** should be set normal or transverse to the direction of traffic. They and their connections should be designed to transmit the deck loads to the trusses.

**Stringers** are longitudinal beams, set parallel to the direction of traffic. They are used to transmit the deck loads to the floorbeams. If stringers are not used, the deck must be designed to transmit vehicular loads to the floorbeams.

**Lateral bracing** should extend between top chords and between bottom chords of the two trusses. This bracing normally consists of trusses placed in the planes of the chords to provide stability and lateral resistance to wind. Trusses should be spaced sufficiently far apart to preclude overturning by design lateral forces.

**Sway bracing** may be inserted between truss verticals to provide lateral resistance in vertical planes. Where the deck is located near the bottom chords, such bracing, placed between truss tops, must be kept shallow enough to provide adequate clearance for passage of traffic below it. Where the deck is located near the top chords, sway bracing should extend in full-depth of the trusses.

**Portal bracing** is sway bracing placed in the plane of end posts. In addition to serving the normal function of sway bracing, portal bracing also transmits loads in the top lateral bracing to the end posts (Art. 13.6).

**Skewed bridges** are structures supported on piers that are not perpendicular to the planes of the trusses. The **skew angle** is the angle between the transverse centerline of bearings and a line perpendicular to the longitudinal centerline of the bridge.

## *13.3 TYPES OF TRUSSES*

Figure 13.3 shows some of the common trusses used for bridges. **Pratt trusses** have diagonals sloping downward toward the center and parallel chords (Fig. 13.3*a*). **Warren trusses,**

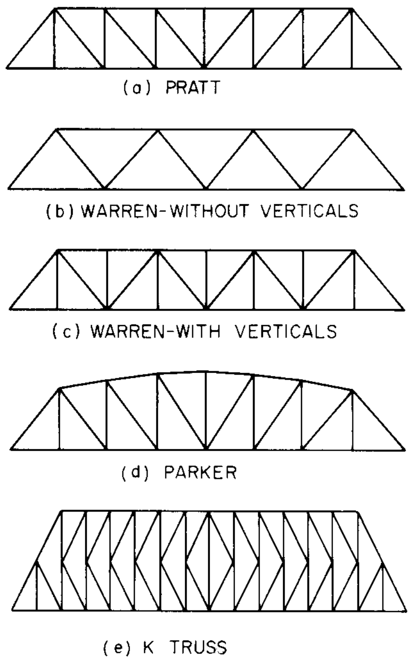



**FIGURE 13.3** Types of simple-span truss bridges.

with parallel chords and alternating diagonals, are generally, but not always, constructed with verticals (Fig. 13.3*c*) to reduce panel size. When rigid joints are used, such trusses are favored because they provide an efficient web system. Most modern bridges are of some type of Warren configuration.

**Parker trusses** (Fig. 13.3*d*) resemble Pratt trusses but have variable depth. As in other types of trusses, the chords provide a couple that resists bending moment. With long spans, economy is improved by creating the required couple with less force by spacing the chords farther apart. The Parker truss, when simply supported, is designed to have its greatest depth at midspan, where moment is a maximum. For greatest chord economy, the top-chord profile should approximate a parabola. Such a curve, however, provides too great a change in slope of diagonals, with some loss of economy in weights of diagonals. In practice, therefore, the top-chord profile should be set for the greatest change in truss depth commensurate with reasonable diagonal slopes; for example, between 40° and 60 ° with the horizontal.

**K trusses** (Fig. 13.3*e*) permit deep trusses with short panels to have diagonals with acceptable slopes. Two diagonals generally are placed in each panel to intersect at midheight of a vertical. Thus, for each diagonal, the slope is half as large as it would be if a single diagonal were used in the panel. The short panels keep down the cost of the floor system. This cost would rise rapidly if panel width were to increase considerably with increase in span. Thus, K trusses may be economical for long spans, for which deep trusses and narrow panels are desirable. These trusses may have constant or variable depth.

Bridges also are classified as highway or railroad, depending on the type of loading the bridge is to carry. Because highway loading is much lighter than railroad, highway trusses generally are built of much lighter sections. Usually, highways are wider than railways, thus requiring wider spacing of trusses.

Trusses are also classified as to location of deck: deck, through, or half-through trusses. **Deck trusses** locate the deck near the top chord so that vehicles are carried above the chord. **Through trusses** place the deck near the bottom chord so that vehicles pass between the trusses. **Half-through trusses** carry the deck so high above the bottom chord that lateral and sway bracing cannot be placed between the top chords. The choice of deck or through construction normally is dictated by the economics of approach construction.

The absence of top bracing in half-through trusses calls for special provisions to resist lateral forces. AASHTO Specifications require that truss verticals, floorbeams, and their end connections be proportioned to resist a lateral force of at least 0.30 kip per lin ft, applied at the top chord panel points of each truss. The top chord of a half-through truss should be designed as a column with elastic lateral supports at panel points. The critical buckling force of the column, so determined, should be at least 50% larger than the maximum force induced in any panel of the top chord by dead and live loads plus impact. Thus, the verticals have to be designed as cantilevers, with a concentrated load at top-chord level and rigid connection to a floorbeam. This system offers elastic restraint to buckling of the top chord. The analysis of elastically restrained compression members is covered in T. V. Galambos, "Guide to Stability Design Criteria for Metal Structures," Structural Stability Research Council.

## 13.4 BRIDGE LAYOUT

Trusses, offering relatively large depth, open-web construction, and members subjected primarily to axial stress, provide large carrying capacity for comparatively small amounts of steel. For maximum economy in truss design, the area of metal furnished for members should be varied as often as required by the loads. To accomplish this, designers usually have to specify built-up sections that require considerable fabrication, which tend to offset some of the savings in steel.

***Truss Spans.*** Truss bridges are generally comparatively easy to erect, because light equipment often can be used. Assembly of mechanically fastened joints in the field is relatively labor-intensive, which may also offset some of the savings in steel. Consequently, trusses seldom can be economical for highway bridges with spans less than about 450 ft.

Railroad bridges, however, involve different factors, because of the heavier loading. Trusses generally are economical for railroad bridges with spans greater than 150 ft.

The current practical limit for simple-span trusses is about 800 ft for highway bridges and about 750 ft for railroad bridges. Some extension of these limits should be possible with improvements in materials and analysis, but as span requirements increase, cantilever or continuous trusses are more efficient. The North American span record for cantilever construction is 1,600 ft for highway bridges and 1,800 ft for railroad bridges.

For a bridge with several truss spans, the most economical pier spacing can be determined after preliminary designs have been completed for both substructure and superstructure. One guideline provides that the cost of one pier should equal the cost of one superstructure span, excluding the floor system. In trial calculations, the number of piers initially assumed may be increased or decreased by one, decreasing or increasing the truss spans. Cost of truss spans rises rapidly with increase in span. A few trial calculations should yield a satisfactory picture of the economics of the bridge layout. Such an analysis, however, is more suitable for approach spans than for main spans. In most cases, the navigation or hydraulic requirement is apt to unbalance costs in the direction of increased superstructure cost. Furthermore, girder construction is currently used for span lengths that would have required approach trusses in the past.

***Panel Dimensions.*** To start economic studies, it is necessary to arrive at economic proportions of trusses so that fair comparisons can be made among alternatives. Panel lengths will be influenced by type of truss being designed. They should permit slope of the diagonals between 40° and 60° with the horizontal for economic design. If panels become too long, the cost of the floor system substantially increases and heavier dead loads are transmitted to the trusses. A subdivided truss becomes more economical under these conditions.

For simple-span trusses, experience has shown that a depth-span ratio of 1:5 to 1:8 yields economical designs. Some design specifications limit this ratio, with 1:10 a common historical limit. For continuous trusses with reasonable balance of spans, a depth-span ratio of 1:12 should be satisfactory. Because of the lighter live loads for highways, somewhat shallower depths of trusses may be used for highway bridges than for railway bridges.

Designers, however, do not have complete freedom in selection of truss depth. Certain physical limitations may dictate the depth to be used. For through-truss highway bridges, for example, it is impractical to provide a depth of less than 24 ft, because of the necessity of including suitable sway frames. Similarly, for through railway trusses, a depth of at least 30 ft is required. The trend toward double-stack cars encourages even greater minimum depths.

Once a starting depth and panel spacing have been determined, permutation of primary geometric variables can be studied efficiently by computer-aided design methods. In fact, preliminary studies have been carried out in which every primary truss member is designed for each choice of depth and panel spacing, resulting in a very accurate choice of those parameters.

***Bridge Cross Sections.*** Selection of a proper bridge cross section is an important determination by designers. In spite of the large number of varying cross sections observed in truss bridges, actual selection of a cross section for a given site is not a large task. For instance, if a through highway truss were to be designed, the roadway width would determine the transverse spacing of trusses. The span and consequent economical depth of trusses would determine the floorbeam spacing, because the floorbeams are located at the panel points. Selection of the number of stringers and decisions as to whether to make the stringers simple spans between floorbeams or continuous over the floorbeams, and whether the stringers and floorbeams should be composite with the deck, complete the determination of the cross section.

Good design of framing of floor system members requires attention to details. In the past, many points of *stress relief* were provided in floor systems. Due to corrosion and wear resulting from use of these points of movement, however, experience with them has not always been good. Additionally, the relative movement that tends to occur between the deck and the trusses may lead to out-of-plane bending of floor system members and possible fatigue damage. Hence, modern detailing practice strives to eliminate small unconnected gaps between stiffeners and plates, rapid change in stiffness due to excessive flange coping, and other distortion fatigue sites. Ideally, the whole structure is made to act as a unit, thus eliminating distortion fatigue.

Deck trusses for highway bridges present a few more variables in selection of cross section. Decisions have to be made regarding the transverse spacing of trusses and whether the top chords of the trusses should provide direct support for the deck. Transverse spacing of the trusses has to be large enough to provide lateral stability for the structure. Narrower truss spacings, however, permit smaller piers, which will help the overall economy of the bridge.

Cross sections of railway bridges are similarly determined by physical requirements of the bridge site. Deck trusses are less common for railway bridges because of the extra length of approach grades often needed to reach the elevation of the deck. Also, use of through trusses offers an advantage if open-deck construction is to be used. With through-trusses, only the lower chords are vulnerable to corrosion caused by salt and debris passing through the deck.

After preliminary selection of truss type, depth, panel lengths, member sizes, lateral systems, and other bracing, designers should review the appearance of the entire bridge. Esthetics can often be improved with little economic penalty.

## 13.5 DECK DESIGN

For most truss members, the percentage of total stress attributable to dead load increases as span increases. Because trusses are normally used for long spans, and a sizable portion of the dead load (particularly on highway bridges) comes from the weight of the deck, a lightweight deck is advantageous. It should be no thicker than actually required to support the design loading.

In the preliminary study of a truss, consideration should be given to the cost, durability, maintainability, inspectability, and replaceability of various deck systems, including transverse, longitudinal, and four-way reinforced concrete decks, orthotropic-plate decks, and concrete-filled or overlaid steel grids. Open-grid deck floors will seldom be acceptable for new fixed truss bridges but may be advantageous in rehabilitation of bridges and for movable bridges.

The design procedure for railroad bridge decks is almost entirely dictated by the proposed cross section. Designers usually have little leeway with the deck, because they are required to use standard railroad deck details wherever possible.

Deck design for a highway bridge is somewhat more flexible. Most highway bridges have a reinforced-concrete slab deck, with or without an asphalt wearing surface. Reinforced concrete decks may be transverse, longitudinal or four-way slabs.

- Transverse slabs are supported on stringers spaced close enough so that all the bending in the slabs is in a transverse direction.
- Longitudinal slabs are carried by floorbeams spaced close enough so that all the bending in the slabs is in a longitudinal direction. Longitudinal concrete slabs are practical for short-span trusses where floorbeam spacing does not exceed about 20 ft. For larger spacing, the slab thickness becomes so large that the resultant dead load leads to an uneconomic truss design. Hence, longitudinal slabs are seldom used for modern trusses.
- Four-way slabs are supported directly on longitudinal stringers and transverse floorbeams. Reinforcement is placed in both directions. The most economical design has a spacing of stringers about equal to the spacing of floorbeams. This restricts use of this type of floor system to trusses with floorbeam spacing of about 20 ft. As for floor systems with a longitudinal slab, four-way slabs are generally uneconomical for modern bridges.

## 13.6 LATERAL BRACING, PORTALS, AND SWAY FRAMES

Lateral bracing should be designed to resist the following: (1) Lateral forces due to wind pressure on the exposed surface of the truss and on the vertical projection of the live load. (2) Seismic forces, (3) Lateral forces due to centrifugal forces when the track or roadway is curved. (4) For railroad bridges, lateral forces due to the nosing action of locomotives caused by unbalanced conditions in the mechanism and also forces due to the lurching movement of cars against the rails because of the play between wheels and rails. Adequate bracing is one of the most important requirements for a good design.

Since the loadings given in design specifications only approximate actual loadings, it follows that refined assumptions are not warranted for calculation of panel loads on lateral trusses. The lateral forces may be applied to the windward truss only and divided between the top and bottom chords according to the area tributary to each. A lateral bracing truss is placed between the top chords or the bottom chords, or both, of a pair of trusses to carry these forces to the ends of the trusses.

Besides its use to resist lateral forces, other purposes of lateral bracing are to provide stability, stiffen structures and prevent unwarranted lateral vibration. In deck-truss bridges, however, the floor system is much stiffer than the lateral bracing. Here, the major purpose of lateral bracing is to true-up the bridges and to resist wind load during erection.

The portal usually is a sway frame extending between a pair of trusses whose purpose also is to transfer the reactions from a lateral-bracing truss to the end posts of the trusses, and, thus, to the foundation. This action depends on the ability of the frame to resist transverse forces.

The portal is normally a statically indeterminate frame. Because the design loadings are approximate, an exact analysis is seldom warranted. It is normally satisfactory to make simplifying assumptions. For example, a plane of contraflexure may be assumed halfway between the bottom of the portal knee brace and the bottom of the post. The shear on the plane may be assumed divided equally between the two end posts.

Sway frames are placed between trusses, usually in vertical planes, to stiffen the structure (Fig. 13.1 and 13.2). They should extend the full depth of deck trusses and should be made as deep as possible in through trusses. The AASHTO SLD Specifications require sway frames in every panel. But many bridges are serving successfully with sway frames in every other panel, even lift bridges whose alignment is critical. Some designs even eliminate sway frames entirely. The AASHTO LRFD Specifications makes the use and number of sway frames a matter of design concept as expressed in the analysis of the structural system.

Diagonals of sway frames should be proportioned for slenderness ratio as compression members. With an X system of bracing, any shear load may be divided equally between the diagonals. An approximate check of possible loads in the sway frame should be made to ensure that stresses are within allowable limits.

## 13.7 RESISTANCE TO LONGITUDINAL FORCES

Acceleration and braking of vehicular loads, and longitudinal wind, apply longitudinal loads to bridges. In highway bridges, the magnitudes of these forces are generally small enough that the design of main truss members is not affected. In railroad bridges, however, chords that support the floor system might have to be increased in section to resist tractive forces. In all truss bridges, longitudinal forces are of importance in design of truss bearings and piers.

In railway bridges, longitudinal forces resulting from accelerating and braking may induce severe bending stresses in the flanges of floorbeams, at right angles to the plane of the web, unless such forces are diverted to the main trusses by traction frames. In single-track bridges, a transverse strut may be provided between the points where the main truss laterals cross the stringers and are connected to them (Fig. 13.4*a*). In double-track bridges, it may be necessary to add a traction truss (Fig. 13.4*b*).

When the floorbeams in a double-track bridge are so deep that the bottoms of the stringers are a considerable distance above the bottoms of the floorbeams, it may be necessary to raise the plane of the main truss laterals from the bottom of the floorbeams to the bottom of the stringers. If this cannot be done, a complete and separate traction frame may be provided either in the plane of the tops of the stringers or in the plane of their bottom flanges.

The forces for which the traction frames are designed are applied along the stringers. The magnitudes of these forces are determined by the number of panels of tractive or braking force that are resisted by the frames. When one frame is designed to provide for several panels, the forces may become large, resulting in uneconomical members and connections.

## 13.8 TRUSS DESIGN PROCEDURE

The following sequence may serve as a guide to the design of truss bridges:

- Select span and general proportions of the bridge, including a tentative cross section.
- Design the roadway or deck, including stringers and floorbeams.
- Design upper and lower lateral systems.
- Design portals and sway frames.
- Design posts and hangers that carry little stress or loads that can be computed without a complete stress analysis of the entire truss.
- Compute preliminary moments, shears, and stresses in the truss members.
- Design the upper-chord members, starting with the most heavily stressed member.
- Design the lower-chord members.
- Design the web members.

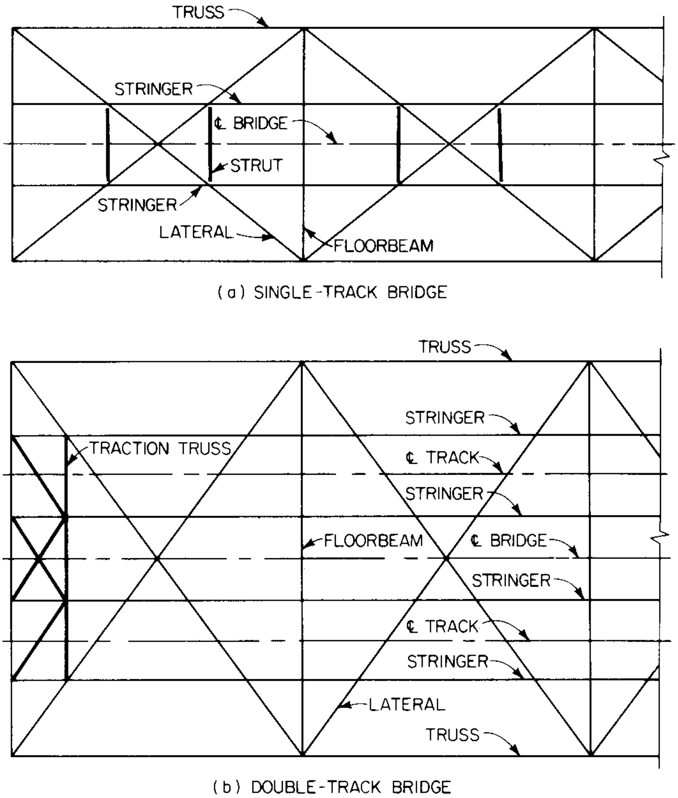



**FIGURE 13.4** Lateral bracing and traction trusses for resisting longitudinal forces on a truss bridge.

- Recalculate the dead load of the truss and compute final moments and stresses in truss members.
- Design joints, connections, and details.
- Compute dead-load and live-load deflections.
- Check secondary stresses in members carrying direct loads and loads due to wind.
- Review design for structural integrity, esthetics, erection, and future maintenance and inspection requirements.

### 13.8.1 Analysis for Vertical Loads

Determination of member forces using conventional analysis based on frictionless joints is often adequate when the following conditions are met:

1. The plane of each truss of a bridge, the planes through the top chords, and the planes through the bottom chords are fully triangulated.
2. The working lines of intersecting truss members meet at a point.

3. Cross frames and other bracing prevent significant distortions of the box shape formed by the planes of the truss described above.
4. Lateral and other bracing members are not cambered; i.e., their lengths are based on the final dead-load position of the truss.
5. Primary members are cambered by making them either short or long by amounts equal to, and opposite in sign to, the axial compression or extension, respectively, resulting from dead-load stress. Camber for trusses can be considered as a correction for dead-load deflection. (If the original design provided excess vertical clearance and the engineers did not object to the sag, then trusses could be constructed without camber. Most people, however, object to sag in bridges.) The cambering of the members results in the truss being out of vertical alignment until all the dead loads are applied to the structure (geometric condition).

When the preceding conditions are met and are rigorously modeled, three-dimensional computer analysis yields about the same dead-load axial forces in the members as the conventional pin-connected analogy and small secondary moments resulting from the self-weight bending of the member. Application of loads other than those constituting the geometric condition, such as live load and wind, will result in sag due to stressing of both primary and secondary members in the truss.

Rigorous three-dimensional analysis has shown that virtually all the bracing members participate in live-load stresses. As a result, total stresses in the primary members are reduced below those calculated by the conventional two-dimensional pin-connected truss analogy. Since trusses are usually used on relatively long-span structures, the dead-load stress constitutes a very large part of the total stress in many of the truss members. Hence, the savings from use of three-dimensional analysis of the live-load effects will usually be relatively small. This holds particularly for through trusses where the eccentricity of the live load, and, therefore, forces distributed in the truss by torsion are smaller than for deck trusses.

The largest secondary stresses are those due to moments produced in the members by the resistance of the joints to rotation. Thus, the secondary stresses in a pin-connected truss are theoretically less significant than those in a truss with mechanically fastened or welded joints. In practice, however, pinned joints always offer frictional resistance to rotation, even when new. If pin-connected joints freeze because of dirt, or rust, secondary stresses might become higher than those in a truss with rigid connections. Three-dimensional analysis will however, quantify secondary stresses, if joints and framing of members are accurately modeled. If the secondary stress exceeds 4 ksi for tension members or 3 ksi for compression members, both the AASHTO SLD and LFD Specifications require that excess be treated as a primary stress. The AASHTO LRFD Specifications take a different approach including:

- A requirement to detail the truss so as to make secondary force effects as small as practical.
- A requirement to include the bending caused by member self-weight, as well as moments resulting from eccentricities of joint or working lines.
- Relief from including both secondary force effects from joint rotation and floorbeam deflection if the component being designed is more than ten times as long as it is wide in the plane of bending.

When the working lines through the centroids of intersecting members do not intersect at the joint, or where sway frames and portals are eliminated for economic or esthetic purposes, the state of bending in the truss members, as well as the rigidity of the entire system, should be evaluated by a more rigorous analysis than the conventional.

The attachment of floorbeams to truss verticals produces out-of-plane stresses, which should be investigated in highway bridges and must be accounted for in railroad bridges, due to the relatively heavier live load in that type of bridge. An analysis of a frame composed of a floorbeam and all the truss members present in the cross section containing the floor beam is usually adequate to quantify this effect.

Deflection of trusses occurs whenever there are changes in length of the truss members. These changes may be due to strains resulting from loads on the truss, temperature variations, or fabrication effects or errors. Methods of computing deflections are similar in all three cases. Prior to the introduction of computers, calculation of deflections in trusses was a laborious procedure and was usually determined by energy or virtual work methods or by graphical or semigraphical methods, such as the Williot-Mohr diagram. With the widespread availability of matrix structural analysis packages, the calculation of deflections and analysis of indeterminant trusses are speedily executed.

(See also Arts. 3.30, 3.31, and 3.34 to 3.39).

### 13.8.2 Analysis for Wind Loads

The areas of trusses exposed to wind normal to their longitudinal axis are computed by multiplying widths of members as seen in elevation by the lengths center to center of intersections. The overlapping areas at intersections are assumed to provide enough surplus to allow for the added areas of gussets. The AREMA Manual specifies that for railway bridges this truss area be multiplied by the number of trusses, on the assumption that the wind strikes each truss fully (except where the leeward trusses are shielded by the floor system). The AASHTO Specifications require that the area of the trusses and floor as seen in elevation be multiplied by a wind pressure that accounts for 1½ times this area being loaded by wind.

The area of the floor should be taken as that seen in elevation, including stringers, deck, railing, and railing pickets.

AREMA specifies that when there is no live load on the structure, the wind pressure should be taken as at least 50 psf, which is equivalent to a wind velocity of about 125 mph. When live load is on the structure, reduced wind pressures are specified for the trusses plus full wind load on the live load: 30 psf on the bridge, which is equivalent to a 97-mph wind, and 300 lb per lin ft on the live load on one track applied 8 ft above the top of rail.

AASHTO SLD Specifications require a wind pressure on the structure of 75 psf. Total force, lb per lin ft, in the plane of the windward chords should be taken as at least 300 and in the plane of the leeward chords, at least 150. When live load is on the structure, these wind pressures can be reduced 70% and combined with a wind force of 100 lb per lin ft on the live load applied 6 ft above the roadway. The AASHTO LFD Specifications do not expressly address wind loads, so the SLD Specifications pertain by default.

Article 3.8 of the AASHTO LRFD Specifications establish wind loads consistent with the format and presentation currently used in meteorology. Wind pressures are related to a base wind velocity, $V_B$, of 100 mph as common in past specifications. If no better information is available, the wind velocity at 30 ft above the ground, $V_{30}$, may be taken as equal to the base wind, $V_B$. The height of 30 ft was selected to exclude ground effects in open terrain. Alternatively, the base wind speed may be taken from Basic Wind Speed Charts available in the literature, or site specific wind surveys may be used to establish $V_{30}$.

At heights above 30 ft, the design wind velocity, $V_{DZ}$, mph, on a structure at a height, $Z$, ft, may be calculated based on characteristic meteorology quantities related to the terrain over which the winds approach as follows. Select the friction velocity, $V_0$, and friction length, $Z_0$, from Table 13.1 Then calculate the velocity from

$$V_{DZ} = 2.5\ V_0 \left(\frac{V_{30}}{V_B}\right) \ln \left(\frac{Z}{Z_0}\right) \tag{13.1}$$

If $V_{30}$ is taken equal to the base wind velocity, $V_B$, then $V_{30}/V_B$ is taken as unity. The correction for structure elevation included in Eq. 13.1, which is based on current meteorological data, replaces the $\frac{1}{7}$ power rule used in the past.

For design, Table 13.2 gives the base pressure, $P_B$, ksf, acting on various structural components for a base wind velocity of 100 mph. The design wind pressure, $P_D$, ksf, for the design wind velocity, $V_{DZ}$, mph, is calculated from

**TABLE 13.1** Basic Wind Parameters

| | Terrain | | |
|---|---|---|---|
| | Open country | Suburban | City |
| $V_0$, mph | 8.20 | 10.9 | 12.0 |
| $Z_0$, ft | 0.23 | 3.28 | 8.20 |

$$P_D = P_B \left(\frac{V_{DZ}}{V_B}\right)^2 \tag{13.2}$$

Additionally, minimum design wind pressures, comparable to those in the AASHTO SLD Specification, are given in the LRFD Specifications.

AASHTO Specifications also require that wind pressure be applied to vehicular live load.

***Wind Analysis.*** Wind analysis is typically carried out with the aid of computers with a space truss and some frame members as a model. It is helpful, and instructive, to employ a simplified, noncomputer method of analysis to compare with the computer solution to expose major modeling errors that are possible with space models. Such a simplified method is presented in the following.

***Idealized Wind-Stress Analysis of a through Truss with Inclined End Posts.*** The wind loads computed as indicated above are applied as concentrated loads at the panel points.

A through truss with parallel chords may be considered as having reactions to the top lateral bracing system only at the main portals. The effect of intermediate sway frames, therefore, is ignored. The analysis is applied to the bracing and to the truss members.

The lateral bracing members in each panel are designed for the maximum shear in the panel resulting from treating the wind load as a moving load; that is, as many panels are loaded as necessary to produce maximum shear in that panel. In design of the top-chord bracing members, the wind load, without live load, usually governs. The span for top-chord bracing is from hip joint to hip joint. For the bottom-chord members, the reduced wind pressure usually governs because of the considerable additional force that usually results from wind on the live load.

For large trusses, wind stress in the trusses should be computed for both the maximum wind pressure without live load and for the reduced wind pressure with live load and full wind on the live load. Because wind on the live load introduces an effect of ‘‘transfer,’’ as

**TABLE 13.2** Base Pressures, $P_B$ for Base Wind Velocity, $V_B$, of 100 mph

| Structural component | Windward load, ksf | Leeward load, ksf |
|---|---|---|
| Trusses, Columns, and Arches | 0.050 | 0.025 |
| Beams | 0.050 | NA |
| Large Flat Surfaces | 0.040 | NA |

described later, the following discussion is for the more general case of a truss with the reduced wind pressure on the structure and with wind on the live load applied 8 ft above the top of rail, or 6 ft above the deck.

The effect of wind on the trusses may be considered to consist of three additive parts:

- **Chord stresses** in the fully loaded top and bottom lateral trusses.
- **Horizontal component,** which is a uniform force of tension in one truss bottom chord and compression in the other bottom chord, resulting from transfer of the top lateral end reactions down the end portals. This may be taken as the top lateral end reaction times the horizontal distance from the hip joint to the point of contraflexure divided by the spacing between main trusses. It is often conservatively assumed that this point of contraflexure is at the end of span, and, thus, the top lateral end reaction is multiplied by the panel length, divided by the spacing between main trusses. Note that this convenient assumption does not apply to the design of portals themselves.
- **Transfer stresses** created by the moment of wind on the live load and wind on the floor. This moment is taken about the plane of the bottom lateral system. The wind force on live load and wind force on the floor in a panel length is multiplied by the height of application above the bracing plane and divided by the distance center to center of trusses to arrive at a total vertical panel load. This load is applied downward at each panel point of the leeward truss and upward at each panel point of the windward truss. The resulting stresses in the main vertical trusses are then computed.

The total wind stress in any main truss member is arrived at by adding all three effects: chord stresses in the lateral systems, horizontal component, and transfer stresses.

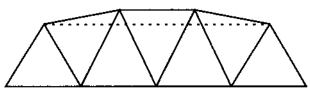


**FIGURE 13.5** Top chord in a horizontal plane approximates a curved top chord.

Although this discussion applies to a parallel-chord truss, the same method may be applied with only slight error to a truss with curved top chord by considering the top chord to lie in a horizontal plane between hip joints, as shown in Fig. 13.5. The nature of this error will be described in the following.

***Wind Stress Analysis of Curved-Chord Cantilever Truss.*** The additional effects that should be considered in curved-chord trusses are those of the vertical components of the inclined bracing members. These effects may be illustrated by the behavior of a typical cantilever bridge, several panels of which are shown in Fig. 13.6.

As transverse forces are applied to the curved top lateral system, the transverse shear creates stresses in the top lateral bracing members. The longitudinal and vertical components of these bracing stresses create wind stresses in the top chords and other members of the main trusses. The effects of these numerous components of the lateral members may be determined by the following simple method:

- Apply the lateral panel loads to the *horizontal projection* of the top-chord lateral system and compute all *horizontal components* of the chord stresses. The stresses in the inclined chords may readily be computed from these horizontal components.

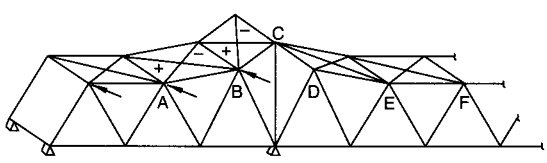



**FIGURE 13.6** Wind on a cantilever truss with curved top chord is resisted by the top lateral system.

- Determine at every point of slope change in the top chord all the vertical forces acting on the point from both bracing diagonals and bracing chords. Compute the truss stresses in the vertical main trusses from those forces.
- The final truss stresses are the sum of the two contributions above and also of any transfer stress, and of any horizontal component delivered by the portals to the bottom chords.

### 13.8.3 Computer Determination of Wind Stresses

For computer analysis, the structural model is a three-dimensional framework composed of all the load-carrying members. Floorbeams are included if they are part of the bracing system or are essential for the stability of the structural model.

All wind-load concentrations are applied to the framework at braced points. Because the wind loads on the floor system and on the live load do not lie in a plane of bracing, these loads must be ‘‘transferred’’ to a plane of bracing. The accompanying vertical required for equilibrium also should be applied to the framework.

Inasmuch as significant wind moments are produced in open-framed portal members of the truss, flexural rigidity of the main-truss members in the portal is essential for stability. Unless the other framework members are released for moment, the computer analysis will report small moments in most members of the truss.

With cantilever trusses, it is a common practice to analyze the suspended span by itself and then apply the reactions to a second analysis of the anchor and cantilever arms.

Some consideration of the rotational stiffness of piers about their vertical axis is warranted for those piers that support bearings that are fixed against longitudinal translation. Such piers will be subjected to a moment resulting from the longitudinal forces induced by lateral loads. If the stiffness (or flexibility) of the piers is not taken into account, the sense and magnitude of chord forces may be incorrectly determined.

### 13.8.4 Wind-Induced Vibration of Truss Members

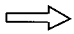
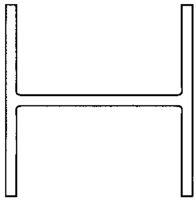
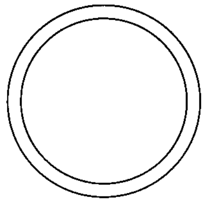
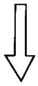
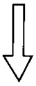
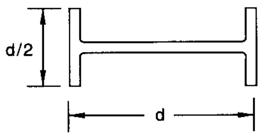
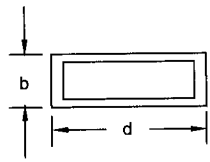
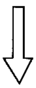
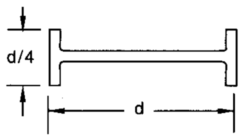
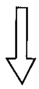
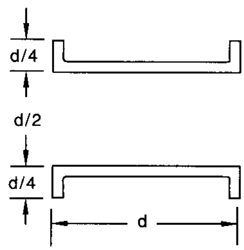
When a steady wind passes by an obstruction, the pressure gradient along the obstruction causes eddies or vortices to form in the wind stream. These occur at stagnation points located on opposite sides of the obstruction. As a vortex grows, it eventually reaches a size that cannot be tolerated by the wind stream and is torn loose and carried along in the wind stream. The vortex at the opposite stagnation point then grows until it is shed. The result is a pattern of essentially equally spaced (for small distances downwind of the obstruction) and alternating vortices called the ‘‘Vortex Street’’ or ‘‘von Karman Trail.’’ This vortex street is indicative of a pulsating periodic pressure change applied to the obstruction. The frequency of the vortex shedding and, hence, the frequency of the pulsating pressure, is given by

$$f = \frac{VS}{D} \tag{13.3}$$

where $V$ is the wind speed, fps, $D$ is a characteristic dimension, ft, and $S$ is the Strouhal number, the ratio of velocity of vibration of the obstruction to the wind velocity (Table 13.3).

When the obstruction is a member of a truss, self-exciting oscillations of the member in the direction perpendicular to the wind stream may result when the frequency of vortex shedding coincides with a natural frequency of the member. Thus, determination of the torsional frequency and bending frequency in the plane perpendicular to the wind and substitution of those frequencies into Eq. (13.3) leads to an estimate of wind speeds at which resonance may occur. Such vibration has led to fatigue cracking of some truss and arch members, particularly cable hangers and I-shaped members. The preceding proposed use of Eq. (13.3) is oriented toward guiding designers in providing sufficient stiffness to reasonably

**TABLE 13.3** Strouhal Number for Various Sections*

| Wind direction | Profile | Strouhal number $S$ | Profile | Strouhal number $S$ |
|---|---|---|---|---|
|  |  | 0.120 |  | 0.200 |
|  | | 0.137 | | |
|  |  | 0.144 |  | |
|  |  | 0.145 | $b/d$ | |
| | | | 2.5 | 0.060 |
| | | | 2.0 | 0.080 |
| | | | 1.5 | 0.103 |
| | | | 1.0 | 0.133 |
|  |  | 0.147 | 0.7 | 0.136 |
| | | | 0.5 | 0.138 |

*As given in "Wind Forces on Structures," *Transactions,* vol. 126, part II, p. 1180, American Society of Civil Engineers.

preclude vibrations. It does not directly compute the amplitude of vibration and, hence, it does not directly lead to determination of vibratory stresses. Solutions for amplitude are available in the literature. See, for example, M. Paz, "Structural Dynamics Theory and Computation," Van Nostrand Reinhold, New York; R. J. Melosh and H. A. Smith, "New Formulation for Vibration Analysis," *ASCE Journal of Engineering Mechanics,* vol. 115, no. 3, March 1989.

C. C. Ulstrup, in "Natural Frequencies of Axially Loaded Bridge Members," *ASCE Journal of the Structural Division,* 1978, proposed the following approximate formula for estimating bending and torsional frequencies for members whose shear center and centroid coincide:

$$f_n = \frac{a}{2\pi}\left(\frac{k_n L}{l}\right)^2 \left[1 + \epsilon_p \left(\frac{KL}{\pi}\right)^2\right]^{1/2} \tag{13.4}$$

where $f_n$ = natural frequency of member for each mode corresponding to $n = 1, 2, 3, \ldots$
$k_nL$ = eigenvalue for each mode (see Table 13.4)
$K$ = effective length factor (see Table 13.4)
$L$ = length of the member, in
$I$ = moment of inertia, in$^4$, of the member cross section
$a$ = coefficient dependent on the physical properties of the member
$= \sqrt{EIg/\gamma A}$ for bending
$= \sqrt{EC_w g/\gamma I_p}$ for torsion
$\epsilon_p$ = coefficient dependent on the physical properties of the member
$= P/EI$ for bending
$= \dfrac{GJA + PI_p}{AEC_w}$ for torsion
$E$ = Young's modulus of elasticity, psi
$G$ = shear modulus of elasticity, psi
$\gamma$ = weight density of member, lb/in$^3$
$g$ = gravitational acceleration, in/s$^2$
$P$ = axial force (tension is positive), lb
$A$ = area of member cross section, in$^2$
$C_w$ = warping constant
$J$ = torsion constant
$I_p$ = polar moment of inertia, in$^4$

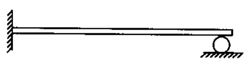
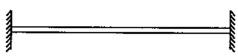
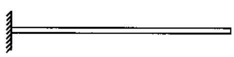
In design of a truss member, the frequency of vortex shedding for the section is set equal to the bending and torsional frequency and the resulting equation is solved for the wind speed $V$. This is the wind speed at which resonance occurs. The design should be such that $V$ exceeds by a reasonable margin the velocity at which the wind is expected to occur uniformly.

## 13.9 TRUSS MEMBER DETAILS

The following shapes for truss members are typically considered:

**H sections,** made with two side segments (composed of angles or plates) with solid web, perforated web, or web of stay plates and lacing. Modern bridges almost exclusively use H sections made of three plates welded together.

**TABLE 13.4** Eigenvalue $k_nL$ and Effective Length Factor $K$

| | $k_nL$ | | | $K$ | | |
|---|---|---|---|---|---|---|
| Support condition | $n = 1$ | $n = 2$ | $n = 3$ | $n = 1$ | $n = 2$ | $n = 3$ |
|  | $\pi$ | $2\pi$ | $3\pi$ | 1.000 | 0.500 | 0.333 |
|  | 3.927 | 7.069 | 10.210 | 0.700 | 0.412 | 0.292 |
|  | 4.730 | 7.853 | 10.996 | 0.500 | 0.350 | 0.259 |
|  | 1.875 | 4.694 | 7.855 | 2.000 | 0.667 | 0.400 |

**Channel sections,** made with two angle segments, with solid web, perforated web, or web of stay plates and lacing. These are seldom used on modern bridges.

**Single box sections,** made with side channels, beams, angles and plates, or side segments of plates only. The side elements may be connected top and bottom with solid plates, perforated plates, or stay plates and lacing. Alternatively, they may be connected at the top with solid cover plates and at the bottom with perforated plates, or stay plates and lacing. Modern bridges use primarily four-plate welded box members. The cover plates are usually solid, except for access holes for bolting joints.

**Double box sections,** made with side channels, beams, angles and plates, or side segments of plates only. The side elements may be connected together with top and bottom perforated cover plates, or stay plates and lacing.

To obtain economy in member design, it is important to vary the area of steel in accordance with variations in total loads on the members. The variation in cross section plus the use of appropriate-strength grades of steel permit designers to use essentially the weight of steel actually required for the load on each panel, thus assuring an economical design.

With respect to shop fabrication of welded members, the H shape usually is the most economical section. It requires four fillet welds and no expensive edge preparation. Requirements for elimination of vortex shedding, however, may offset some of the inherent economy of this shape.

Box shapes generally offer greater resistance to vibration due to wind, to buckling in compression, and to torsion, but require greater care in selection of welding details. For example, various types of welded cover-plate details for boxes considered in design of the second Greater New Orleans Bridge and reviewed with several fabricators resulted in the observations in Table 13.5.

Additional welds placed inside a box member for development of the cover plate within the connection to the gusset plate are classified as AASHTO category E at the termination of the inside welds and should be not be used. For development of the cover plate within the gusset-plate connection, groove welds, large fillet welds, large gusset plates, or a combination of the last two should be used.

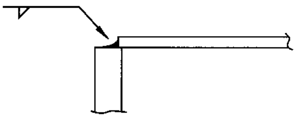
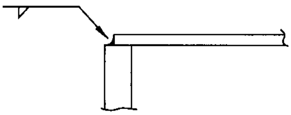
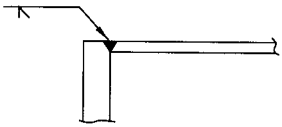
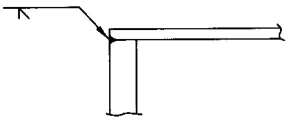
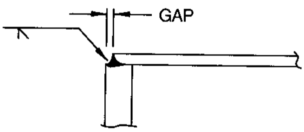
***Tension Members.*** Where practical, these should be arranged so that there will be no bending in the members from eccentricity of the connections. If this is possible, then the total stress can be considered uniform across the entire net area of the member. At a joint, the greatest practical proportion of the member surface area should be connected to the gusset or other splice material.

Designers have a choice of a large variety of sections suitable for tension members, although box and H-shaped members are typically used. The choice will be influenced by the proposed type of fabrication and range of areas required for tension members. The design should be adjusted to take full advantage of the selected type. For example, welded plates are economical for tubular or box-shaped members. Structural tubing is available with almost 22 in$^2$ of cross-sectional area and might be advantageous in welded trusses of moderate spans. For longer spans, box-shape members can be shop-fabricated with almost unlimited areas.

Tension members for bolted trusses involve additional considerations. For example, only 50% of the unconnected leg of an angle or tee is commonly considered effective, because of the eccentricity of the connection to the gusset plate at each end.

To minimize the loss of section for fastener holes and to connect into as large a proportion of the member surface area as practical, it is desirable to use a staggered fastener pattern. In Fig. 13.7, which shows a plate with staggered holes, the net width along Chain 1-1 equals plate width $W$, minus three hole diameters. The net width along Chain 2-2 equals $W$, minus five hole diameters, plus the quantity $S^2/4g$ for each off four gages, where $S$ is the pitch and $g$ the gage.

**TABLE 13.5** Various Welded Cover-Plate Designs for Second Greater New Orleans Bridge

| | |
|---|---|
|  | Conventional detail. Has been used extensively in the past. It may be susceptible to lamellar tearing under lateral or torsional loads. |
|  | Overlap increases for thicker web plate. Cover plate tends to curve up after welding. |
|  | Very difficult to hold out-to-out dimension of webs due to thickness tolerance of the web plates. Groove weld is expensive, but easier to develop cover plate within the connection to gusset plate. |
|  | The detail requires a wide cover plate and tight tolerance of the cover-plate width. With a large overlap, the cover may curve up after welding. Groove weld is expensive, but easier to develop cover plate within the connection to the gusset plate. |
|   | Same as above, except the fabrication tolerance, which will be better with this detail. |

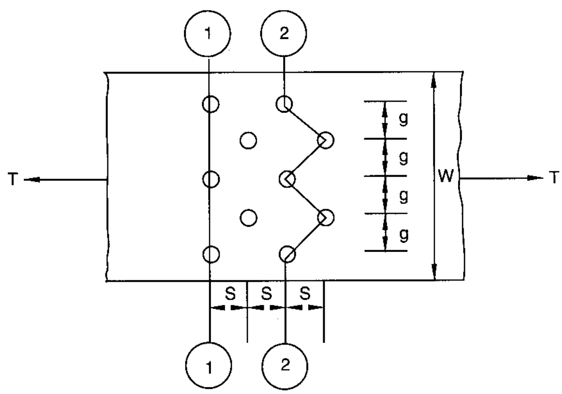



**FIGURE 13.7** Chains of bolt holes used for determining the net section of a tension member.

***Compression Members.*** These should be arranged to avoid bending in the member from eccentricity of connections. Though the members may contain fastener holes, the gross area may be used in design of such columns, on the assumption that the body of the fastener fills the hole. Welded box and H-shaped members are typically used for compression members in trusses.

Compression members should be so designed that the main elements of the section are connected directly to gusset plates, pins, or other members. It is desirable that member components be connected by solid webs. Care should be taken to ensure that the criteria for slenderness ratios, plate buckling, and fastener spacing are satisfied.

***Posts and Hangers.*** These are the vertical members in truss bridges. A post in a Warren deck truss delivers the load from the floorbeam to the lower chord. A hanger in a Warren through-truss delivers the floorbeam load to the upper chord.

Posts are designed as compression members. The posts in a single-truss span are generally made identical. At joints, overall dimensions of posts have to be compatible with those of the top and bottom chords to make a proper connection at the joint.

Hangers are designed as tension members. Although wire ropes or steel rods could be used, they would be objectionable for esthetic reasons. Furthermore, to provide a slenderness ratio small enough to maintain wind vibration within acceptable limits will generally require rope or rod area larger than that needed for strength.

***Truss-Member Connections.*** Main truss members should be connected with gusset plates and other splice material, although pinned joints may be used where the size of a bolted joint would be prohibitive. To avoid eccentricity, fasteners connecting each member should be symmetrical about the axis of the member. It is desirable that fasteners develop the full capacity of each element of the member. Thickness of a gusset plate should be adequate for resisting shear, direct stress, and flexure at critical sections where these stresses are maximum. Re-entrant cuts should be avoided; however, curves made for appearance are permissible.

## 13.10 MEMBER AND JOINT DESIGN EXAMPLES—LFD AND SLD

Design of a truss member by the AASHTO LFD and SLD Specifications is illustrated in the following examples, The design includes a connection in a Warren truss in which splicing of a truss chord occurs within a joint. Some designers prefer to have the chord run continuously through the joint and be spliced adjacent to the joint. Satisfactory designs can be produced using either approach. Chords of trusses that do not have a diagonal framing into each joint, such as a Warren truss, are usually continuous through joints with a post or hanger. Thus, many of the chord members are usually two panels long. Because of limitations on plate size and length for shipping, handling, or fabrication, it is sometimes necessary, however, to splice the plates within the length of a member. Where this is necessary, common practice is to offset the splices in the plates so that only one plate is spliced at any cross section.

### 13.10.1 Load-Factor Design of Truss Chord

A chord of a truss is to be designed to withstand a factored compression load of 7,878 kips and a factored tensile load of 1,748 kips. Corresponding service loads are 4,422 kips compression and 391 kips tension. The structural steel is to have a specified minimum yield stress of 36 ksi. The member is 46 ft long and the slenderness factor $K$ is to be taken as unity. A preliminary design yields the cross section shown in Fig. 13.8. The section has the following properties:

$$A_g = \text{gross area} = 281 \text{ in}^2$$

$$I_{gx} = \text{gross moment of inertia with respect to } x \text{ axis}$$
$$= 97{,}770 \text{ in}^4$$

$$I_{gy} = \text{gross moment of inertia with respect to } y \text{ axis}$$
$$= 69{,}520 \text{ in}^4$$

$$w = \text{weight per linear foot} = 0.98 \text{ kips}$$

Ten 1¼-in-dia. bolt holes are provided in each web at the section for the connections at joints. The welds joining the cover plates and webs are minimum size, ⅜ in, and are classified as AASHTO fatigue category B.

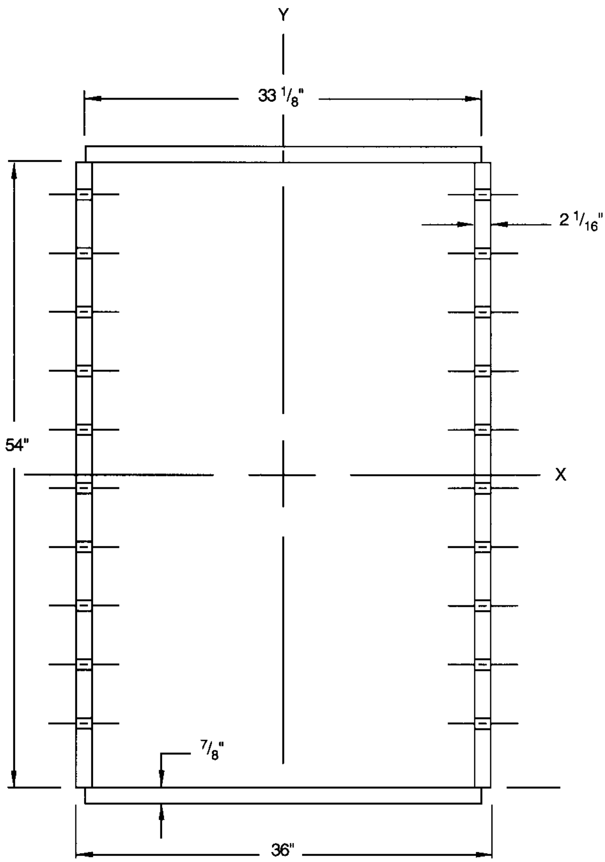



**FIGURE 13.8** Cross section of a truss chord with a box section.

Although the AASHTO LFD Specification specifies a load factor for dead load of 1.30, the following computation uses 1.50 to allow for about 15% additional weight due to paint, diaphragms, weld metal and fasteners.

***Compression in Chord from Factored Loads.*** The uniform stress on the section is

$$f_c = 7878/281 = 28.04 \text{ ksi}$$

The radius of gyration with respect to the weak axis is

$$r_y = \sqrt{I_{gy}/A_g} = \sqrt{69{,}520/281} = 15.73 \text{ in}$$

and the slenderness ratio with respect to that axis is

$$\frac{KL}{r_y} = \frac{1 \times 46 \times 12}{15.73} = 35 < \left(\sqrt{\frac{2\pi^2 E}{F_y}} = 126\right)$$

where $E$ = modulus of elasticity of the steel = 29,000 ksi. The critical buckling stress in compression is

$$\begin{aligned} F_{cr} &= F_y \left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL}{r_y}\right)^2\right] \\ &= 36\left[1 - \frac{36}{4\pi^2 E}(35)^2\right] = 34.6 \text{ ksi} \end{aligned} \tag{13.5}$$

The maximum strength of a concentrically loaded column is $P_u = A_g f_{cr}$ and

$$f_{cr} = 0.85 F_{cr} = 0.85 \times 34.6 \times 29.42 \text{ ksi}$$

For computation of the bending strength, the sum of the depth-thickness ratios for the web and cover plates is

$$\sum \frac{s}{t} = 2 \times \frac{54}{2.0625} + 2 \times \frac{36 - 2.0625}{0.875} = 129.9$$

The area enclosed by the centerlines of the plates is

$$A = 54.875(36 - 2.0625) = 1{,}862 \text{ in}^2$$

Then, the design bending stress is given by

$$\begin{aligned} F_a &= F_y\left[1 - \frac{0.0641 F_y S_g L\sqrt{\Sigma(s/t)}}{EA\sqrt{I_y}}\right] \\ &= 36\left[1 - \frac{0.0641 \times 36 \times 3{,}507 \times 46 \times 12\sqrt{129.9}}{29{,}000 \times 1862\sqrt{69{,}520}}\right] \\ &= 35.9 \text{ ksi} \end{aligned} \tag{13.6}$$

For the dead load of 0.98 kips/ft, the dead-load factor of 1.50, the 46-ft span, and a factor of 1/10 for continuity in bending, the dead-load bending moment is

$$M_{DL} = 0.98(46)^2 \times 12 \times 1.50/10 = 3733 \text{ kip-in}$$

The section modulus is

$$S_g = I_{gx}/c = 97{,}770/(54/2 + 0.875) = 3507 \text{ in}^3$$

Hence, the maximum compressive bending stress is

$$f_b = M_{DL}/S_g = 3733/3507 = 1.06 \text{ ksi}$$

The plastic section modulus is

$$Z_g = 2(33.125 \times 0.875(54/2 + 0.875/2) + 2 \times 2 \times 2.0625 \times 54/2 \times 54/4 = 4598 \text{ in}^4$$

The ratio of the plastic section modulus to the elastic section modulus is $Z_g/S_g = 4{,}598/3{,}507 = 1.31$.

For combined axial load and bending, the axial force $P$ and moment $M$ must satisfy the following equations:

$$\frac{P}{0.85A_gF_{cr}} + \frac{MC}{M_u(1 - P/A_gF_e)} \leq 1.0 \tag{13.7a}$$

$$\frac{P}{0.85A_gF_y} + \frac{M}{M_p} \leq 1.0 \tag{13.8a}$$

where $M_u$ = maximum strength, kip-in, in bending alone
$= S_gf_a$
$M_p$ = full plastic moment, kip-in, of the section
$= ZF_y$
$Z$ = plastic modulus $= 1.31S_g$
$C$ = equivalent moment factor, taken as 0.85 in this case
$F_e$ = Euler buckling stress, ksi, with 0.85 factor $= 0.85E\pi^2/(KL/r_x)^2$

The effective length factor $K$ is taken equal to unity and the radius of gyration $r_x$ with respect to the $x$ axis, the axis of bending, is

$$r_x = \sqrt{I_g/A_g} = \sqrt{97{,}770/281} = 18.65 \text{ in}$$

The slenderness ratio $KL/r_x$ then is $46 \times 12/18.65 = 29.60$.

$$F_e = 0.85 \times 29{,}000\pi^2/29.60^2 = 278 \text{ ksi}$$

For convenience of calculation, Eq. (13.7*a*) can be rewritten, for $P = A_gF_c$, $0.85F_{cr} = f_{cr}$, $M = S_gf_b$, and $M_u = S_gF_a$, as

$$\frac{f_c}{f_{cr}} + \frac{f_b}{F_a}\cdot\frac{C}{1 - P/A_gF_e} \leq 1.0 \tag{13.7b}$$

Substitution of previously calculated stress values in Eq. (13.7*b*) yields

$$\frac{28.04}{29.42} + \frac{1.06}{35.9}\cdot\frac{0.85}{1 - 7878/(281 \times 278)} = 0.953 + 0.028$$

$$= 0.981 \leq 1.0$$

Similarly, Eq. (13.8*a*) can be rewritten as

$$\frac{f_c}{0.85F_y} + \frac{f_b}{F_y Z / S_g} \leq 1.0 \tag{13.8b}$$

Substitution of previously calculated stress values in Eq. (13.8*b*) yields

$$\frac{28.04}{0.85 \times 36} + \frac{1.06}{36 \times 1.31} = 0.916 + 0.022 = 0.938 \leq 1.0$$

The sum of the ratios, 0.981, governs (stability) and is satisfactory. The section is satisfactory for compression.

***Local Buckling.*** The AASHTO specifications limit the depth-thickness ratio of the webs to a maximum of

$$d/t = 180/\sqrt{f_c} = 180/\sqrt{28.04} = 34.0$$

The actual $d/t$ is $54/2.0625 = 26.2 < 34.0$—OK

Maximum permissible width-thickness ratio for the cover plates is

$$b/t = 213.4/\sqrt{f_c} = 213.4/\sqrt{28.04} = 40.3$$

The actual $b/t$ is $33.125/0.875 = 37.9 < 40.3$—OK

***Tension in Chord from Factored Loads.*** The following treatment is based on a composite of AASHTO SLD Specifications for the capacity of tension members, and other aspects from the AASHTO LFD Specifications. This is done because the AASHTO LFD Specifications have not been updated. Clearly, this is not in complete compliance with the AASHTO LFD Specifications. Based on the above, the tensile capacity will be the lesser of the yield strength times the design gross area, or 90% of the tensile strength times the net area. Both areas are defined below. For determinations of the design strength of the section, the effect of the bolt holes must be taken into account by deducting the area of the holes from the gross section area to obtain the net section area. Furthermore, the full gross area should not be used if the holes occupy more than 15% of the gross area. When they do, the excess above 15% of the holes not greater than 1-¼ in in diameter, and all of area of larger holes, should be deducted from the gross area to obtain the design gross area. The holes occupy $10 \times 1.25 = 12.50$ in of web-plate length, and 15% of the 54-in plate is 8.10 in. The excess is 4.40 in. Hence, the net area is $A_n = 281 - 12.50 \times 2.0625 = 255$ in$^2$ and the design gross area, $A_{DG} = 281 - 2 \times 4.40 \times 2.0625 = 263$ in$^2$. The tensile capacity is the lesser of $0.90 \times 255 \times 58 = 13{,}311$ kips or $263 \times 36 = 9{,}468$ kips. Thus, the design gross section capacity controls and the tensile capacity is 9,468 kips.

For computation of design gross moment of inertia, assume that the excess is due to 4 bolts, located 7 and 14 in on both sides of the neutral axis in bending about the $x$ axis. Equivalent diameter of each hole is $4.40/4 = 1.10$ in. The deduction from the gross moment of inertia $I_g = 97{,}770$ in$^4$ then is

$$I_d = 2 \times 2 \times 1.10 \times 2.0625(7^2 + 14^2) = 2220 \text{ in}^4$$

Hence, the design gross moment of inertia $I_{DG}$ is $97{,}770 - 2{,}220 = 95{,}550$ in$^4$, and the design gross elastic section modulus is

$$S_{DG} = \frac{95{,}550}{54/2 + 0.875} = 3428 \text{ in}^3$$

The stress on the design gross section for the axial tension load of 1,748 kips alone is

$$f_t = 1748/263 = 6.65 \text{ ksi}$$

The bending stress due to $M_{DL} = 3733$ kip-in, computed previously, is

$$f_b = 3733/3428 = 1.09 \text{ ksi}$$

For combined axial tension and bending, the sum of the ratios of required strength to design strength is

$$\frac{P}{P_u} + \frac{M}{M_p} = \frac{6.65}{36} + \frac{1.09}{36 \times 1.31} = 0.208 < 1\text{—OK}$$

The section is satisfactory for tension.

***Fatigue at Welds.*** Fatigue is to be investigated for the truss as a nonredundant path structure subjected to 500,000 cycles of loading. The category B welds between web plates and cover plates have an allowable stress range of 23 ksi. Maximum service loads on the chord are 391 kips tension and 4,422 kips compression. The stress range then is

$$f_{sr} = \frac{391 - (-4{,}422)}{281} = 17.1 \text{ ksi} < 23 \text{ ksi}$$

The section is satisfactory for fatigue.

### 13.10.2 Service-Load Design of Truss Chord

The truss chord designed in Art. 13.10.1 by load-factor design and with the cross section shown in Fig. 13.8 is designed for service loads in the following, for illustrative purposes. Properties of the section are given in Art 13.10.1.

***Compression in Chord for Service Loads.*** The uniform stress in the section for the 4,422-kip load on the gross area $A_g = 281$ in$^2$ is

$$f_c = 4422/281 = 15.74 \text{ ksi}$$

The AASHTO standard specifications give the following formula for the allowable axial stress for $F_y = 36$ ksi:

$$F_a = 16.98 - 0.00053(KL/r_y)^2 \tag{13.9}$$

For the slenderness ratio $KL/r_y = 35$, determined in Art. 13.10.1, the allowable stress then is

$$F_a = 16.98 - 0.00053(35)^2 = 16.33 \text{ ksi} > 15.74 \text{ ksi—OK}$$

The allowable bending stress is $f_b = 20$ ksi. Due to the 0.98 kips/ft weight of the 46-ft-long chord, the dead-load bending moment with a continuity factor of $1/10$ is

$$M_{DL} = 0.98(46)^2 \times 12/10 = 2488 \text{ kip-in}$$

For the section modulus $S_{gx} = 97{,}770/27.875 = 3507$ in$^3$, the dead-load bending stress is

$$f_b = 2488/3507 = 0.709 \text{ ksi}$$

For combined bending and compression, AASHTO specifications require that the following interaction formula be satisfied:

$$\frac{f_c}{F_a} + \frac{f_b}{F_b} \cdot \frac{C_m}{1 - f_c/F'_e} \tag{13.10}$$

The coefficient $C_m$ is taken as 0.85 for the condition of transverse loading on a compression member with joint translation prevented. For bending about the $x$ axis, with a slenderness ratio of $KL/r_x = 29.60$, as determined in Art. 13.10.1, the Euler buckling stress with a 2.12 safety factor is

$$F'_e = \frac{\pi^2 E}{2.12(KL/r_x)^2} = \frac{\pi^2 \times 29{,}000}{2.12(29.60)^2} = 154 \text{ ksi}$$

Substitution of the preceding stresses in Eq. (13.10) yields

$$\frac{15.74}{16.33} + \frac{0.709}{20} \cdot \frac{0.85}{1 - 15.74/154} = 0.964 + 0.034 = 0.998 < 1\text{—OK}$$

The section is satisfactory for compression.

***Tension in Chord from Service Loads.*** The section shown in Fig. 13.8 has to withstand a tension load of 391 kips on the net area of 263 in$^2$ computed in Art. 13.10.1. It was determined in Art. 13.10.1 that the capacity was controlled by the design gross section, and while SLD allowable stresses are 0.50 $F_u$ on the net section and 0.55 $F_y$ on the design gross section, the same conclusion is reached here. The allowable tensile stress $F_t$ is 20 ksi. The uniform tension stress on the design gross section is

$$f_t = 391/263 = 1.49 \text{ ksi}$$

As computed in Art. 13.10.1, the moment of inertia of the design gross section is 95,550 in$^4$ and the corresponding section modulus in $S_n = 3{,}428$ in$^3$. Also, as computed previously for compression in the chord, the dead-load bending moment $M_{DL} = 2{,}488$ kip-in. Hence, the maximum bending stress is

$$f_b = 2488/3428 = 0.726 \text{ ksi}$$

The allowable bending stress $F_b$ is 20 ksi.

For combined axial tension and bending, the sum of the ratios of actual stress to allowable stress is

$$\frac{f_t}{F_t} + \frac{f_b}{F_b} = \frac{1.49}{20} + \frac{0.726}{20} = 0.075 + 0.036 = 0.111 < 1\text{—OK}$$

The section is satisfactory for tension.

***Fatigue Design.*** See Art. 13.10.1.

## 13.11 MEMBER DESIGN EXAMPLE—LRFD

The design of a truss hanger by the AASHTO LRFD Specifications is presented subsequently. This is preceded by the following introduction to the LRFD member design provisions.

### 13.11.1 LRFD Member Design Provisions

***Tension Members.*** The net area, $A_n$, of a member is the sum of the products of thickness and the smallest net width of each element. The width of each standard bolt hole is taken as the nominal diameter of the bolt plus 0.125 in. The width deducted for oversize and slotted holes, where permitted in AASHTO LRFD Art. 6.13.2.4.1, is taken as 0.125 in greater than the hole size specified in AASHTO LRFD Art. 6.13.2.4.2. The net width is determined for each chain of holes extending across the member along any transverse, diagonal, or zigzag line, as discussed in Art. 13.9.

In designing a tension member, it is conservative and convenient to use the least net width for any chain together with the full tensile force in the member. It is sometimes possible to achieve an acceptable, but slightly less conservative design, by checking each possible chain with a tensile force obtained by subtracting the force removed by each bolt ahead of that chain (bolt closer to midlength of the member), from the full tensile force in the member. This approach assumes that the full force is transferred equally by all bolts at one end.

Members and splices subjected to axial tension must be investigated for two conditions: yielding on the gross section (Eq. 13.11), and fracture on the net section (Eq. 13.12). Determination of the net section requires consideration of the following:

- The gross area from which deductions will be made, or reduction factors applied, as appropriate
- Deductions for all holes in the design cross-section
- Correction of the bolt hole deductions for the stagger rule
- Application of a reduction factor $U$, to account for shear lag
- Application of an 85% maximum area efficiency factor for splice plates and other splicing elements

The factored tensile resistance, $P_r$, is the lesser of the values given by Eqs. 13.11 and 13.12.

$$P_r = \varphi_y P_{ny} = \varphi_y F_y A_g \tag{13.11}$$

$$P_r = \varphi_u P_{nu} = \varphi_y F_u A_n U \tag{13.12}$$

where $P_{ny}$ = nominal tensile resistance for yielding in gross section (kip)
$F_y$ = yield strength (ksi)
$A_g$ = gross cross-sectional area of the member (in$^2$)
$P_{nu}$ = nominal tensile resistance for fracture in net section (kip)
$F_u$ = tensile strength (ksi)
$A_n$ = net area of the member as described above (in$^2$)
$U$ = reduction factor to account for shear lag; 1.0 for components in which force effects are transmitted to all elements; as described below for other cases
$\varphi_y$ = resistance factor for yielding of tension members, 0.95
$\varphi_u$ = resistance factor for fracture of tension members, 0.80

The reduction factor, $U$, does not apply when checking yielding on the gross section because yielding tends to equalize the non-uniform tensile stresses over the cross section caused by shear lag.

Unless a more refined analysis or physical tests are utilized to determine shear lag effects, the reduction factors specified in the AASHTO LRFD Specifications may be used to account for shear lag in connections as explained in the following.

The reduction factor, $U$, for sections subjected to a tension load transmitted directly to each of the cross-sectional elements by bolts or welds may be taken as:

$$U = 1.0 \tag{13.13}$$

For bolted connections, the following three values of $U$ may be used depending on the details of the connection:

For rolled I-shapes with flange widths not less than two-thirds the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than three fasteners per line in the direction of stress,

$$U = 0.90 \tag{13.14a}$$

For all other members having no fewer than three fasteners per line in the direction of stress,

$$U = 0.85 \tag{13.14b}$$

For all members having only two fasteners per line in the direction of stress,

$$U = 0.75 \tag{13.14c}$$

Due to strain hardening, a ductile steel loaded in axial tension can resist a force greater than the product of its gross area and its yield strength prior to fracture. However, excessive elongation due to uncontrolled yielding of gross area not only marks the limit of usefulness, it can precipitate failure of the structural system of which it is a part. Depending on the ratio of net area to gross area and the mechanical properties of the steel, the component can fracture by failure of the net area at a load smaller than that required to yield the gross area. General yielding of the gross area and fracture of the net area both constitute measures of component strength. The relative values of the resistance factors for yielding and fracture reflect the different reliability indices deemed proper for the two modes.

The part of the component occupied by the net area at fastener holes generally has a negligible length relative to the total length of the member. As a result, the strain hardening is quickly reached and, therefore, yielding of the net area at fastener holes does not constitute a strength limit of practical significance, except, perhaps, for some built-up members of unusual proportions.

For welded connections, $A_n$ is the gross section less any access holes in the connection region.

***Compression Members.*** Bridge members in axial compression are generally proportioned with width/thickness ratios such that the yield point can be reached before the onset of local buckling. For such members, the nominal compressive resistance, $P_n$, is taken as:

$$\text{If } \lambda \leq 2.25, \text{ then } P_n = 0.66^{\lambda} F_y A_s \tag{13.15}$$

$$\text{If } \lambda > 2.25, \text{ then } P_n = \frac{0.88 F_y A_s}{\lambda} \tag{13.16}$$

for which:

$$\lambda = \left(\frac{Kl}{r_s \pi}\right)^2 \frac{F_y}{E} \tag{13.17}$$

where $A_s$ = gross cross-sectional area (in$^2$)
$F_y$ = yield strength (ksi)
$E$ = modulus of elasticity (ksi)
$K$ = effective length factor
$l$ = unbraced length (in)
$r_s$ = radius of gyration about the plane of buckling (in)

To avoid premature local buckling, the width-to-thickness ratios of plate elements for compression members must satisfy the following relationship:

$$\frac{b}{t} \leq k\sqrt{\frac{E}{F_y}} \tag{13.18}$$

where $k$ = plate buckling coefficient, $b$ = plate width (in), and $t$ = thickness (in). See Table 13.6 for values for $k$ and descriptions of $b$.

**TABLE 13.6** Values of $k$ for Calculating Limiting Width-Thickness Ratios

| Element | Coefficient, $k$ | Width, $b$ |
|---|---|---|
| *a. Plates supported along one edge* | | |
| Flanges and projecting legs or plates | 0.56 | Half-flange width of I-sections.<br>Full-flange width of channels.<br>Distance between free edge and first line of bolts or weld in plates.<br>Full-width of an outstanding leg for pairs of angles in continuous contact. |
| Stems of rolled tees | 0.75 | Full-depth of tee. |
| Other projecting elements | 0.45 | Full-width of outstanding leg for single angle strut or double angle strut with separator.<br>Full projecting width for others |
| *b. Plates supported along two edges* | | |
| Box flanges and cover plates | 1.40 | Clear distance between webs minus inside corner radius on each side for box flanges.<br>Distance between lines of welds or bolts for flange cover plates. |
| Webs and other plate elements | 1.49 | Clear distance between flanges minus fillet radii for webs of rolled beams.<br>Clear distance between edge supports for all others. |
| Perforated cover plates | 1.86 | Clear distance between edge supports. |

***Source:*** Adapted from *AASHTO LRFD Bridge Design Specification,* American Association of State Highway and Transporation Officials, 444 North Capital St., N.W., Ste. 249, Washington, DC 20001.

***Members Under Tension and Flexure.*** A component subjected to tension and flexure must satisfy the following interaction equations:

$$\text{If } \frac{P_u}{P_r} < 0.2, \text{ then} \qquad (13.19)$$
$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

$$\text{If } \frac{P_u}{P_r} \geq 0.2, \text{ then} \qquad (13.20)$$
$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

where $P_r$ = factored tensile resistance (kip)
$M_{rx}$, $M_{ry}$ = factored flexural resistances about the $x$ and $y$ axes, respectively (k-in)
$M_{ux}$, $M_{uy}$ = moments about $x$ and $y$ axes, respectively, resulting from factored loads (k-in)
$P_u$ = axial force effect resulting from factored loads (kip)

Interaction equations in tension and compression members are a design simplification. Such equations involving exponents of 1.0 on the moment ratios are usually conservative. More exact, nonlinear interaction curves are also available and are discussed in the literature. If these interaction equations are used, additional investigation of service limit state stresses is necessary to avoid premature yielding.

A flange or other component subjected to a net compressive stress due to tension and flexure should also be investigated for local buckling.

***Members Under Compression and Flexure.*** For a component subjected to compression and flexure, the axial compressive load, $P_u$, and the moments, $M_{ux}$ and $M_{uy}$, are determined for concurrent factored loadings by elastic analytical procedures. The following relationships must be satisfied:

$$\text{If } \frac{P_u}{P_r} < 0.2, \text{ then} \qquad (13.21)$$
$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

$$\text{If } \frac{P_u}{P_r} \geq 0.2, \text{ then} \qquad (13.22)$$
$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

where $P_r$ = factored compressive resistance, $\varphi P_n$ (kip)
$M_{rx}$ = factored flexural resistance about the $x$ axis (k-in)
$M_{ry}$ = factored flexural resistance about the $y$ axis (k-in)
$M_{ux}$ = factored flexural moment about the $x$ axis calculated as specified below (k-in)
$M_{uy}$ = factored flexural moment about the $y$ axis calculated as specified below (k-in)
$\varphi$ = resistance factor for compression members

The moments about the axes of symmetry, $M_{ux}$ and $M_{uy}$, may be determined by either (1) a second order elastic analysis that accounts for the magnification of moment caused by the factored axial load, or (2) the approximate single step adjustment specified in AASHTO LRFD Art. 4.5.3.2.2b.

**TABLE 13.7** Unfactored Design Loads

| Load component | Axial tension load, $P$, kN | Bending moment, $M_x$, kN-m | Bending moment, $M_y$, kN-m |
|---|---|---|---|
| Dead load of structural components, $DC$ | 1344 | 0 | −9.01 |
| Dead load of wearing surfaces and utilities, $DW$ | 149 | 0 | −1.07 |
| Truck live load per lane, $LL_{TR}$ | 32.9 | 0 | 35.8 |
| Lane live load per lane, $LL_{LA}$ | 82.4 | 0 | 90.0 |
| Fatigue live load, $LL_{FA}$ | 44.0, −1.10 | 0 | 15.0, −4.40 |

## 13.11.2 LRFD Design of Truss Hanger

The following example, prepared in the SI system of units, illustrates the design of a tensile member that also supports a primary live load bending moment. The existence of the bending moment is not common in truss members, but can result from unusual framing. In this example, the bending moment serves to illustrate the application of various provisions of the LRFD Specifications.

A fabricated H-shaped hanger member is subjected to the unfactored design loads listed in Table 13.7. The applicable AASHTO load factors for the Strength-I Limit State and the Fatigue Limit State are listed in Table 13.8. The impact factor, I, is 1.15 for the fatigue limit state and 1.33 for all other limit states.

For the overall bridge cross section, the governing live load condition places three lanes of live load on the structure with a distribution factor, $DF$, of 2.04 and a multiple presence factor, $MPF$, of 0.85. For the fatigue limit state, the placement of the single fatigue truck produces a distribution factor of 0.743. The multiple presence factor is not applied to the fatigue limit state.

The factored force effect, $Q$, in the member is calculated for the axial force and the moment in Table 13.7 from the following equation to obtain the factored member load and moment:

**TABLE 13.8** AASHTO Load Factors

| Type of factor | Strength-I limit state* | Fatigue limit state |
|---|---|---|
| Ductility, $\eta_D$ | 1.00 | 1.0 |
| Redundancy, $\eta_R$ | 1.05 | 1.0 |
| Importance, $\eta_I$ | 1.05 | 1.0 |
| $\eta = \eta_D \eta_R \eta_I$** | 1.10 | 1.0 |
| Dead load, $\gamma_{DC}$ | 1.25/0.90 | — |
| Dead load, $\gamma_{DW}$ | 1.50/0.65 | — |
| Live load + impact, $LL + I$ | 1.75 | 0.75 |

*Basic load combination relating to normal vehicular use of bridge without wind.
** $\eta \geq 0.95$ for loads for which a maximum load factor is appropriate; $1/\eta \leq 1.10$ for loads for which a minimum load factor is appropriate.

**TABLE 13.9** Factored Design Loads (Nominal Force Effects)

| Limit state | Axial tension load, $P_u$, kN | Bending moment, $M_{ux}$, kN-m | Bending moment, $M_{uy}$, kN-m |
|---|---|---|---|
| Strength-I | 2515 | 0 | 450 |
| Fatigue | 28.2, −0.70 | 0 | 9.61, −2.82 |

$$Q = \eta[\gamma_{DC}DC + \gamma_{DW}DW + \gamma_{LL+I}(DF)(MPF)(LL_{TR} * I + LL_{LA})] \tag{13.23}$$

where *DF* is the distribution factor, *MPF* is the multiple presence factor, *I* is the impact factor, and the other terms are defined in Tables 13.7 and 13.8. For example, for the axial load, $Q$ is calculated as follows:

$$Q = 1.10[1.25 * 1344 + 1.50 * 149 + 1.75(2.04)(0.85)(32.9 * 1.33 + 82.4)]$$
$$= 2515 \text{ kN}$$

Table 13.9 summarizes the nominal force effects for the member.

The preliminary section selected is shown in Fig. 13.9. The member length is 20 m, the yield stress 345 MPa, the tensile strength 450 MPa, and the diameter of A325 bolts is 24 mm. Section properties are listed in Table 13.10.

***Tensile Resistance.*** The tensile resistance is calculated as the lesser of Eqs. 13.11 and 13.12. From Eq. 13.11, gross section yielding, $P_r = 0.95 \times 345 \times 26{,}456/1000 = 8671$ kN. From Eq. 13.12, net section fracture, assuming the force effects are transmitted to all components so that $U = 1.00$, $P_r = 0.80 \times 450 \times 20{,}072/1000 = 7226$ kN. Thus, net section fracture controls and $P_r = 7226$ kN.

***Flexural Resistance.*** Because net section fracture controls, use net section properties for calculating flexural resistance. Also, because $M_x = 0$, only investigate weak axis bending. The nominal moment strength, $M_n$, is defined by AASHTO in this case as the plastic moment. Thus, for an H-section about the weak axis, in terms of the yield stress, $F_y$, and section modulus, $S$,

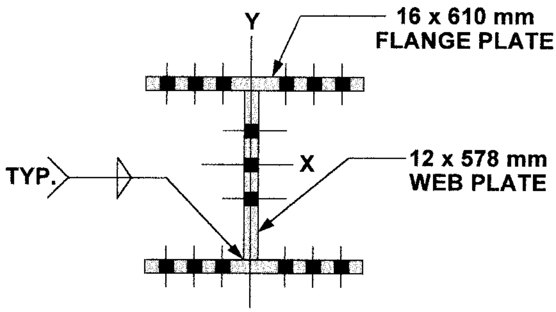



**FIGURE 13.9** Cross section of H-shaped hanger.

**TABLE 13.10** Section Properties for Example Problem

| | | |
|---|---|---|
| Area | $A_g$ | 26,456 mm$^2$ |
| | $A_n$ | 20,0772 mm$^2$ |
| Moment of Inertia | $I_{xg}$ | $1.92 \times 10^9$ mm$^4$ |
| | $I_{xn}$ | $1.44 \times 10^9$ mm$^4$ |
| | $I_{yg}$ | $6.05 \times 10^8$ mm$^4$ |
| | $I_{yn}$ | $4.56 \times 10^8$ mm$^4$ |
| Section Modulus | $S_{xg}$ | $6.30 \times 10^6$ mm$^3$ |
| | $S_{xn}$ | $4.71 \times 10^6$ mm$^3$ |
| | $S_{yg}$ | $1.98 \times 10^6$ mm$^3$ |
| | $S_{yn}$ | $1.49 \times 10^6$ mm$^3$ |

$$M_{ny} = 1.5F_yS \tag{13.24}$$

Substituting $y$-axis values, $M_{ny} = 1.5 \times 345 \times 1.49 \times 10^6/1000^2 = 771$ kN-m. The factored flexural resistance, $M_r$ is defined as

$$M_r = \varphi_f M_n \tag{13.24a}$$

where $\varphi_f$ is the resistance factor for flexure (1.00). Therefore, in this case, $M_{ry} = 1.00\ M_{ny} = 771$ kN-m.

***Combined Tension and Flexure.*** This will be checked for the Strength-I limit state using the nominal force effects listed in Table 13.9. First calculate $P_u/P_r = 2515/7226 = 0.348$. Because this exceeds 0.2, Eq. 13.20 applies. Substitute appropriate values as follows:

$$\frac{2515}{7226} + \frac{8}{9}\left(0 + \frac{450}{771}\right) = 0.87 \leq 1.00 \qquad \text{OK}$$

***Slenderness Ratio.*** AASHTO requires that tension members other than rods, eyebars, cables and plates satisfy certain slenderness ratio ($l/r$) requirements. For main members subject to stress reversal, $l/r \leq 140$. If the present case the least radius of gyration is $r = \sqrt{I_{yg}/A_g} = \sqrt{6.05 * 10^8/26{,}456} = 151$ mm and $l/r = 20{,}000/151 = 132$. This is within the limit of 140.

***Fatigue Limit State.*** The member is fabricated from plates with continuous fillet welds parallel to the applied stress. Slip-critical bolts are used for the end connections. Both of these are category B fatigue details. The average daily truck traffic, ADTT, is 2250 and three lanes are available to trucks. The number of trucks per day in a single-lane, averaged over the design life is calculated from the AASHTO expression,

$$ADTT_{SL} = p * ADTT \tag{13.25}$$

where $p$ is the fraction of truck traffic in a single lane as follows: 1.00 for 1 truck lane, 0.85 for two truck lanes, and 0.80 for three or more truck lanes. Therefore, $ADTT_{SL} = 0.80 * 2250 = 1800$. The nominal fatigue resistance is calculated as a maximum permissible stress range as follows:

$$\Delta F = \left(\frac{A}{N}\right)^{1/3} \geq \frac{1}{2}(\Delta F)_{TH} \tag{13.26}$$

where

$$N = (365)(75)(n)(ADTT_{SL}) \tag{13.27}$$

In the above, $A$ is a fatigue constant that varies with the fatigue detail category, $n$ is the number of stress range cycles per truck, and $(\Delta F)_{TH}$ is the constant amplitude fatigue threshold. These constants are found in the AASHTO LRFD Specification for the present case as follows: $A = 39.3 * 10^{11}$ MPa$^3$, $n = 1.0$, and $(\Delta F)_{TH} = 110$ MPa. Substitute in Eq. 13.26:

$$\Delta F = \left(\frac{39.3 * 10^{11}}{365 * 75 * 1.0 * 1800}\right)^{1/3} = 43.0 \text{ MPa and } \frac{1}{2}\,(\Delta F)_{TH} = 55 \text{ MPa}$$

Therefore, $\Delta F = 55$ MPa. Next calculate the stress range for the force effects in Table 13.9. For the web-to-flange welds, which lie near the neutral axis, only the axial load is considered, and net section properties are used as the worst case:

$$\frac{28.2 - (-0.70)}{20{,}072} * 1000 = 1.44 \text{ MPa} < 55 \text{ MPa} \qquad \text{OK}$$

For the extreme fiber at the slip-critical connections, both axial load and flexure is considered, and gross section properties are used:

$$\frac{28.2 - (-0.70)}{26{,}456} * 10^3 + \frac{9.61 - (-2.82)}{1.98 * 10^6} * 10^6 = 7.37 \text{ MPa} < 55 \text{ MPa} \qquad \text{OK}$$

Thus, fatigue does not control and the member selection is satisfactory. A separate check shows that the bolts are also adequate.

## 13.12 TRUSS JOINT DESIGN PROCEDURE

At every joint in a truss, working lines of the intersecting members preferably should meet at a point to avoid eccentric loading (Art. 13.2). While the members may be welded directly to each other, most frequently they are connected to each other by bolting to gusset plates. Angle members may be bolted to a single gusset plate, whereas box and H shapes may be bolted to a pair of gusset plates.

A gusset plate usually is a one-piece element. When necessary, it may be spliced with groove welds. When the free edges of the plate will be subjected to compression, they usually are stiffened with plates or angles. Consideration should be given in design to the possibility of the stresses in gusset plates during erection being opposite in sense to the stresses that will be imposed by service loads.

Gusset plates are sometimes designed by the *method of sections* based on conventional strength of materials theory. The method of sections involves investigation of stresses on various planes through a plate and truss members. Analysis of gusset plates by finite-element methods, however, may be advisable where unusual geometry exists.

Transfer of member forces into and out of a gusset plate invokes the potential for block shear around the connector groups and is assumed to have about a 30° angle of distribution with respect to the gage line, as illustrated in Fig. 13.10 (line 1-5 and 4-6).

The following summarizes a procedure for load-factor design of a truss joint. Splices are assumed to occur within the truss joints. (See examples in Arts. 13.13 and 13.14.) The concept employed in the procedure can also be applied to working-stress design.

1. Lay out the centerlines of truss members to an appropriate scale and the members to a scale of ½ in = 1 ft, with gage lines.
2. Detail the fixed parts, such as floorbeam, strut, and lateral connections.
3. Determine the grade and size of bolts to be used.

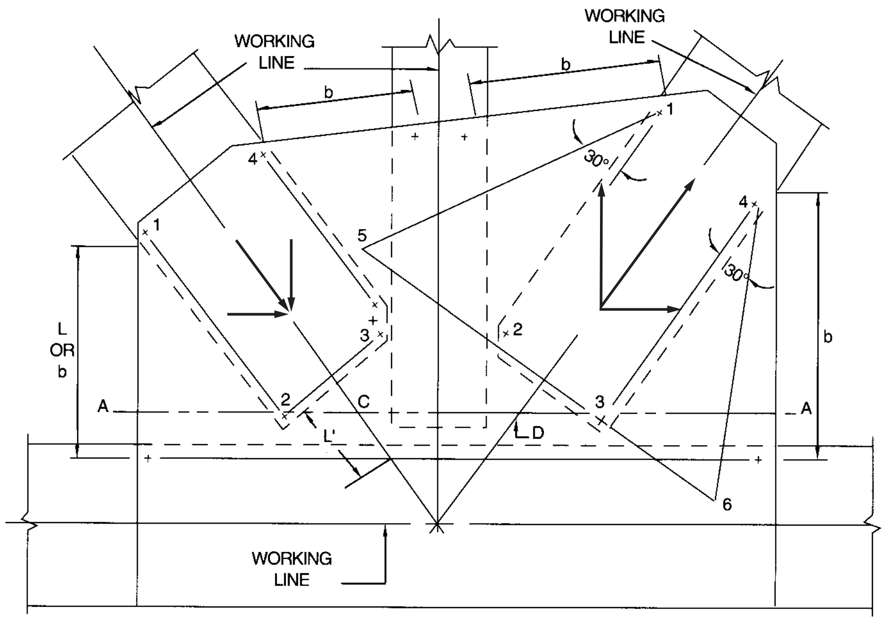



**FIGURE 13.10** Typical design sections for a gusset plate.

4. Detail the end connections of truss diagonals. The connections should be designed for the average of the design strength of the diagonals and the factored load they carry but not less than 75% of the design strength. The design strength should be taken as the smallest of the following: (*a*) member strength, (*b*) column capacity, and (*c*) strength based on the width-thickness ratio $b/t$. A diagonal should have at least the major portion of its ends normal to the working line (square) so that milling across the ends will permit placing of templates for bolt-hole alignment accurately. The corners of the diagonal should be as close as possible to the cover plates of the chord and verticals. Bolts for connection to a gusset plate should be centered about the neutral axis of the member.
5. Design fillet welds connecting a flange plate of a welded box member to the web plates, or the web plate of an H member to the flange plates, to transfer the connection load from the flange plate into the web plates over the length of the gusset connection. Weld lengths should be designed to satisfy fatigue requirements. The weld size should be shown on the plans if the size required for loads or fatigue is larger than the minimum size allowed.
6. Avoid the need for fills between gusset plates and welded-box truss members by keeping the out-to-out dimension of web plates and the in-to-in dimension of gusset plates constant.
7. Determine gusset-plate outlines. This step is influenced principally by the diagonal connections.
8. Select a gusset-plate thickness $t$ to satisfy the following criteria, as illustrated in Fig. 13.10:
   *a.* The loads for which a diagonal is connected may be resolved into components parallel to and normal to line *A-A* in Fig. 13.10 (horizontal and vertical). A shearing stress is induced along the gross section of line *A-A* through the last line of bolts. Equal to the sum of the horizontal components of the diagonals (if they act in the same

direction), this stress should not exceed $F_y/1.35\sqrt{3}$, where $F_y$ is the yield stress of the steel, ksi.

***b.*** A compression stress is induced in the edge of the gusset plate along Section *A-A* (Fig. 13.10) by the vertical components of the diagonals (applied at *C* and *D*) and the connection load of the vertical or floorbeam, when compressive. The compression stress should not exceed the permissible column stress for the unsupported length of the gusset plate (*L* or *b* in Fig. 13.10). A stiffening angle should be provided if the slenderness ratio $L/r = L\sqrt{12}/t$ of the compression edge exceeds 120, or if the permissible column stress is exceeded. The $L/r$ of the section formed by the angle plus a 12-in width of the gusset plate should be used to recheck that $L/r \leq 120$ and the permissible column stress is not exceeded. In addition to checking the $L/r$ of the gusset in compression, the width-thickness ratio $b/t$ of every free edge should be checked to ensure that it does not exceed $348/\sqrt{F_y}$.

***c.*** At a diagonal (Fig. 13.10),

$$V_1 + V_2 \geq P_d \tag{13.28}$$

where $P_d$ = load from the diagonal, kips
$V_1$ = shear strength, kips, along lines 1-2 and 3-4
$= A_g F_y/\sqrt{3}$
$A_g$ = gross area, in$^2$, along those lines
$V_2$ = strength, kips, along line 2-3 based on $A_n F_y$ for tension diagonals or $A_g F_a$ for compression diagonals
$A_n$ = net area, in$^2$, of the section
$F_a$ = allowable compressive stress, ksi

The distance $L'$ in Fig. 13.10 is used to compute $F_a$ for sections 2-3 and 5-6.

***d.*** Assume that the connection stress transmitted to the gusset plate by a diagonal spreads over the plate within lines that diverge outward at 30° to the axis of the member from the first bolt in each exterior row of bolts, as indicated by path 1-5-6-4 (on the right in Fig. 13.10). Then, the stress on the section normal to the axis of the diagonal at the last row of bolts (along line 5-6) and included between these diverging lines should not exceed $F_y$ on the net-section for tension diagonals and $F_a$ for compression diagonals.

**9.** Design the chord splice (at the joint) for the full capacity of the chords. Arrange the gusset plates and additional splice material to balance, as much as practical, the segment being spliced.

**10.** When the chord splice is to be made with a web splice plate on the inside of a box member (Fig. 13.11), provide extra bolts between the chords and the gusset on each side of the inner splice plate when the joint lies along the centerline of the floorbeam. This should be done because in the diaphragm bolts at floorbeam connections deliver some floorbeam reaction across the chords. When a splice plate is installed on the outer side of the gusset, back of the floorbeam connection angles (Fig. 13.11), the entire group of floorbeam bolts will be stressed, both vertically and horizontally, and should not be counted as splice bolts.

**11.** Determine the size of standard perforations and the distances from the ends of the member.

## 13.13 EXAMPLE—LOAD FACTOR DESIGN OF TRUSS JOINT

The joint shown in Fig. 13.11 is to be designed to satisfy the criteria listed in Table 13.11. Fasteners to be used are 1⅛-in-dia. A325 high-strength bolts in a slip-critical connection

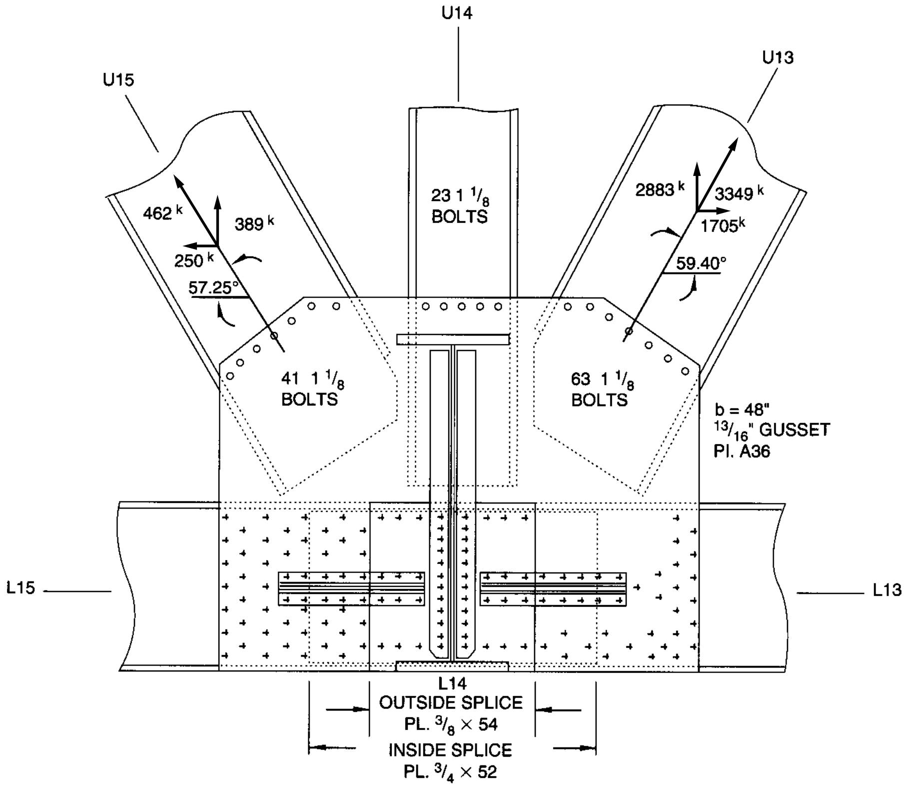



**FIGURE 13.11** Truss joint for example of load-factor design.

**TABLE 13.11** Allowable Stresses for Truss Joint, ksi*

| Design section | Yield stress of steel, ksi: 36 | 50 |
|---|---|---|
| Shear on line *A-A* | 15.4 | 21.4 |
| Shear on lines 1-2 and 3-4 | 20.8 | 28.9 |
| Tension on lines 2-3 and 5-5 | 36.0 | 50.0 |

*Figs. 13.10 and 13.11.

with Class A surfaces, with an allowable shear stress $F_v = 15.5$ ksi assume 16 ksi for this example. The bolts connecting a diagonal or vertical to a gusset plate then have a shear capacity, kips, for service loads

$$P_v = NA_vF_v = 16\text{NA}_v \tag{13.29}$$

where $N$ = number of bolts and $A_v$ = cross-sectional area of a bolt, in$^2$. For load-factor design, $P_v$ is multiplied by a load factor. For example, for Group I loading,

$$1.5[D + (4/3)(L + I)] = 1.5(1 + R/3)P_v \tag{13.30}$$

where $R$ = ratio of live load $L$ to the total service load. Hence, for this loading, and load factor is $1.5(1 + R/3)$.

***Diagonal U15-L14.*** The diagonal is subjected to factored loads of 2,219 kips compression and 462 kips tension. It has a design strength of 2,379 kips. The AASHTO SLD Specifications require that the connection to the gusset plate transmit 75% of the design strength or the average of the factored load and the design strength, whichever is larger. Thus, the design load for the connection is

$$P = (2219 + 2379)/2 = 2299 \text{ kips} > 0.75 \times 2379$$

The ratio of the service live load to the total service load for the diagonal is $R = 0.55$. Hence, for Group I loading on the bolts, the load factor is $1.5(1 + R/3) = 1.775$. For service loads, the 1⅛-in-dia. bolts have a capacity of 15.90 kips per shear plane. Therefore, since the member is connected to two gusset plates, the number of bolts required for diagonal U15-L14 is

$$N = \frac{2299}{2 \times 1.775 \times 15.90} = 41 \text{ per side}$$

***Diagonal L14-U13.*** The diagonal is subjected to factored loads of a maximum of 3272 kips tension and a minimum of 650 kips tension. It has a design strength of 3425 kips. The design load for the connection is

$$P = (3272 + 3425)/2 = 3349 \text{ kips} > 0.75 \times 3425$$

The ratio of the service live load to the total service load is $R = 0.374$, and the load factor for the bolts is $1.5(1 + 0.374/3) = 1.687$. Then, the number of 1⅛-in bolts required is

$$N = \frac{3349}{2 \times 1.687 \times 15.90} = 63 \text{ per side}$$

***Vertical U14-L14.*** The vertical carries a factored compression load of 362 kips. It has a design strength of 1439 kips, limited by $b/t$ at a perforation. The design load for the connection is

$$P = 0.75 \times 1439 = 1079 \text{ kips} > (362 + 1439)/2$$

Since the vertical does not carry any live load, the load factor for the bolts is 1.5. Hence, the number of 1⅛-in bolts required for the vertical is

$$N = \frac{1079}{2 \times 1.5 \times 15.90} = 23 \text{ per side}$$

***Splice of Chord Cover Plates.*** Each cover plate of the box chord is to be spliced with a plate on the inner and outer face (Fig. 13.12). A36 steel will be used for the splice material, as for the chord. Fasteners are ⅞-in-dia. A325 bolts, with a capacity for service loads of 9.62 kips per shear plane. The bolt load factor is 1.791.

The cover plate on chord L14-L15 (Fig. 13.11) is $^{13}/_{16} \times 34\frac{3}{4}$ in but has 12-in-wide access perforations. Usable area of the plate is 18.48 in$^2$. The cover plate for chord L13-L14 is $^{13}/_{16} \times 34$ in, also with 12-in-wide access perforations. Usable area of this plate is 17.88 in$^2$. Design of the chord splice is based on the 17.88-in$^2$ area. The difference of 0.60 in$^2$ between this area and that of the larger cover plate will be made up on the L14-L15 side of the web-plate splice as "cover excess."

Where the design section of the joint elements is controlled by allowances for bolts, only the excess exceeding 15% of the gross section area is deducted from the gross area to obtain the design area. (This is the designer's interpretation of the applicable requirements for splices in the AASHTO SLD Specifications. The interpretation is based on the observation that, for the typical dimensions of members, holes, bolt patterns and grades of steel used on the bridge in question, the capacity of tension members was often controlled by the design gross area as illustrated in Arts. 13.10.1 and 13.10.2. The current edition of the specifications should be consulted on this and other interpretations, inasmuch as the specifications are under constant reevaluation.)

The number of bolts needed for a cover-plate splice is

$$N = \frac{17.88 \times 36}{2 \times 1.791 \times 9.62} = 19 \text{ per side}$$

Try two splice plates, each ⅜ × 31 in, with a gross area of 23.26 in$^2$. Assume eight 1-in-dia. bolt holes in the cross section. The area to be deducted for the holes then is

$$2 \times 0.375(8 \times 1 - 0.15 \times 31) = 2.51 \text{ in}^2$$

Consequently, the area of the design net section is

$$A_n = 23.26 - 2.51 = 20.75 \text{ in}^2 > 17.88 \text{ in}^2\text{—OK}$$

***Tension Splice of Chord Web Plate.*** A splice is to be provided between the 1¼ × 54-in web of chord L14-L15 and the 1⅝ × 54-in web of the L13-L14 chord. Because of the difference in web thickness, a ⅜-in fill will be place on the inner face of the 1¼-in web (Fig. 13.13). The gusset plate can serve as part of the needed splice material. The remainder is supplied by a plate on the inner face of the web and a plate on the outer face of the gusset. Fasteners are 1⅛-in-dia. A325 bolts, with a capacity for service loads of 15.90 kips. Load factor is 1.791.

The web of the L13-L14 chord has a gross area of 87.75 in$^2$. After deduction of the 15% excess of seven 1¼-in-dia. bolt holes, the design area of this web is 86.69 in$^2$.

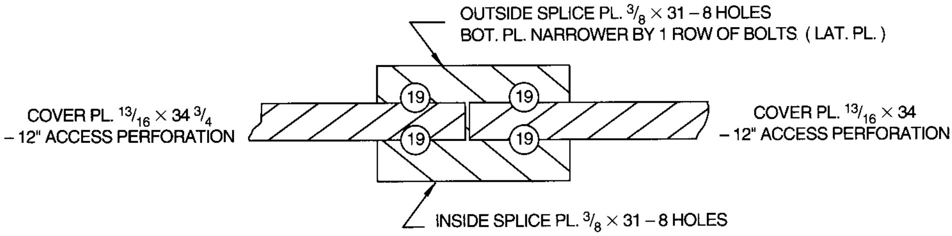



**FIGURE 13.12** Cross section of chord cover-plate splice for example of load-factor design.

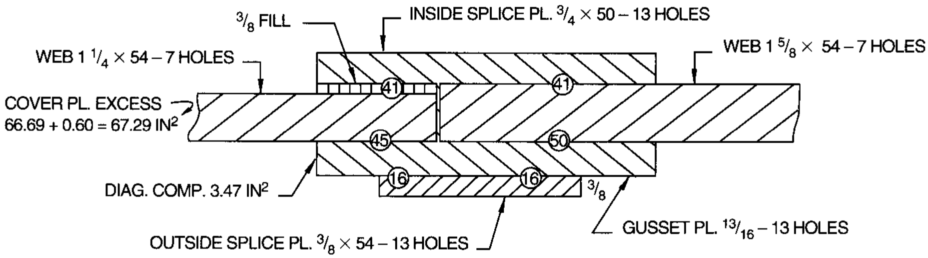



**FIGURE 13.13** Cross section of chord web-plate slice for example of load-factor design.

The web on the L14-L15 chord has a gross area of 67.50 in$^2$. After deduction of the 15% excess of seven bolt holes from the chord splice and addition of the "cover excess" of 0.60 in$^2$, the design area of this web is 67.29 in$^2$.

The gusset plate is $^{13}\!/_{16}$ in thick and 118 in high. Assume that only the portion that overlaps the chord web; that is, 54 in, is effective in the splice. To account for the eccentric application of the chord load to the gusset, an effectiveness factor may be applied to the overlap, with the assumption that only the overlapping portion of the gusset plate is stressed by the chord load.

The **effectiveness factor** $E_f$ is defined as the ratio of the axial stress in the overlap due to the chord load to the sum of the axial stress on the full cross section of the gusset and the moment due to the eccentricity of the chord relative to the gusset centroid.

$$E_f = \frac{P/A_o}{P/A_g + Pey/I} \tag{13.31}$$

where $P$ = chord load
$A_o$ = overlap area = $54t$
$A_g$ = full area of gusset plate = $118t$
$e$ = eccentricity of $P$ = $118/2 - 54/2 = 32$ in
$y$ = $118/2 = 59$ in
$I$ = $118^3t/12 = 136{,}900t$ in$^4$

Substitution in Eq. (13.31) yields

$$E_f = \frac{P/54t}{P/118t + 32 \times 59P/136{,}900t} = 0.832$$

The gross area of the gusset overlap is $^{13}\!/_{16} \times 54 = 43.88$ in$^2$. After deduction of the 15% excess of thirteen 1¼-in-dia. bolt holes, the design area is 37.25 in$^2$. Then, the effective area of the gusset as a splice plate is $0.832 \times 37.25 = 30.99$ in$^2$.

In addition to the 67.29 in$^2$ of web area, the gusset has to supply an area for transmission of the 250-kip horizontal component from diagonal U15-L14 (Fig. 13.11). With $F_y = 36$ ksi, this area equals $250/(36 \times 2) = 3.47$ in$^2$. Hence, the equivalent web area from the L14-L15 side of the joint is $67.29 + 3.47 = 70.76$ in$^2$. The number of bolts required to transfer the load to the inside and outside of the web should be determined based on the effective areas of gusset that add up to 70.76 in$^2$ but that provide a net moment in the joint close to zero.

The sum of the moments of the web components about the centerline of the combination of outside splice plate and gusset plate is $3.47 \times 0.19 + 67.29 \times 1.22 = 0.66 + 82.09 = 82.75$ in$^3$. Dividing this by 2.59 in, the distance to the center of the inside splice plate, yields an effective area for the inside splice plate of 31.95 in$^2$. Hence, the effective area of the combination of the gusset and outside splice plates in $70.76 - 31.95 = 38.81$ in². This is then distributed to the plates in proportion to thickness: gusset, 24.96 in², and splice plate, 13.85 in².

The number of 1⅛-in A325 bolts required to develop a plate with area $A$ is given by

$$N = AF_y/(1.791 \times 15.90) = 36A/28.48 = 1.264A$$

Table 13.12 list the number of bolts for the various plates.

***Check of Gusset Plates.*** At Section *A-A* (Fig. 13.11), each plate is 128 in wide and 118 in high, 13⁄16 in thick. The design shear stress is 15.4 ksi (Table 13.11). The sum of the horizontal components of the loads on the truss diagonals is $1244 + 1705 = 2949$ kips. This produces a shear stress on section *A-A* of

$$f_v = \frac{2{,}949}{2 \times 128 \times 13/16} = 14.18 \text{ ksi} < 15.4 \text{ ksi—OK}$$

The vertical component of diagonal U15-L14 produces a moment about the centroid of the gusset of $1{,}934 \times 21 = 40{,}600$ kip-in and the vertical component of U13-L14 produces a moment $2{,}883 \times 20.5 = 59{,}100$ kip-in. The sum of these moments is $M = 99{,}700$ kip-in. The stress at the edge of one gusset plate due to this moment is

$$f_b = \frac{6M}{td^2} = \frac{6 \times 99{,}700}{2(13/16)128^2} = 22.47 \text{ ksi}$$

The vertical, carrying a 362-kip load, imposes a stress

$$f_c = \frac{P}{A} = \frac{362}{2 \times 128 \times 13/16} = 1.74 \text{ ksi}$$

The total stress then is $f = 22.47 + 1.74 = 24.21$ ksi.

The width $b$ of the gusset at the edge is 48 in. Hence, the width-thickness ratio is $b/t = 48/(13/16) = 59$. From step $b$ in Art. 13.12, the maximum permissible $b/t$ is $348/\sqrt{F_y} = 348/\sqrt{36} = 58 < 59$. The edge has to be stiffened. Use a stiffener angle $3 \times 3 \times ½$ in.

For computation of the design compressive stress, assume the angle acts with a 12-in width of gusset plates. The slenderness ratio of the edge is $48/0.73 = 65.75$. The maximum permissible slenderness ratio is

$$\sqrt{2\pi^2 E/F_y} = \sqrt{2\pi^2 \times 29{,}000/36} = 126 > 65.75$$

Hence, the design compressive stress is

**TABLE 13.12** Number of Bolts for Plate Development

| Plate | Area, in² | Bolts |
|---|---|---|
| Inside splice plate | 31.95 | 41 |
| Outside splice plate | 13.85 | 18 |
| Gusset plate on L14-L15 side | $(13.85 + 24.96 - 3.47) = 35.34$ | 45 |
| Gusset plate on L13-L14 side | $(13.85 + 24.96) = 38.81$ | 50 |

$$f_a = 0.85F_y \left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{L}{r}\right)^2\right] \tag{13.32}$$

$$= 0.85 \times 36 \left[1 - \frac{36}{4\pi^2 \times 29{,}000}\left(\frac{48}{0.73}\right)^2\right]$$

$$= 26.44 \text{ ksi} > 24.21 \text{ ksi—OK}$$

Next, the gusset plate is checked for shear and compression at the connection with diagonal U15-L14. The diagonal carries a factored compression load of 2,299 kips. Shear paths 1-2 and 3-4 (Fig. 13.10) have a gross length of 93 in. From Table 13.11, the design shear stress is 20.8 ksi. Hence, design shear on these paths is

$$V_d = 2 \times 20.8 \times 93 \times 13/16 = 3143 \text{ kips} > 2299 \text{ kips—OK}$$

Path 2-3 need not be investigated for compression. For compression on path 5-6, a 30° distribution from the first bolt in the exterior row is assumed (Art. 13.12, step 8*d*). The length of path 5-6 between the 30° lines in 82 in. The design stress, computed from Eq. (13.32) with a slenderness ratio of 52.9, is 27.9 ksi. The design strength of the gusset plate then is

$$P = 2 \times 27.9 \times 82 \times 13/16 = 3718 \text{ kips} > 2299 \text{ kips—OK}$$

Also, the gusset plate is checked for shear and tension at the connection with diagonal L14-U13. The diagonal carries a tension load of 3,272 kips. Shear paths 1-2 and 3-4 (Fig. 13.10) have a gross length of 98 in. From Table 13.11, the allowable shear stress is 20.8 ksi. Hence, the allowable shear on these paths is

$$V_d = 2 \times 20.8 \times 98 \times 13/16 = 3312 \text{ kips} > 3{,}272 \text{ kips—OK}$$

For path 2-3, capacity in tension with $F_y = 36$ ksi is

$$P_{23} = 2 \times 36 \times 27 \times 13/16 = 1580 \text{ kips}$$

For tension on path 5-6 (Fig. 13.10), a 30° distribution from the first bolt in the exterior row is assumed (Art. 13.12, step 8*d*). The length of path 5-6 between the 30° lines is a net of 83 in. The allowable tension then is

$$P_{56} = 2 \times 36 \times 83 \times 13/16 = 4856 \text{ kips} > 3272 \text{ kips—OK}$$

***Welds to Develop Cover Plates.*** The fillet weld sizes selected are listed in Table 13.13 with their capacities, for an allowable stress of 26.10 ksi. A 5/16-in weld is selected for the diagonals. It has a capacity of 5.76 kips/in.

The allowable compressive stress for diagonal U15-L14 is 22.03 ksi. Then, length of fillet weld required is

$$\frac{22.03(7/8)23\tfrac{1}{8}}{2 \times 5.76} = 38.7 \text{ in}$$

For $F_y = 36$ ksi, the length of fillet weld required for diagonal L14-U13 is

$$\frac{36(1/2)23\tfrac{1}{8}}{2 \times 5.76} = 36.1 \text{ in}$$

**TABLE 13.13** Weld Capacities—Load-Factor Design

| Weld size, in | Capacity of weld, kips per in |
|---|---|
| 5⁄16 | 5.76 |
| 3⁄8 | 6.92 |
| 7⁄16 | 8.07 |
| 1⁄2 | 9.23 |

## 13.14 EXAMPLE—SERVICE-LOAD DESIGN OF TRUSS JOINT

The joint shown in Fig. 13.14 is to be designed for connections with 1⅛-in-dia. A325 bolts with an allowable stress $F_v$ = 16 ksi. Shear capacity of the bolts is 15.90 kips.

***Diagonal U15-L14.*** The diagonal is subjected to loads of 1250 kips compression and 90 kips tension. The connection is designed for 1288 kips, 3% over design load. The number of bolts required for the connection to the 11⁄16-in-thick gusset plate is

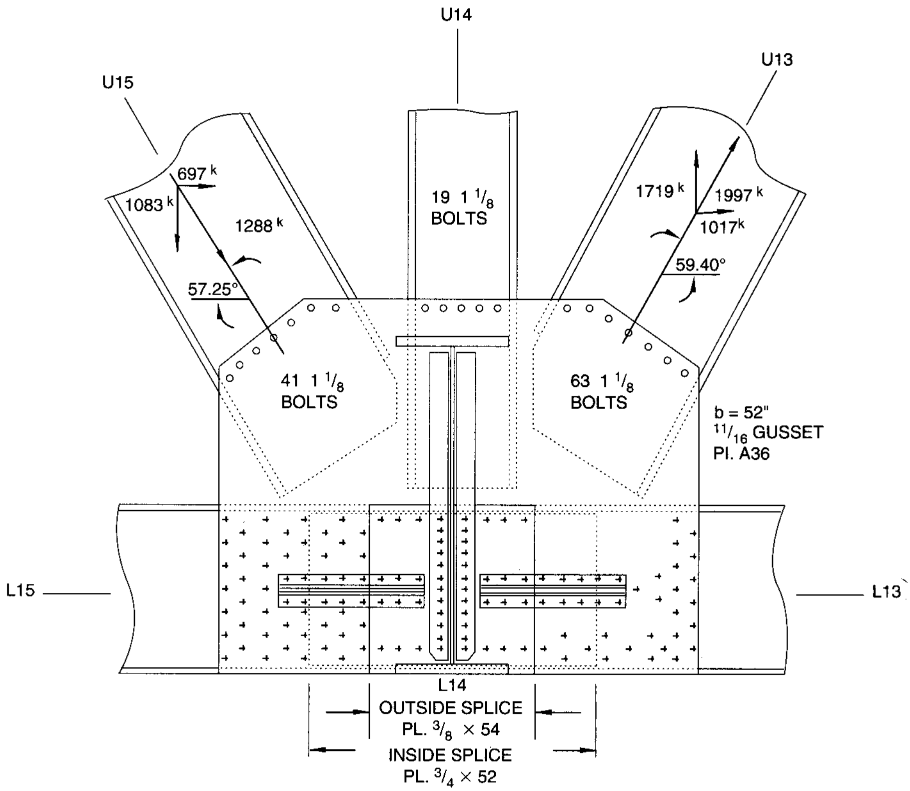



**FIGURE 13.14** Truss joint for example of service-load design.

$$N = 1288/(2 \times 15.90) = 41 \text{ per side}$$

***Diagonal L14-U13.*** The diagonal is subjected to a maximum tension of 1939 kips and a minimum tension of 628 kips. The connection is designed for 1997 kips, 3% over design load. The number of 1⅛-in-dia. A325 bolts required is

$$N = 1997/(2 \times 15.90) = 63 \text{ per side}$$

***Vertical U14-L14.*** The vertical carries a compression load of 241 kips. The member is 74.53 ft long and has a cross-sectional area of 70.69 $\text{in}^2$. It has a radius of gyration $r$ = 10.52 in and slenderness ratio of $KL/r = 74.53 \times 12/10.52 = 85.0$ with $K$ taken as unity. The allowable compression stress then is

$$\begin{aligned} F_a &= 16.98 - 0.00053(KL/r)^2 \\ &= 16.98 - 0.00053 \times 85.0^2 = 13.15 \text{ ksi} \end{aligned} \tag{13.33}$$

The allowable unit stress for width-thickness ratio $b/t$, however, is $11.10 < 13.15$ and governs. Hence, the allowable load is

$$P = 70.69 \times 11.10 = 785 \text{ kips}$$

The number of bolts required is determined for 75% of the allowable load:

$$N = 0.75 \times 785/(2 \times 15.90) = 19 \text{ bolts per side}$$

***Splice of Chord Cover Plates.*** Each cover plate of the box chord is to be spliced with a plate on the inner and outer face (Fig. 13.15). A36 steel will be used for the splice material, as for the chord. Fasteners are ⅞-in-dia. A325 bolts, with a capacity of 9.62 kips per shear plane.

The cover for L14-L15 (Fig. 13.14) is $^{13}\!/_{16}$ by 34¾ in but has 12-in-wide access perforations. Usable area of the plate is 18.48 $\text{in}^2$. The cover plate for L13-L14 is $^{13}\!/_{16} \times 34$ in, also with 12-in-wide access perforations. Usable area of this plate is 17.88 $\text{in}^2$. Design of the chord splice is based on the 17.88-$\text{in}^2$ area. The difference of 0.60 $\text{in}^2$ between this area and that of the larger cover plate will be made up on the L14-L15 side of the web plate splice as "cover excess."

Where the net section of the joint elements is controlled by the allowance for bolts, only the excess exceeding 15% of the gross area is deducted from the gross area to obtain the design gross area, as in load-factor design (Art. 13.13).

For an allowable stress of 20 ksi in the cover plate, the number of bolts needed for the cover-plate splice is

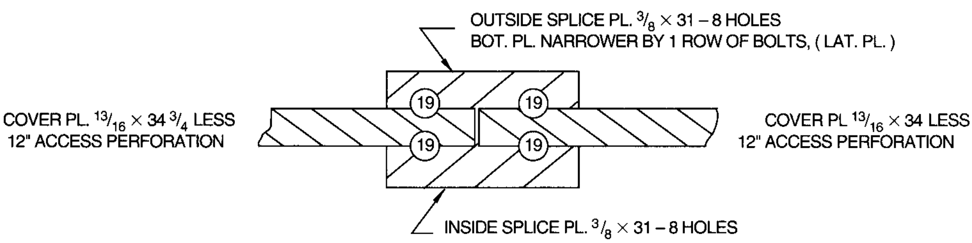



**FIGURE 13.15** Cross section of chord cover-plate splice for example of service-load design.

$$N = \frac{17.88 \times 20}{2 \times 9.62} = 19 \text{ per side}$$

Try two splice plates, each ⅜ × 31 in, with a gross area of 23.26 in$^2$. Assume eight 1-in-dia. bolt holes in the cross section. The area to be deducted for the holes then is

$$2 \times 0.375(8 \times 1 - 0.15 \times 31) = 2.51 \text{ in}^2$$

Consequently, the area of the design gross section is

$$A_n = 23.26 - 2.51 = 20.75 \text{ in}^2 > 17.88 \text{ in}^2\text{—OK}$$

***Splice of Chord Web Plate.*** A splice is to be provided between the 1¼ × 54-in web of chord L14-L15 and the 1⅝ × 54-in web of the L13-L14 chord. Because of the difference in web thickness, a ⅜-in fill will be placed on the inner face of the 1¼-in web (Fig. 13.16). The gusset plate can serve as part of the needed splice material. The remainder is supplied by a plate on the inner face of the web and a plate on the outer face of the gusset. Fasteners are 1⅛-in-dia. A325 bolts, with a capacity of 15.90 kips.

The web of the L13-L14 chord has a design gross area of 87.75 in$^2$. After deduction of the 15% excess of seven 1¼-in-dia. bolt holes, the net area of this web is 86.69 in$^2$.

The web of the L14-L15 chord has a design gross area of 67.50 in$^2$. After deduction of the 15% excess of seven bolt holes from the chord splice and addition of the "cover excess" of 0.60 in$^2$, the net area of this web is 67.29 in$^2$.

The gusset plate is 11⁄16 in thick and 123 in high. Assume that only the portion that overlaps the chord web, that is, 54 in, is effective in the splice. To account for the eccentric application of the chord load to the gusset, an effectiveness factor $E_f$ [Eq. (13.31)] may be applied to the overlap (Art. 13.13). The moment of inertia of the gusset is $123^3t/12 = 155{,}100t$ in$^4$.

$$E_f = \frac{P/54t}{P/123t + P(123/2 - 54/2)(123/2)/155{,}100t} = 0.849$$

The gross area of the gusset overlap is 11⁄16 × 54 = 37.13 in$^2$. After the deduction of the excess of thirteen 1¼-in-dia. bolt holes, the net area is 31.52 in$^2$. Then, the effective area of the gusset as a splice plate is 0.849 × 31.52 = 26.76 in$^2$.

In addition to the 67.29 in$^2$ of web area, the gusset has to supply an area for transmission of the 49-kip horizontal component from diagonal U15-L14. With an allowable stress of 20 ksi, the area is 49/(20 × 2) = 1.23 in$^2$. hence, the equivalent web area from the L14-L15 side of the joint is 67.29 + 1.23 = 68.52 in$^2$. The number of bolts required to transfer the load to the inside and outside of the web should be based on the effective areas of gusset that add up to 68.52 in$^2$ but that provide a net moment in the joint close to zero.

The sum of the moments of the web components about the centerline of the combination of outside splice plate and gusset plate is 1.23 × 0.19 + 67.29 × 1.16 = 78.29 kip-in.

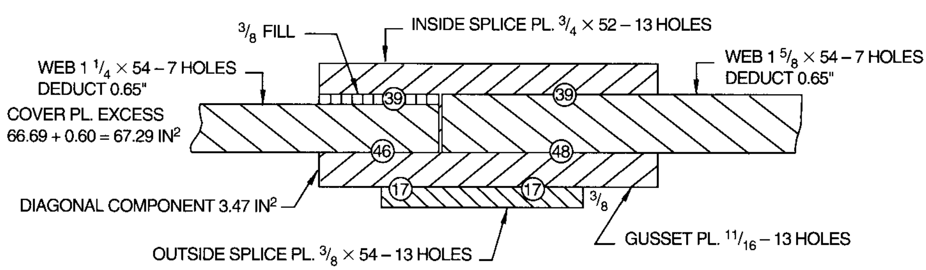



**FIGURE 13.16** Cross section of chord web-plate splice for example of service load design.

Dividing this by 2.53, the distance to the center of the inside splice plate, yields an effective area for the inside splice plate of 30.94 in$^2$. Hence, the effective area of the combination of the gusset and outside splice plates is $68.52 - 30.94 = 37.58$ in$^2$. This is then distributed to the plates as follows: gusset, 22.88 in$^2$, and outside splice plate, 14.70 in$^2$.

The number of 1⅛-in-dia. A325 bolts required to develop a plate with area $A$ and allowable stress of 20 ksi is

$$N = 20A/15.90 = 1.258A$$

Table 13.14 lists the number of bolts for the various plates.

***Check of Gusset Plates.*** At section $A$-$A$ (Fig. 13.11), each plate is 134 in wide and 123 in high, $^{11}\!/_{16}$ in thick. The allowable shear stress is 10 ksi. The sum of the horizontal components of the loads on the truss diagonals is $697 + 1017 = 1714$ kips. This produces a shear stress on Section $A$-$A$ of

$$f_v = \frac{1714}{2 \times 134 \times {}^{11}\!/_{16}} = 9.30 \text{ ksi} < 10 \text{ ksi—OK}$$

The vertical component of diagonal U15-L14 produces a moment about the centroid of the gusset of $1083 \times 21 = 22{,}740$ kip-in and the vertical component of U13-L14 produces a moment $1719 \times 20.5 = 35{,}240$ kip-in. The sum of these moments is 57,980 kip-in. The stress at the edge of one gusset plate due to this moment is

$$f_b = \frac{6M}{td^2} = \frac{6 \times 57{,}980}{2({}^{11}\!/_{16})134^2} = 14.09 \text{ ksi}$$

The vertical carrying a 241-kip load, imposes a stress

$$f_c = \frac{P}{A} = \frac{241}{2 \times 134 \times {}^{11}\!/_{16}} = 1.31 \text{ ksi}$$

The total stress then is $14.09 + 1.31 = 15.40$ ksi

The width $b$ of the gusset at the edge is 52 in. Hence, the width-thickness ratio is $b/t = 52/({}^{11}\!/_{16}) = 75.6$. From step 8$b$ in Art. 13.12, the maximum permissible $b/t$ is $348\sqrt{F_y} = 348/\sqrt{36} = 58 < 75.6$. The edge has to stiffened. Use a stiffener angle $4 \times 3 \times \frac{1}{2}$ in.

For computation of the allowable compressive stress, assume the angle acts with a 12-in width of gusset plate. The slenderness ratio of the edge is $52/1.00 = 52.0$. The maximum permissible slenderness ratio is

$$\sqrt{2\pi^2 E/F_y} = \sqrt{2\pi^2 \times 29{,}000/36} = 126 > 552$$

Hence, the allowable stress from Eq. (13.33) is

**TABLE 13.14** Number of Bolts for Plate Development

| Plate | Area, in$^2$ | Bolts |
|---|---|---|
| Inside splice plate | 30.94 | 39 |
| Outside splice plate | 14.70 | 19 |
| Gusset plate on L14-L15 side | $(14.70 + 22.88 - 1.16) = 36.42$ | 46 |
| Gusset plate on L13-L14 side | $(14.70 + 22.88) = 37.58$ | 48 |

$$F_a = 16.98 - 0.00053 \times 52^2 = 15.55 \text{ ksi} > 15.40 \text{ ksi—OK}$$

Next, the gusset plate is checked for shear and compression at the connection with diagonal U15-L14. The diagonal carries a load of 1,288 kips. Shear paths 1-2 and 3-4 (Fig. 13.10) have a gross length of 105 in. The allowable shear stress is 12 ksi. Hence, the allowable shear on these paths is

$$V_d = 2 \times 12 \times 105 \times {}^{11}\!/_{16} = 1733 \text{ kips} > 1288 \text{ kips—OK}$$

Path 2-3 need not be investigated for compression. For compression on path 5-6, a 30° distribution from the first bolt in the exterior row is assumed (Art. 13.12, step 8*d*). The length of path 5-6 between the 30° lines is 88 in. The allowable stress, computed from Eq. (13.33) with a slenderness ratio $KL/r = 0.5 \times 25/0.198 = 63$, is 14.88 ksi. This permits the gusset to withstand a load

$$P = 2 \times 14.88 \times 88 \times {}^{11}\!/_{16} = 1800 \text{ kips} > 1288 \text{ kips}$$

Also, the gusset plate is checked for shear and tension at the connection with diagonal L14-U13. The diagonal carries a tension load of 1,997 kips. Shear paths 1-2 and 3-4 (Fig. 13.10) have a gross length of 102 in. The allowable shear stress is 12 ksi. Hence, the allowable shear on these paths is

$$V_d = 2 \times 12 \times 102 \times {}^{11}\!/_{16} = 1683 \text{ kips}$$

For path 2-3, capacity in tension with an allowable stress of 20 ksi is

$$P_{23} = 2 \times 20 \times 21.6 \times {}^{11}\!/_{16} = 594 \text{ kips} > (1997 - 1683)\text{—OK}$$

For tension on path 5-6 (Fig. 13.10), a 30° distribution from the first bolt in the exterior row is assumed (Art. 13.12, step 8*d*). The length of path 5-6 between the 30° lines is a net of 88 in. The allowable tension then is

$$P = 2 \times 20 \times 88 \times {}^{11}\!/_{16} = 2420 \text{ kips} > 1997 \text{ kips—OK}$$

***Welds to Develop Cover Plates.*** The fillet weld sizes selected are listed in Table 13.15 with their capacities, for an allowable stress of 15.66 ksi. A 5⁄16-in weld is selected for the diagonals. It has a capacity of 3.46 kips/in.

The allowable compressive stress for diagonal U15-L14 is 11.93 ksi. Then, length of fillet weld required is

**TABLE 13.15** Weld Capacities—Service-Load Design

| Weld size, in | Capacity of weld, kips per in |
|---|---|
| 5⁄16 | 3.46 |
| 3⁄8 | 4.15 |
| 7⁄16 | 4.84 |
| 1⁄2 | 5.54 |

$$\frac{11.93(7/8)23\tfrac{1}{8}}{2 \times 3.46} = 34.9 \text{ in}$$

The allowable tensile stress for diagonal L14-U13 is 20.99 ksi. In this case, the required weld length is

$$\frac{20.99(1/2)23\tfrac{1}{8}}{2 \times 3.46} = 35.1 \text{ in}$$

## 13.15 SKEWED BRIDGES

To reduce scour and to avoid impeding stream flow, it is generally desirable to orient piers with centerlines parallel to direction of flow; therefore skewed spans may be required. Truss construction does not lend itself to bridges where piers are not at right angles to the superstructure (skew crossings). Hence, these should be avoided and this can generally be done by using longer spans with normal piers. In economic comparisons, it is reasonable to assume some increased cost of steel fabrication if skewed trusses are to be used.

If a skewed crossing is a necessity, it is sometimes possible to establish a panel length equal to the skew distance $W \tan \phi$, where $W$ is the distance between trusses and $\phi$ the skew angle. This aligns panels and maintains perpendicular connections of floorbeams to the trusses (Fig. 13.17). If such a layout is possible, there is little difference in cost and skewed spans and normal spans. Design principles are similar. If the skewed distance is less than the panel length, it might be possible to take up the difference in the angle of inclination of the end post, as shown in Fig. 13.17. This keeps the cost down, but results in trusses that are not symmetrical within themselves and, depending on the proportions, could be very unpleasing esthetically. If the skewed distance is greater than the panel length, it may be necessary to vary panel lengths along the bridge. One solution to such a skew is shown in Fig. 13.18, where a truss, similar to the truss in Fig. 13.17, is not symmetrical within itself

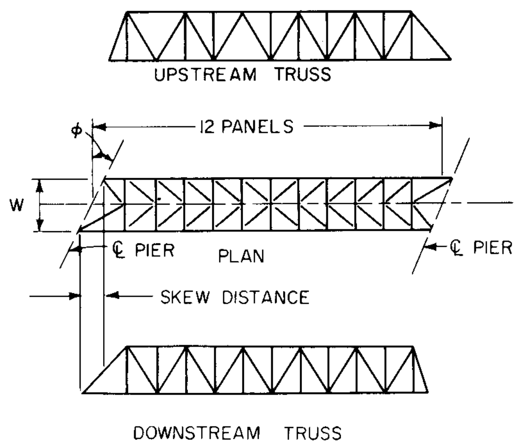



**FIGURE 13.17** Skewed bridge with skew distance less than panel length.

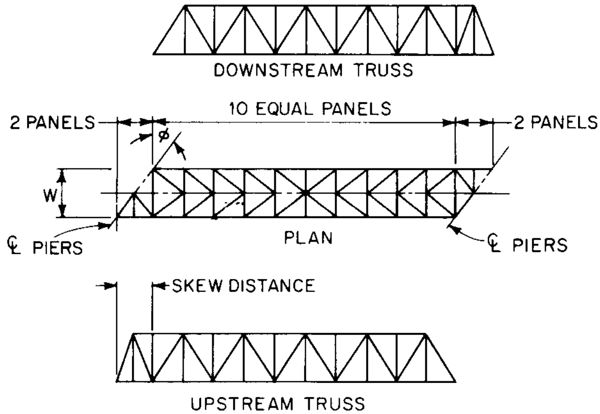



**FIGURE 13.18** Skewed bridge with skew distance exceeding panel length.

and, again, might not be esthetically pleasing. The most desirable solution for skewed bridges is the alternative shown in Fig. 13.17.

Skewed bridges require considerably more analysis than normal ones, because the load distribution is nonuniform. Placement of loads for maximum effect, distribution through the floorbeams, and determination of panel point concentrations are all affected by the skew. Unequal deflections of the trusses require additional checking of sway frames and floor system connections to the trusses.

## 13.16 TRUSS BRIDGES ON CURVES

When it is necessary to locate a truss bridge on a curve, designers should give special consideration to truss spacing, location of bridge centerline, and stresses.

For highway bridges, location of bridge centerline and stresses due to centrifugal force are of special concern. For through trusses, the permissible degree of curvature is limited because the roadway has to be built on a curve, while trusses are planar, constructed on chords. Thus, only a small degree of throw, or offset from a tangent, can be tolerated. Regardless of the type of bridge, horizontal centrifugal forces have to be transmitted through the floor system to the lateral system and then to supports.

For railroad truss bridges, truss spacing usually provides less clearance than the spacing for highway bridges. Thus, designers must take into account tilting of cars due to superelevation and the swing of cars overhanging the track. The centerline of a through-truss bridge on a curve often is located so that the overhang at midspan equals the overhang at each span end. For bridges with more than one truss span, layout studies should be made to determine the best position for the trusses.

Train weight on a bridge on a curve is not centered on the centerline of track. Loads are greater on the outer truss than on the inner truss because the resultant of weight and centrifugal force is closer to the outer truss. Theoretically, the load on each panel point would be different and difficult to determine exactly. Because the difference in loading on inner and outer trusses is small compared with the total load, it is generally adequate to make a simple calculation for a percentage increase to be applied throughout a bridge.

Stress calculations for centrifugal forces are similar to those for any horizontal load. Floorbeams, as well as the lateral system, should be analyzed for these forces.

## 13.17 TRUSS SUPPORTS AND OTHER DETAILS

End bearings transmit the reactions from trusses to substructure elements, such as abutments or piers. Unless trusses are supported on tall slender piers that can deflect horizontally without exerting large forces on the trusses, it is customary to provide expansion bearings at one end of the span and fixed bearings at the other end.

Anchoring a truss to the support, a fixed bearing transmits the longitudinal loads from wind and live-load traction, as well as vertical loads and transverse wind. This bearing also must incorporate a hinge, curved bearing plate, pin arrangement, or elastomeric pads to permit end rotation of the truss in its plane.

An expansion bearing transmits only vertical and transverse loads to the support. It permits changes in length of trusses, as well as end rotation.

Many types of bearings are available. To ensure proper functioning of trusses in accordance with design principles, designers should make a thorough study of the bearings, including allowances for reactions, end rotations and horizontal movements. For short trusses, a rocker may be used for the expansion end of a truss. For long trusses, it generally is necessary to utilize some sort of roller support. See also Arts. 10.22 and 11.9.

***Inspection Walkways.*** An essential part of a truss design is provision of an inspection walkway. Such walkways permit thorough structural inspection and also are of use during erection and painting of bridges. The additional steel required to support a walkway is almost insignificant.

## 13.18 CONTINUOUS TRUSSES

Many river crossings do not require more than one truss span to meet navigational requirements. Nevertheless, continuous trusses have made possible economical bridge designs in many localities. Studies of alternative layouts are essential to ensure selection of the lowest-cost arrangement. The principles outlined in preceding articles of this section are just as applicable to continuous trusses as to simple spans. Analysis of the stresses in the members of continuous trusses, however, is more complex, unless computer-aided design is used. In this latter case, there is no practical difference in the calculation of member loads once the forces have been determined. However, if the truss is truly continuous, and, therefore, the truss in each span is statically indeterminant, the member forces are dependent on the stiffness of the truss members. This may make several iterations of member-force calculations necessary. But where sufficient points of articulation are provided to make each individual truss statically determinant, such as the case where a suspended span is inserted in a cantilever truss, the member forces are not a function of member stiffness. As a result, live-load forces need be computed only once, and dead-load member forces need to be updated only for the change in member weight as the design cycle proceeds. When the stresses have been computed, design proceeds much as for simple spans.

The preceding discussion implies that some simplification is possible by using cantilever design rather than continuous design. In fact, all other things being equal, the total weight of members will not be much different in the two designs if points of articulation are properly selected. More roadway joints will be required in the cantilever, but they, and the bearings, will be subject to less movement. However, use of continuity should be considered because elimination of the joints and devices necessary to provide for articulation will generally reduce maintenance, stiffen the bridge, increase redundancy and, therefore, improve the general robustness of the bridge.