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# SECTION 14

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## ARCH BRIDGES

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Basic principles of arch construction have been known and used successfully for centuries. Magnificent stone arches constructed under the direction of engineers of the ancient Roman Empire are still in service after 2000 years, as supports for aqueducts or highways. One of the finest examples is the Pont du Gard, built as part of the water-supply system for the city of Nîmes, France.

Stone was the principal material for arches until about two centuries ago. In 1779, the first metal arch bridge was built. Constructed of cast iron, it carried vehicles over the valley of the Severn River at Coalbrookdale, England. The bridge is still in service but now is restricted to pedestrian traffic. Subsequently, many notable iron or steel arches were built. Included was Eads' Bridge, with three tubular steel arch spans, 502, 520, and 502 ft, over the Mississippi River at St. Louis. Though completed in 1874, it now carries large daily volumes of heavy highway traffic.

Until 1900, stone continued as a strong competitor of iron and steel. After 1900, concrete became the principal competitor of steel for shorter-span arch bridges.

Development of structural steels made it feasible to construct long-span arches economically. The 1675-ft Bayonne Bridge, between Bayonne, N.J., and Staten Island, N.Y., was completed in 1931. The 1000-ft Lewiston-Queenston Bridge over the Niagara River on the United States–Canadian border was put into service in 1962. Availability of more high-strength steels and improved fabrication techniques expanded the feasibility of steel arches for long spans. Examples include the 1255-ft-span Fremont Bridge in Portland, Ore., finished in 1973, and the 1700-ft-span New River Gorge Bridge near Fayetteville, W. Va., opened in 1977.

Nearly all the steel arches that have been built lie in vertical planes. Accordingly, this section discusses design principles for such arches. A few arch bridges, however, have been constructed with ribs inclined toward each other. This construction is effective in providing lateral stability and offers good appearance. Also, the decrease in average distance between the arch ribs of a bridge often makes possible the use of more economical Vierendeel-girder bracing instead of trussed bracing. Generally, though, inclined arches are not practicable for bridges with very wide roadways unless the span is very long, because of possible interference with traffic clearances. Further, inclined arch ribs result in more complex beveled connections between members.

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\*Revised from Sec. 13, "Arch Bridges," by George S. Richardson (deceased), Richardson, Gordon and Associates, Pittsburgh, in *Structural Steel Designer's Handbook*, 1st ed., McGraw-Hill Book Company, New York.

## 14.1 TYPES OF ARCHES

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In the most natural type of arch, the horizontal component of each reaction, or thrust, is carried into a buttress, which also carries the vertical reaction. This type will be referred to as the *true arch*. The application of arch construction, however, can be greatly expanded economically by carrying the thrust through a tie, a tension member between the ends of the span. This type will be referred to as a *tied arch*.

Either a truss or girder may be used for the arch member. Accordingly, arch bridges are classified as *trussed* or *solid-ribbed*.

Arch bridges are also classified according to the degree of articulation. A *fixed arch*, in which the construction prevents rotation at the ends of the span, is statically indeterminate, so far as external reactions are concerned, to the third degree. If the span is articulated at the ends, it becomes *two-hinged* and statically indeterminate to the first degree. In recent years, most arch bridges have been constructed as either fixed or two-hinged. Sometimes a hinge is included at the crown in addition to the end hinges. The bridge then becomes *three-hinged* and statically determinate.

In addition, arch bridges are classified as *deck* construction when the arches are entirely below the deck. This is the most usual type for the true arch. Tied arches, however, normally are constructed with the arch entirely above the deck and the tie at deck level. This type will be referred to as a *through arch*. Both true and tied arches, however, may be constructed with the deck at some intermediate elevation between springing and crown. These types are classified as *half-through*.

The arch also may be used as one element combined with another type of structure. For example, many structures have been built with a three-span continuous truss as the basic structure and with the central span arched and tied. This section is limited to structures in which the arch type is used independently.

## 14.2 ARCH FORMS

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A great variety of forms have been used for trussed or solid-ribbed arch bridges. The following are some of the principal forms used.

Lindenthal's Hell Gate Bridge over the East River in New York has trusses deep at the ends and shallow at the crown. The bottom chord is a regular arch form. The top chord follows a reversed curve transitioning from the deep truss at the end to the shallow truss at the center. Accordingly, it is customary to refer to arch trusses of this form as Hell-Gate-type trusses. In another form commonly used, top and bottom chords are parallel. For a two-hinged arch, a crescent-shaped truss is another logical form.

For solid-ribbed arches, single-web or box girders may be used. Solid-ribbed arches usually are built with girders of constant depth. Variable-depth girders, tapering from deep sections at the springing to shallower sections at the crown, however, have been used occasionally for longer spans. As with trussed construction, a crescent-shaped girder is another possible form for a two-hinged arch.

Tied arches permit many variations in form to meet specific site conditions. In a true arch (without ties), the truss or solid rib must carry both thrust and moment under variable loading conditions. These stresses determine the most effective depth of truss or girder. In a tied arch, the thrust is carried by the arch truss or solid rib, but the moment for variable loading conditions is divided between arch and tie, somewhat in proportion to the respective stiffnesses of these two members. For this reason, for example, if a deep girder is used for the arch and a very shallow member for the tie, most of the moment for variable loading is carried by the arch rib. The tie acts primarily as a tension member. But if a relatively deep member is used for the tie, it carries a high proportion of the moment, and a relatively

shallow member may be used for the arch rib. In some cases, a truss has been used for the arch tie in combination with a shallow, solid rib for the arch. This combination may be particularly applicable for double-deck construction.

Rigid-framed bridges, sometimes used for grade-separation structures, are basically another form of two-hinged or fixed arch. The generally accepted arch form is a continuous, smooth-curve member or a segmental arch (straight between panel points) with breaks located on a smooth-curve axis. For a rigid frame, however, the arch axis becomes rectangular in form. Nevertheless, the same principles of stress analysis may be used as for the smooth-curve arch form.

The many different types and forms of arch construction make available to bridge engineers numerous combinations to meet variable site conditions.

## 14.3 SELECTION OF ARCH TYPE AND FORM

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Some of the most important elements influencing selection of type and form of arch follow.

**Foundation Conditions.** If a bridge is required to carry a roadway or railroad across a deep valley with steep walls, an arch is probably a feasible and economical solution. (This assumes that the required span is within reasonable limits for arch construction.) The condition of steep walls indicates that foundation conditions should be suitable for the construction of small, economical abutments. Generally, it might be expected that under these conditions the solution would be a deck bridge. There may be other controls, however, that dictate otherwise. For example, the need for placing the arch bearings safely above high-water elevation, as related to the elevation of the deck, may indicate the advisability of a half-through structure to obtain a suitable ratio of rise to span. Also, variable foundation conditions on the walls of the valley may fix a particular elevation as much more preferable to others for the construction of the abutments. Balancing of such factors will determine the best layout to satisfy foundation conditions.

**Tied-Arch Construction.** At a bridge location where relatively deep foundations are required to carry heavy reactions, a true arch, transmitting reactions directly to buttresses, is not economical, except for short spans. There are two alternatives, however, that may make it feasible to use arch construction.

If a series of relatively short spans can be used, arch construction may be a good solution. In this case, the bridge would comprise a series of equal or nearly equal spans. Under these conditions, dead-load thrusts at interior supports would be balanced or nearly balanced. With the short spans, unbalanced live-load thrusts would not be large. Accordingly, even with fairly deep foundations, intermediate pier construction may be almost as economical as for some other layout with simple or continuous spans. There are many examples of stone, concrete, and steel arches in which this arrangement has been used.

The other alternative to meet deep foundation requirements is tied-arch construction. The tie relieves the foundation of the thrust. This places the arch in direct competition with other types of structures for which only vertical reactions would result from the application of dead and live loading.

There has been some concern over the safety of tied-arch bridges because the ties can be classified as fracture-critical members. A fracture-critical member is one that would cause collapse of the bridge if it fractured. Since the horizontal thrust of a tied-arch is resisted by its tie, most tied arches would collapse if the tie were lost. While some concern over fracture of welded tie girders is well-founded, methods are available for introducing redundancy in the construction of ties. These methods include using ties fabricated from multiple bolted-together components and multiple post-tensioning tendons. Tied arches often provide cost-

effective and esthetically pleasing structures. This type of structure should not be dismissed over these concerns, because it can be easily designed to address them.

***Length of Span.*** Generally, determination of the best layout for a bridge starts with trial of the shortest feasible main span. Superstructure costs per foot increase rapidly with increase in span. Unless there are large offsetting factors that reduce substructure costs when spans are lengthened, the shortest feasible span will be the most economical.

Arch bridges are applicable over a wide range of span lengths. The examples in Art. 14.8 cover a range from a minimum of 193 ft to a maximum of 1700 ft. With present high-strength steels and under favorable conditions, spans on the order of 2000 ft are feasible for economical arch construction.

In addition to foundation conditions, many other factors may influence the length of span selected at a particular site. Over navigable waters, span is normally set by clearance requirements of regulatory agencies. For example, the U.S. Coast Guard has final jurisdiction over clearance requirements over navigable streams. In urban or other highly built-up areas, the span may be fixed by existing site conditions that cannot be altered.

***Truss or Solid Rib.*** Most highway arch bridges with spans up to 750 ft have been built with solid ribs for the arch member. There may, however, be particular conditions that would make it more economical to use trusses for considerably shorter spans. For example, for a remote site with difficult access, truss arches may be less expensive than solid-ribbed arches, because the trusses may be fabricated in small, lightweight sections, much more readily transported to the bridge site.

In the examples of Art. 14.8, solid ribs have been used in spans up to 1255 ft, as for the Fremont Bridge, Portland, Ore. For spans over 750 ft, however, truss arches should be considered. Also, for spans under this length for very heavy live loading, as for railroad bridges, truss arches may be preferable to solid-rib construction.

For spans over about 600 ft, control of deflection under live loading may dictate the use of trusses rather than solid ribs. This may apply to bridges designed for heavy highway loading or heavy transit loading as well as for railroad bridges. For spans above 1000 ft, truss arches, except in some very unusual case, should be used.

***Articulation.*** For true, solid-ribbed arches the choice between fixed and hinged ends will be a narrow one. In a true arch it is possible to carry a substantial moment at the springing line if the bearing details are arranged to provide for it. This probably will result in some economy, particularly for long spans. It is, however, common practice to use two-hinged construction.

An alternative is to let the arch act as two-hinged under partial or full dead load and then fix the end bearings against rotation under additional load.

Tied arches act substantially as two-hinged, regardless of the detail of the connection to the tie.

Some arches have been designed as three-hinged under full or partial dead load and then converted to the two-hinged condition. In this case, the crown hinge normally is located on the bottom chord of the truss. If the axis of the bottom chord follows the load thrust line for the three-hinged condition, there will be no stress in the top chord or web system of the truss. Top chord and web members will be stressed only under load applied after closure. These members will be relatively light and reasonably uniform in section. The bottom chord becomes the main load-bearing member.

If, however, the arch is designed as two-hinged, the thrust under all loading conditions will be nearly equally divided between top and bottom chords. For a given ratio of rise to span, the total horizontal thrust at the end will be less than that for the arrangement with part of the load carried as a three-hinged arch. Shifting from three to two hinges has the effect of increasing the rise of the arch over the rise measured from springing to centerline of bottom chord.

**Esthetics.** For arch or suspension-type bridges, a functional layout meeting structural requirements normally results in simple, clean-cut, and graceful lines. For long spans, no other bridge type offered serious competition so far as excellent appearance is concerned until about 1950. Since then, introduction of cable-stayed bridges and orthotropic-deck girder construction has made construction of good-looking girders feasible for spans of 2500 ft or more. Even with conventional deck construction but with the advantage of high-strength steels, very long girder spans are economically feasible and esthetically acceptable.

The arch then must compete with suspension, cable-stayed, and girder bridges so far as esthetic considerations are concerned. From about 1000 ft to the maximum practical span for arches, the only competitors are the cable-supported types.

Generally, architects and engineers prefer, when all other things are equal, that deck structures be used for arch bridges. If a through or half-through structure must be used, solid-ribbed arches are desirable when appearance is of major concern, because the overhead structure can be made very light and clean-cut (Figs. 14.5 to 14.8 and 14.15 to 14.18).

**Arch Form as Related to Esthetics.** For solid-ribbed arches, designers are faced with the decision as to whether the rib should be curved or constructed on segmental chords (straight between panel points). A rib on a smooth curve presents the best appearance. Curved ribs, however, involve some increase in material and fabrication costs.

Another decision is whether to make the rib of constant depth or tapered.

One factor that has considerable bearing on both these decisions is the ratio of panel length to span. As panel length is reduced, the angular break between chord segments is reduced, and a segmental arch approaches a curved arch in appearance. An upper limit for panel length should be about  $\frac{1}{15}$  of the span.

In a study of alternative arch configurations for a 750-ft span, four solid-ribbed forms were considered. An architectural consultant rated these in the following order:

- Tapered rib, curved
- Tapered-rib on chords
- Constant-depth rib, curved
- Constant-depth rib on chords

He concluded that the tapered rib, 7 ft deep at the springing line and 4 ft deep at the crown, added considerably to the esthetic quality of the design as compared with a constant-depth rib. He also concluded that the tapered rib would minimize the angular breaks at panel points with the segmental chord axis. The tapered rib on chords was used in the final design of the structure. The effect of some of these variables on economy is discussed in Art. 14.6.

## 14.4 COMPARISON OF ARCH WITH OTHER BRIDGE TYPES

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Because of the wide range of span length within which arch construction may be used (Art. 14.3), it is competitive with almost all other types of structures.

**Comparison with Simple Spans.** Simple-span girder or truss construction normally falls within the range of the shortest spans used up to a maximum of about 800 ft. Either true arches under favorable conditions or tied arches under all conditions are competitive within the range of 200 to 800 ft. (There will be small difference in cost between these two types within this span range.) With increasing emphasis on appearance of bridges, arches are generally selected rather than simple-span construction, except for short spans for which beams or girders may be used.

**Comparison with Cantilever or Continuous Trusses.** The normal range for cantilever or continuous-truss construction is on the order of 500 to 1800 ft for main spans. More likely, a top limit is about 1500 ft. Tied arches are competitive for spans within the range of 500 to 1000 ft. True arches are competitive, if foundation conditions are favorable, for spans from 500 ft to the maximum for the other types. The relative economy of arches, however, is enhanced where site conditions make possible use of relatively short-span construction over the areas covered by the end spans of the continuous or cantilever trusses.

The economic situation is approximately this: For three-span continuous or cantilever layouts arranged for the greatest economy, the cost per foot will be nearly equal for end and central spans. If a tied or true arch is substituted for the central span, the cost per foot may be more than the average for the cantilever or continuous types. If, however, relatively short spans are substituted for the end spans of these types, the cost per foot over the length of those spans is materially reduced. Hence, for a combination of short spans and a long arch span, the overall cost between end piers may be less than for the other types. In any case, the cost differential should not be large.

**Comparison with Cable-Stayed and Suspension Bridges.** Such structures normally are not used for spans of less than 500 ft. Above 3000 ft, suspension bridges are probably the most practical solution. In the shorter spans, self-anchored construction is likely to be more economical than independent anchorages. Arches are competitive in cost with the self-anchored suspension type or similar functional type with cable-stayed girders or trusses. There has been little use of suspension bridges for spans under 1000 ft, except for some self-anchored spans. For spans above 1000 ft, it is not possible to make any general statement of comparative costs. Each site requires a specific study of alternative designs.

## 14.5 ERECTION OF ARCH BRIDGES

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Erection conditions vary so widely that it is not possible to cover many in a way that is generally applicable to a specific structure.

**Cantilever Erection.** For arch bridges, except short spans, cantilever erection usually is used. This may require use of two or more temporary piers. Under some conditions, such as an arch over a deep valley where temporary piers are very costly, it may be more economical to use temporary tiebacks.

Particularly for long spans, erection of trussed arches often is simpler than erection of solid-ribbed arches. The weights of individual members are much smaller, and trusses are better adapted to cantilever erection. The Hell-Gate-type truss (Art. 14.2) is particularly suitable because it requires little if any additional material in the truss on account of erection stresses.

For many double-deck bridges, use of trusses for the arch ties simplifies erection when trusses are deep enough and the sections large enough to make cantilever erection possible and at the same time to maintain a clear opening to satisfy temporary navigation or other clearance requirements.

**Control of Stress Distribution.** For trussed arches designed to act as three-hinged, under partial or full dead load, closure procedures are simple and positive. Normally, the two halves of the arch are erected to ensure that the crown hinge is high and open. A top-chord member at the crown is temporarily omitted. The trusses are then closed by releasing the tiebacks or lowering temporary intermediate supports. After all dead load for the three-hinged condition is on the span, the top chord is closed by inserting the final member. During this operation consideration must be given to temperature effects to ensure that closure conditions conform to temperature-stress assumptions.

If a trussed arch has been designed to act as two-hinged under all conditions of loading, the procedure may be first to close the arch as three-hinged. Then, jacks are used at the crown to attain the calculated stress condition for top and bottom chords under the closing erection load and temperature condition. This procedure, however, is not as positive and not as certain of attaining agreement between actual and calculated stresses as the other procedure described. (There is a difference of opinion among bridge engineers on this point.)

Another means of controlling stress distribution may be used for tied arches. Suspender lengths are adjusted to alter stresses in both the arch ribs and the ties.

**Fixed Bases.** For solid-ribbed arches to be erected over deep valleys, there may be a considerable advantage in fixing the ends of the ribs. If this is not provided for in design, it may be necessary to provide temporary means for fixing bases for cantilever erection of the first sections of the ribs. If the structure is designed for fixed ends, it may be possible to erect several sections as cantilevers before it becomes necessary to install temporary tiebacks.

## 14.6 DESIGN OF ARCH RIBS AND TIES

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Computers greatly facilitate preliminary and final design of all structures. They also make possible consideration of many alternative forms and layouts, with little additional effort, in preliminary design. Even without the aid of a computer, however, experienced designers can, with reasonable ease, investigate alternative layouts and arrive at sound decisions for final arrangements of structures.

**Rise-Span Ratio.** The generally used ratios of rise to span cover a range of about 1:5 to 1:6. For all but two of the arch examples in Art. 14.8, the range is from a maximum of 1:4.7 to a minimum of 1:6.3. The flatter rise is more desirable for through arches, because appearance will be better. Cost will not vary materially within the rise limits of 1:5 to 1:6. These rise ratios apply both to solid ribs and to truss arches with rise measured to the bottom chord.

**Panel Length.** For solid-ribbed arches fabricated with segmental chords, panel length should not exceed  $\frac{1}{15}$  of the span. This is recommended for esthetic reasons, to prevent too large angular breaks at panel points. Also, for continuously curved axes, bending stresses in solid-ribbed arches become fairly severe if long panels are used. Other than this limitation, the best panel length for an arch bridge will be determined by the usual considerations, such as economy of deck construction.

**Ratio of Depth to Span.** In the examples in Art. 14.8, the true arches (without ties) with constant-depth solid ribs have depth-span ratios from 1:58 to 1:79. The larger ratio, however, is for a short span. A more normal range is 1:70 to 1:80. These ratios also are applicable to solid-ribbed tied arches with shallow ties. In such cases, since the ribs must carry substantial bending moments, depth requirements are little different from those for a true arch. For structures with variable-depth ribs, the depth-span ratio may be relatively small (Fig. 14.7).

For tied arches with solid ribs and deep ties, depth of rib may be small, because the ties carry substantial moments, thus reducing the moments in the ribs. For a number of such structures, the depth-span ratio ranges from 1:140 to 1:190, and for the Fremont Bridge, Portland, Ore., is as low as 1:314. Note that such shallow ribs can be used only with girder or trussed ties of considerable depth.

For truss arches, whether true or tied, the ratio of crown depth to span may range from 1:25 to 1:50. Depth of tie has little effect on depth of truss required. Except for some unusual arrangement, the moment of inertia of the arch truss is much larger than the moment of

inertia of its tie, which primarily serves as a tension member to carry the thrust. Hence, an arch truss carries substantial bending moments whether or not it is tied, and required depth is not greatly influenced by presence or absence of a tie.

**Single-Web or Box Girders.** For very short arch spans, single-web girders are more economical than box girders. For all the solid-ribbed arches in Art. 14.8, however, box girders were used for the arch ribs. These examples include a minimum span of 193 ft. Welded construction greatly facilitates use of box members in all types of structures.

For tied arches for which shallow ties are used, examples in Art. 14.8 show use of members made up of web plates with diaphragms and rolled shapes with post-tensioned strands. More normally, however, the ties, like solid ribs, would be box girders.

**Truss Arches.** All the usual forms of bolted or welded members may be used in truss arches but usually sealed, welded box members are preferred. These present a clean-cut appearance. There also is an advantage in the case of maintenance.

Another variation of truss arches that can be considered is use of Vierendeel trusses (web system without diagonals). In the past, complexity of stress analysis for this type discouraged their use. With computers, this disadvantage is eliminated. Various forms of Vierendeel truss might well be used for both arch ribs and ties. There has been some use of Vierendeel trusses for arch bracing, as shown in the examples in Art. 14.8. This design provides an uncluttered, good-looking bracing system.

**Dead-Load Distribution.** It is normal procedure for both true and tied solid-ribbed arches to use an arch axis conforming closely to the dead-load thrust line. In such cases, if the rib is cambered for dead load, there will be no bending in the rib under that load. The arch will be in pure compression. If a tied arch is used, the tie will be in pure tension. If trusses are used, the distribution of dead-load stress may be similarly controlled. Except for three-hinged arches, however, it will be necessary to use jacks at the crown or other stress-control procedures to attain the stress distribution that has been assumed.

**Live-Load Distribution.** One of the advantages of arch construction is that fairly uniform live loading, even with maximum-weight vehicles, creates relatively low bending stresses in either the rib or the tie. Maximum bending stresses occur only under partial loading not likely to be realized under normal heavy traffic flow. Maximum live-load deflection occurs in the vicinity of the quarter point with live load over about half the span.

**Wind Stresses.** These may control design of long-span arches carrying two-lane roadways or of other structures for which there is relatively small spacing of ribs compared with span length. For a spacing-span ratio larger than 1:20, the effect of wind may not be severe. As this ratio becomes substantially smaller, wind may affect sections in many parts of the structure.

**Thermal Stresses.** Temperature causes stress variation in arches. One effect sometimes neglected but which should be considered is that of variable temperature throughout a structure. In a through, tied arch during certain times of the day or night, there may be a large difference in temperature between rib and tie due to different conditions of exposure. This difference in temperature easily reaches 30°F and may be much larger.

**Deflection.** For tied arches of reasonable rigidity, deflection under live load causes relatively minor changes in stress (secondary stresses). For a 750-ft span with solid-ribbed arches 7 ft deep at the springing line and 4 ft deep at the crown and designed for a maximum live-load deflection of  $1/800$  of the span, the secondary effect of deflections was computed as less than 2% of maximum allowable unit stress. For a true arch, however, this effect may be considerably larger and must be considered, as required by design specifications.



**Dead-Load to Total-Load Ratios.** For some 20 arch spans checked, the ratio of dead load to total load varied within the narrow range of 0.74 to 0.88. A common ratio is about 0.85. This does not mean that the ratio of dead-load stress to maximum total stress will be 0.85. This stress ratio may be fairly realistic for a fully loaded structure, at least for most of the members in the arch system. For partial live loading, however, which is the loading condition causing maximum live-load stress, the ratio of dead to total stress will be much lower, particularly as span decreases.

For most of the arches checked, the ratio of weight of arch ribs or, in the case of tied arches, weight of ribs and ties to, total load ranged from about 0.20 to 0.30. This is true despite the wide range of spans included and the great variety of steels used in their construction.

Use of high-strength steels helps to maintain a low ratio for the longer spans. For example, for the Fort Duquesne Bridge, Pittsburgh, a double-deck structure of 423-ft span with a deep truss as a tie, the ratio of weight of arch ribs plus truss ties to total load is about 0.22, or a normal factor within the range previously cited. For this bridge, arch ribs and trusses were designed with 77% of A440 steel and the remainder A36. These are suitable strength steels for this length of span.

For the Fort Pitt Bridge, Pittsburgh, with a 750-ft span and the same arrangement of structure with shallow girder ribs and a deep truss for the ties, the ratio of weight of steel in ribs plus trussed ties to total load is 0.33. The same types of steel in about the same percentages were used for this structure as for the Fort Duquesne Bridge. A higher-strength steel, such as A514, would have resulted in a much lower percentage for weight of arch ribs and trusses and undoubtedly in considerable economy. When the Fort Pitt arch was designed, however, the owner decided there had not been sufficient research and testing of the A514 steel to warrant its use in this structure.

For a corresponding span of 750 ft designed later for the Glenfield Bridge at Pittsburgh, a combination of A588 and A514 steels was used for the ribs and ties. The ratio of weight of ribs plus ties to total load is 0.19.

Incidentally, the factors for this structure, a single-deck bridge with six lanes of traffic plus full shoulders, are almost identical with the corresponding factors for the Sherman Minton Bridge at Louisville, Ky., an 800-ft double-deck structure with truss arches carrying three lanes of traffic on each deck. The factors for the Pittsburgh bridge are 0.88 for ratio of dead load to total load and 0.19 for ratio of weight of ribs plus ties to total load. The corresponding factors for the Sherman Minton arch are 0.85 and 0.19. Although these factors are almost identical, the total load for the Pittsburgh structure is considerably larger than that for the Louisville structure. The difference may be accounted for primarily by the double-deck structure for the latter, with correspondingly lighter deck construction.

For short spans, particularly those on the order of 250 ft or less, the ratio of weight of arch rib to total load may be much lower than the normal range of 0.20 to 0.30. For example, for a short span of 216 ft, this ratio is 0.07. On the other hand, for a span of only 279 ft, the ratio is 0.18, almost in the normal range.

A ratio of arch-rib weight to total load may be used by designers as one guide in selecting the most economical type of steel for a particular span. For a ratio exceeding 0.25, there is an indication that a higher-strength steel than has been considered might reduce costs and its use should be investigated, if available.

**Effect of Form on Economy of Construction.** For solid-ribbed arches, a smooth-curve axis is preferable to a segmental-chord axis (straight between panel points) so far as appearance is concerned. The curved axis, however, involves additional cost of fabrication. At the least, some additional material is required in fabrication of the arch because of the waste in cutting the webs to the curved shape. In addition to this waste, some material must be added to the ribs to provide for increased stresses due to bending. This occurs for the following reason: Since most of the load on the rib is applied at panel points, the thrust line is nearly straight between panel points. Curving the axis of the rib causes eccentricity of the thrust line with

respect to the axis and thus induces increased bending moments, particularly for dead load. All these effects may cause an increase in the cost of the curved rib on the order of 5 to 10%.

For tied solid-ribbed arches for which it is necessary to use a very shallow tie, costs are larger than for shallow ribs and deep ties. (A shallow tie may be necessary to meet under-clearance restrictions and vertical grades of the deck.) A check of a 750-ft span for two alternate designs, one with a 5-ft constant-depth rib and 12.5-ft-deep tie and the other with a 10-ft-deep rib and 4-ft-deep tie, showed that the latter arrangement, with shallow tie, required about 10% more material than the former, with deep tie. The actual increased construction cost might be more on the order of 5%, because of some constant costs for fabrication and erection that would not be affected by the variation in weight of material.

Comparison of a tapered rib with a constant-depth rib indicates a small percentage saving in material in favor of the tapered rib. Thus, costs for these two alternatives would be nearly equal.

## 14.7 DESIGN OF OTHER ELEMENTS

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A few special conditions relating to elements of arch bridges other than the ribs and ties should be considered in design of arch bridges.

**Floor System.** Tied arches, particularly those with high-strength steels, undergo relatively large changes in length of deck due to variation in length of tie under various load conditions. It therefore is normally necessary to provide deck joints at intermediate points to provide for erection conditions and to avoid high participation stresses.

**Bracing.** During design of the Bayonne Bridge arch (Art. 14.8), a study in depth explored the possibility of eliminating most of the sway bracing (bracing in a vertical plane between ribs). In addition to detailed analysis, studies were made on a scaled model to check the effect of various arrangements of this bracing. The investigators concluded that, except for a few end panels, the sway bracing could be eliminated. Though many engineers still adhere to an arbitrary specification requirement calling for sway bracing at every panel point of any truss, more consideration should be given to the real necessity for this. Furthermore, elimination of sway frames not only reduces costs but it also greatly improves the appearance of the structure. For several structures from which sway bracing has been omitted, there has been no adverse effect.

Various arrangements may be used for lateral bracing systems in arch bridges. For example, a diamond pattern, omitting cross struts at panel points, is often effective. Also, favorable results have been obtained with a Vierendeel truss.

In the design of arch bracing, consideration must be given to the necessity for the lateral system to prevent lateral buckling of the two ribs functioning as a single compression member. The lateral bracing thus is the lacing for the two chords of this member.

**Hangers.** These must be designed with sufficient rigidity to prevent adverse vibration under aerodynamic forces or as very slender members (wire rope or bridge strand). A number of long-span structures incorporate the latter device. Vibration problems have developed with some bridges for which rigid members with high slenderness ratios have been used. Corrosion resistance and provision for future replacement are other concerns which must be addressed in design of wire hangers. While not previously discussed in this section, the use of inclined hangers has been employed for some tied arch bridges. This hanger arrangement can add considerable stiffness to the arch-tie structure and cause it to function similar to a truss system with crossing diagonals. For such an arrangement, stress reversal, fatigue, and more complex details must be investigated and addressed.

## 14.8 EXAMPLES OF ARCH BRIDGES

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Thanks to the cooperation of several engineers in private and public practice, detailed information on about 25 arch bridges has been made available. Sixteen have been selected from this group to illustrate the variety of arch types and forms in the wide range and span length for which steel arches have been used. Many of these bridges have been awarded prizes in the annual competition of the American Institute of Steel Construction.

The examples include only bridges constructed within the United States, though there are many notable arch bridges in other countries. A noteworthy omission is the imaginative and attractive Port Mann Bridge over the Fraser River in Canada. C.B.A. Engineering Ltd., consulting engineers, Vancouver, B.C., were the design engineers. By use of an orthotropic deck and stiffened, tied, solid-ribbed arch, an economical layout was developed with a central span of 1,200 ft, flanked by side spans of 360 ft each. A variety of steels were used, including A373, A242, and A7.

Following are data on arch bridges that may be useful in preliminary design. (*Text continues on page 14.44.*)



FIGURE 14.1

NEW RIVER GORGE BRIDGE

LOCATION: Fayetteville, West Virginia  
TYPE: Trussed deck arch, 40 panels, 36 at 40 ± to 43 ± ft  
SPAN: 1,700 ft    RISE: 353 ft    RISE/SPAN = 1:4.8  
NO. OF LANES OF TRAFFIC: 4  
HINGES: 2    CROWN DEPTH: 34 ft    DEPTH/SPAN = 1:50

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing of roadway .....	8,600
Railings and parapets .....	1,480
Floor steel for roadway .....	3,560
Arch trusses .....	11,180
Arch bracing .....	1,010
Arch bents and bracing .....	2,870
TOTAL .....	28,700

SPECIFICATION FOR LIVE LOADING: H520-44  
EQUIVALENT LIVE + IMPACT LOADING PER ARCH FOR FULLY LOADED  
STRUCTURE: 1,126 lb per ft  
TYPES OF STEEL IN STRUCTURE:

Arch .....	A588
Floor system .....	A588

OWNER: State of West Virginia  
ENGINEER: Michael Baker, Jr., Inc.  
FABRICATOR/ERECTOR: American Bridge Division, U.S. Steel Corporation  
DATE OF COMPLETION: October, 1977

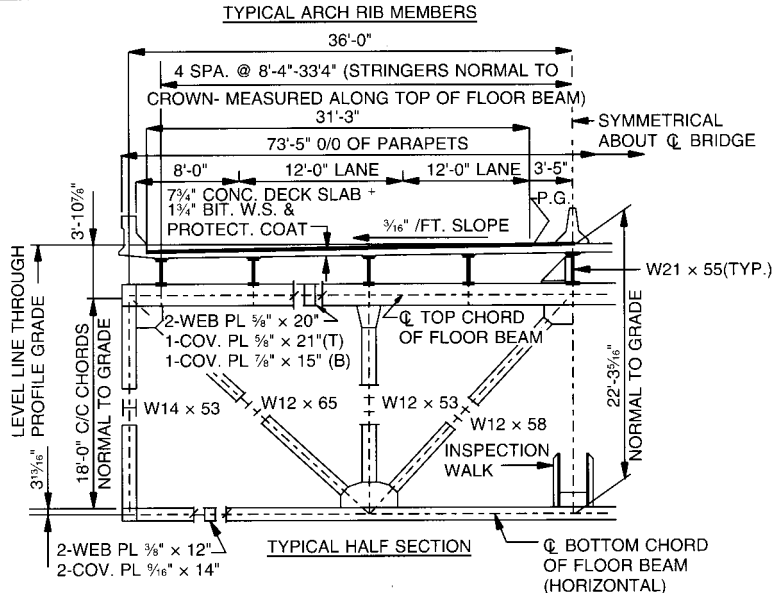
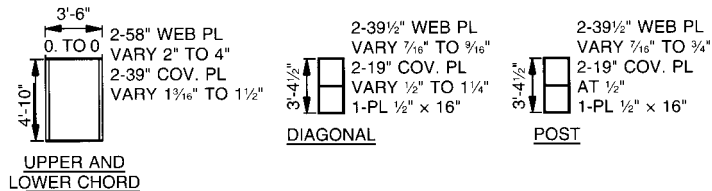
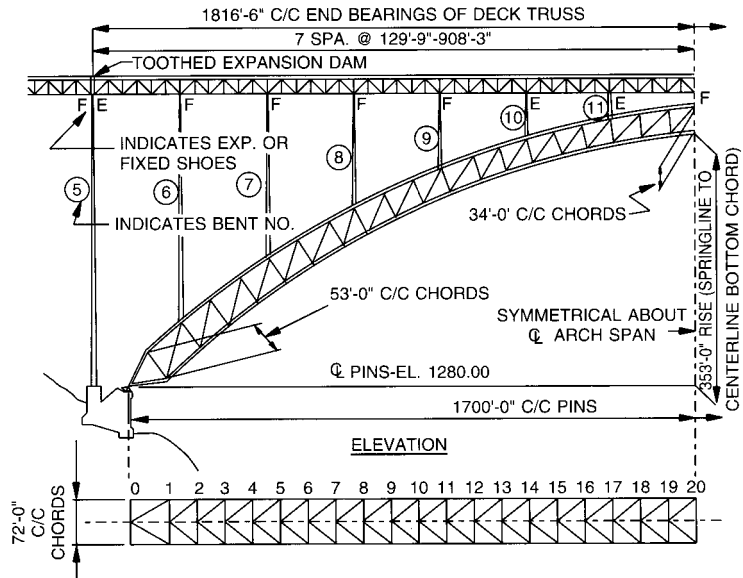


FIGURE 14.2 Details of New River Gorge Bridge.



FIGURE 14.3

BAYONNE BRIDGE

LOCATION: Between Bayonne, N.J., and Port Richmond, Staten Island, N.Y.

TYPE: Half-through truss arch, 40 panels at 41.3 ft

SPAN: 1,675 ft      RISE: 266 ft      RISE/SPAN = 1:6.3

NO. OF LANES OF TRAFFIC: 4 plus 2 future rapid transit

HINGES: 2      CROWN DEPTH: 37.5 FT      DEPTH/SPAN = 1:45

AVERAGE DEAD LOAD:	LB PER FT
Track, paving .....	6,340
Floor steel and floor bracing .....	6,160
Arch truss and bracing .....	14,760
Arch hangers .....	540
Miscellaneous .....	200
TOTAL .....	28,000

SPECIFICATION LIVE LOADING:	LB PER FT
2 rapid-transit lines at 6,000 lb per ft .....	12,000
4 roadway lanes at 2,500 lb per ft .....	10,000
2 sidewalks at 600 lb .....	1,200
TOTAL (unreduced) .....	23,200

EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED  
STRUCTURE WITH REDUCTION FOR MULTIPLE LANES AND LENGTH OF LOADING:  
2,800 lb per lin ft

TYPES OF STEEL IN STRUCTURE: About 50% carbon steel, 30% silicon steel, and 20% high-  
alloy steel (carbon-manganese)

OWNER: The Port Authority of New York and New Jersey

ENGINEER: O. H. Ammann, Chief Engineer

FABRICATOR: American Bridge Co., U.S. Steel Corp. (also erector)

DATE OF COMPLETION: 1931

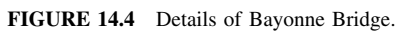




FIGURE 14.5

FREMONT BRIDGE

LOCATION: Portland, Oregon  
TYPE: Half-through, tied, solid ribbed arch, 28 panels at 44.83 ft  
SPAN: 1,255 ft      RISE: 341 ft      RISE/SPAN = 1:3.7  
NO. OF LANES OF TRAFFIC: 4 each upper and lower roadways  
HINGES: 2      DEPTH: 4 ft      DEPTH/SPAN = 1:314  
AVERAGE DEAD LOAD:

	LB PER FT
Decks and surfacing .....	10,970
Railings and Parapets .....	1,280
Floor steel for roadway .....	4,000
Floor bracing .....	765
Arch ribs .....	2,960
Arch bracing .....	1,410
Arch hangers or columns and bracing .....	1,250
Arch tie girders .....	4,200
TOTAL .....	26,835

SPECIFICATION FOR LIVE LOADING: AASHTO HS20-44  
EQUIVALENT LIVE + IMPACT LOADING FOR ARCH FOR FULLY LOADED  
STRUCTURE: 2,510 lb per ft  
TYPES OF STEEL IN STRUCTURE:  
Arch ribs and tie girders ..... A514, A588, A441, A36  
Floor system ..... A588, A441, A36  
OWNER: State of Oregon, Department of Transportation  
ENGINEER: Parson, Brinckerhoff, Quade & Douglas  
FABRICATOR: American Bridge Division, U.S. Steel Corp.  
ERECTOR: Murphy Pacific Corporation  
DATE OF COMPLETION: 1973



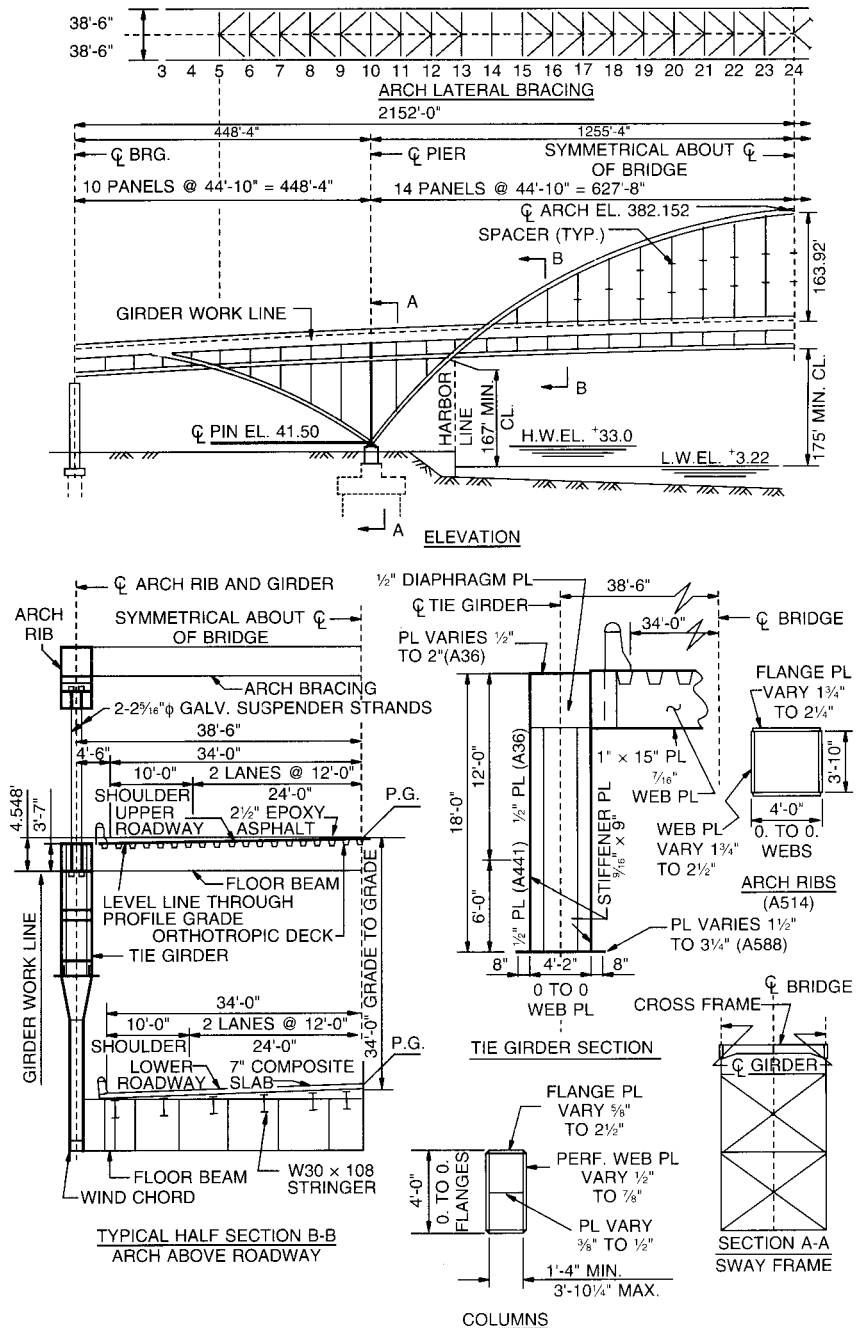


FIGURE 14.6 Details of Fremont Bridge.



FIGURE 14.7

ROOSEVELT LAKE BRIDGE

LOCATION: Roosevelt, Arizona, SR 188  
TYPE: Half through, solid rib arch, 16 panels at 50 ft  
SPAN: 1,080 ft      RISE: 230 ft      RISE/SPAN = 1:4.7  
NO. OF LANES OF TRAFFIC: 2  
HINGES: 0      CROWN DEPTH: 8 ft      DEPTH/SPAN = 1:135  
AVERAGE DEAD LOAD:

	LB PER FT
Deck slab, and surfacing of roadway .....	4,020
Railings and parapets .....	800
Floor steel for roadway .....	1,140
Floor bracing .....	190
Arch ribs .....	4,220
Arch bracing .....	790
Arch hangers .....	80
TOTAL .....	11,240

SPECIFICATION FOR LIVE LOADING: HS20-44  
EQUIVALENT LIVE + IMPACT LOADING PER ARCH FOR FULLY LOADED STRUCTURE:  
971 lb per ft  
TYPES OF STEEL IN STRUCTURE:

Arch ribs and ties .....	A572
Hanger floorbeams and stringers .....	A572
All others .....	A36

OWNER: Arizona Department of Transportation  
ENGINEER: Howard Needles Tammen and Bergendoff  
CONTRACTOR: Edward Kraemer & Sons, Inc.  
FABRICATOR: Pittsburgh DesMoines Steel Co./Schuff Steel  
ERECTOR: John F. Beasley Construction Co.  
DATE OF COMPLETION: October 23, 1991 Public Opening

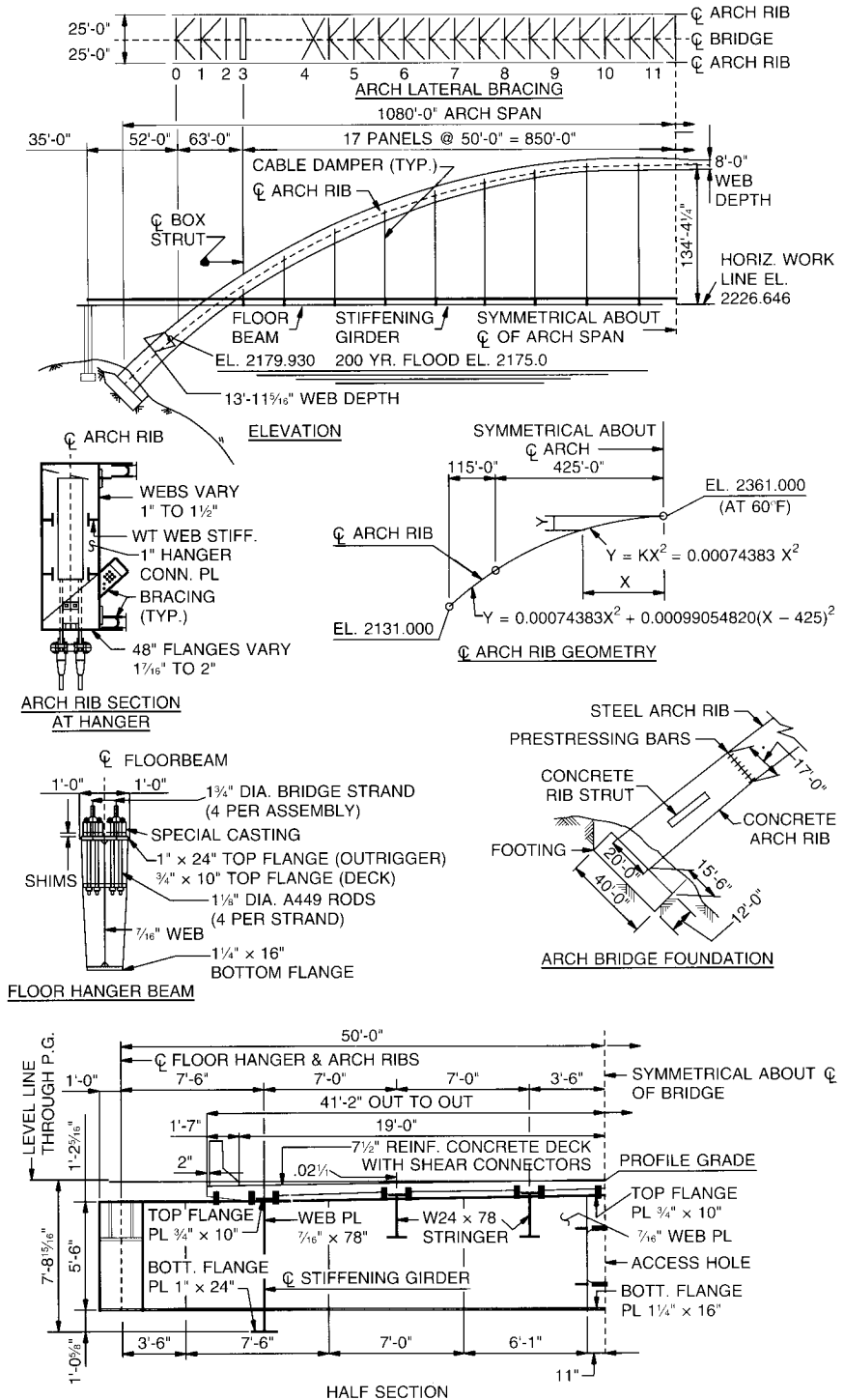


FIGURE 14.8 Details of Roosevelt Lake Bridge.



FIGURE 14.9

LEWISTON-QUEENSTON BRIDGE

LOCATION: Over the Niagara River between Lewiston, N.Y., and Queenston, Ontario

TYPE: Solid-ribbed deck arch, 23 panels at 41.6 ft

SPAN: 1,000 ft      RISE: 159 ft      RISE/SPAN = 1:6.3

NO. OF LANES OF TRAFFIC: 4

HINGES: 0      DEPTH: 13.54 ft      DEPTH/SPAN = 1:74

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway .....	5,700
Slabs for sidewalks .....	495
Railings and parapets .....	780
Floor steel for roadway and sidewalks .....	2,450
Floor bracing .....	110
Arch ribs .....	7,085
Arch bracing .....	1,060
Miscellaneous—utilities, excess, etc. ....	300
TOTAL .....	19,370

SPECIFICATION LIVE LOADING: HS20-S16-44

EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED  
STRUCTURE: 1,357 lb per ft

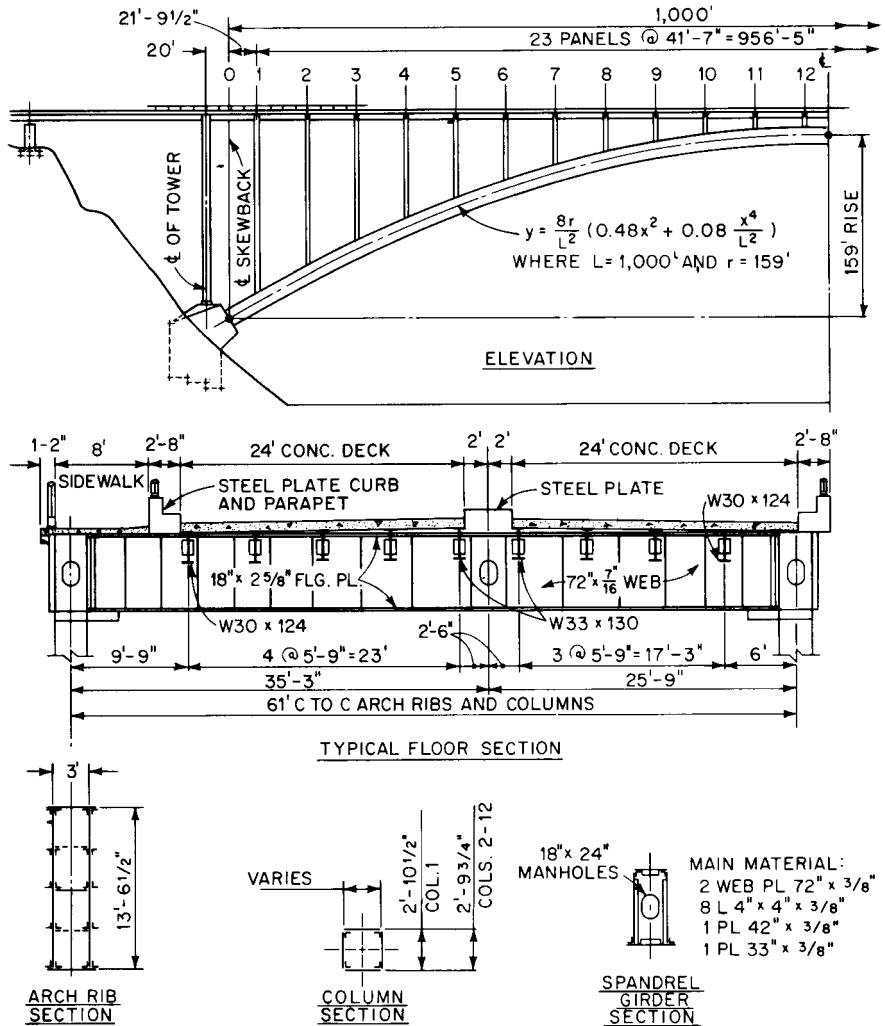
TYPES OF STEEL IN STRUCTURE:		%
Arch ribs .....	A440	100
Spandrel columns .....	A7	94
	A440	6
Rib bracing and end towers .....	A7	100
Floor system .....	A373 and A7	

OWNER: Niagara Falls Bridge Commission

ENGINEER: Hardesty & Hanover

FABRICATOR: Bethlehem Steel Co. and Dominion Steel and Coal Corp., Ltd., Subcontractor

DATE OF COMPLETION: Nov. 1, 1962



COMPOSITION OF ARCH RIB AND COLUMN SECTIONS			
	PANEL POINT 1	PANEL POINT 3	PANEL POINT 11
ARCH RIBS	6 COVER PL 54" x 7/8"	4 COVER PL 54" x 3/4"	4 COVER PL 54" x 9/16"
	8 L 8" x 8" x 1 1/16"	8 L 8" x 8" x 1"	8 L 8" x 8" x 1"
	2 WEB PL 162" x 15/16"	2 WEB PL 162" x 15/16"	2 WEB PL 162" x 15/16"
	2 WEB PL 146" x 5/8"	6 L 7" x 4" x 5/8"	6 L 7" x 4" x 5/8"
	6 L 7" x 4" x 5/8"	6 L 4" x 4" x 1/2"	6 L 4" x 4" x 1/2"
	6 L 4" x 4" x 1/2"	1 PL 33" x 13/16"	1 PL 33" x 13/16"
	1 PL 32" x 13/16"		
COLS.	2 COVER PL 37" x 1"	2 COVER PL 35" x 5/8"	2 COVER PL 27" x 1/2"
	4 L 6" x 6" x 9/16"	4 L 6" x 6" x 9/16"	4 L 6" x 6" x 9/16"
	2 WEB PL 32" x 5/8"	2 WEB PL 32" x 5/8"	2 WEB PL 32" x 5/8"

MATERIAL FOR  
ARCH RIBS,  
A440 STEEL

FIGURE 14.10 Details of Lewiston-Queenston Bridge.

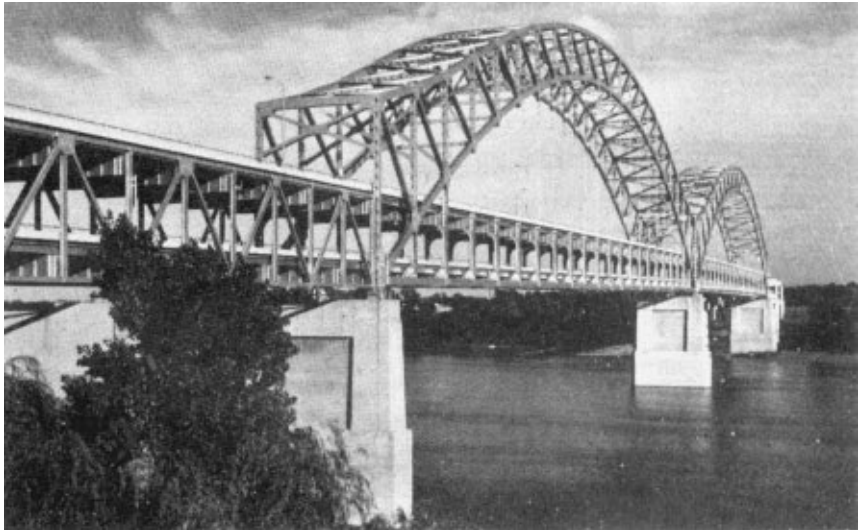


FIGURE 14.11

SHEARMAN MINTON BRIDGE

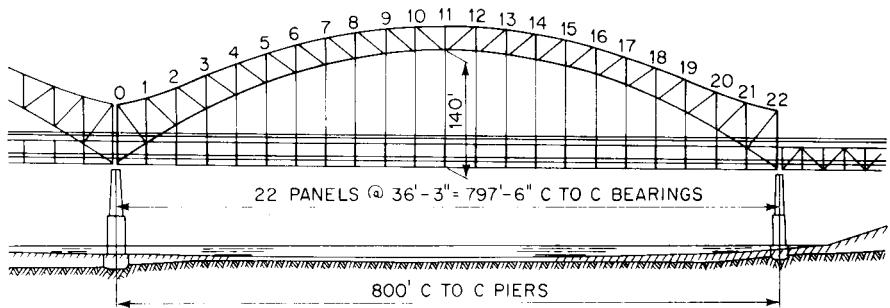
LOCATION: On Interstate 64 over the Ohio River between Louisville, Ky., and New Albany, Ind.  
TYPE: Tied, through, truss arch, 22 panels at 36.25 ft  
SPAN: 800 ft      RISE: 140 ft      RISE/SPAN = 1:5.7  
NO. OF LANES OF TRAFFIC: 6, double deck  
HINGES: 2      CROWN DEPTH: 30 ft      DEPTH/SPAN = 1:27  
AVERAGE DEAD LOAD:

	LB PER FT
Deck slab and surfacing for roadway .....	7,600
Slabs for sidewalks .....	1,656
Railings and parapets .....	804
Floor steel for roadway and sidewalks .....	2,380
Floor bracing .....	420
Arch trusses .....	3,400
Arch bracing .....	880
Arch hangers and bracing .....	160
Arch ties .....	1,040
Miscellaneous—utilities, excess, etc. (including future searing surface) .....	1,680
TOTAL .....	20,020

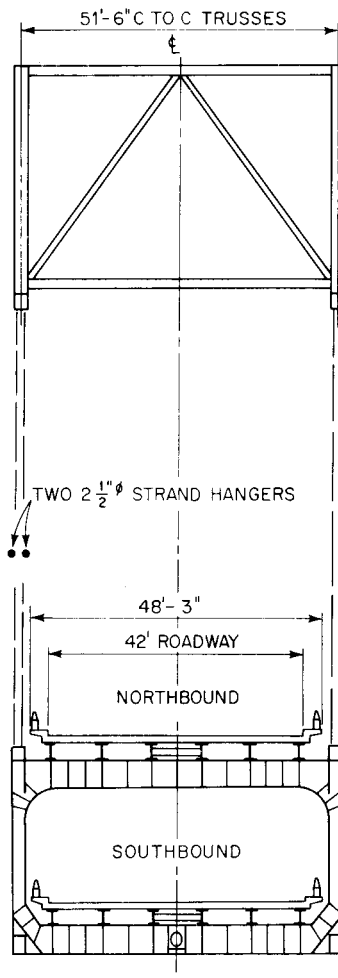
SPECIFICATION LIVE LOADING: H20-S16  
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 1,755 LB PER FT  
TYPES OF STEEL IN STRUCTURE:

		%
Arch trusses .....	A514	69
	A242	18
	A373	13
Floor system .....	A242	36
	A7	62
	A373	2

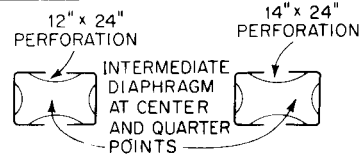
OWNER: Indiana Department of Transportation and Kentucky Transportation Cabinet  
ENGINEER: Hazelet & Erdal, Louisville, Ky.  
FABRICATOR: R. C. Mahon Co.  
DATE OF COMPLETION: Dec. 22, 1961, opened to traffic



## ELEVATION



## CROSS SECTION



## DIAGONALS:

U<sub>2</sub>L<sub>3</sub> (A242)

2 VERT. WEB PL 18" x 11/16"

2 PERF. COV. PL 26" x 3/8"

U<sub>10</sub>L<sub>11</sub> (A373)

2 VERT. WEB PL 15" x 5/8"

2 PERF. COV. PL 26" x 9/16"

## VERTICALS:

U<sub>2</sub>L<sub>2</sub> (A514)

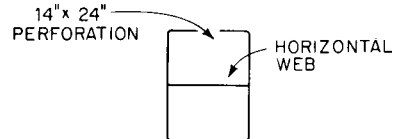
2 VERT. WEB PL 18" x 3/4"

2 PERF. COV. PL 26" x 11/16"

U<sub>11</sub>L<sub>11</sub> (A373)

2 VERT. WEB PL 15" x 3/8"

2 PERF. COV. PL 26 1/4" x 3/8"



## TOP CHORD:

U<sub>2</sub>U<sub>3</sub> (A242)

2 VERT. WEB PL 38" x 1 1/16"

1 HORIZ. WEB PL 24 7/8" x 15 5/16"

2 PERF. COV. PL 26" x 11/16"

U<sub>10</sub>U<sub>11</sub> (A514)

2 VERT. WEB PL 38" x 1 9/16"

1 HORIZ. WEB PL 23 7/8" x 1 1/16"

2 PERF. COV. PL 26" x 7/8"

## BOTTOM CHORD:

L<sub>2</sub>L<sub>3</sub> (A514)

2 VERT. WEB PL 38" x 1 9/16"

1 HORIZ. WEB PL 23 7/8" x 1"

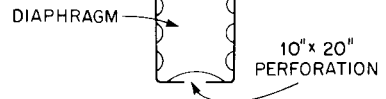
2 PERF. COV. PL 26" x 13/16"

L<sub>10</sub>L<sub>11</sub> (A373)

2 VERT. WEB PL 38" x 9/16"

1 HORIZ. WEB PL 25 7/8" x 9/16"

2 PERF. COV. PL 26" x 7/16"



## TIE:

L<sub>0</sub>L<sub>22</sub> (A514)

2 VERT. WEB PL 38" x 1 7/16"

2 PERF. COV. PL 26" x 1 1/2"

ALL SHOP WORK WELDED.  
SHOP AND FIELD CONNECTIONS:  
HIGH STRENGTH BOLTS

FIGURE 14.12 Details of Sherman Minton Bridge.

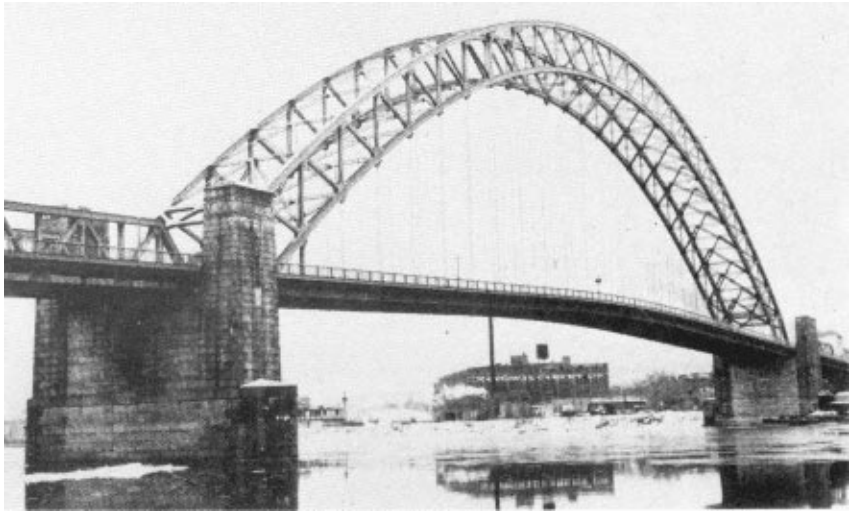


FIGURE 14.13

WEST END-NORTH SIDE BRIDGE

LOCATION: Pittsburgh, Pennsylvania, over Ohio River  
TYPE: Tied, through, truss arch, 28 panels at 27.8 ft  
SPAN: 778 ft      RISE: 151 ft      RISE/SPAN = 1:5.2  
NO. OF LANES OF TRAFFIC: 4, including 2 street-railway tracks  
HINGES: Two      CROWN DEPTH: 25      DEPTH/SPAN = 1:31

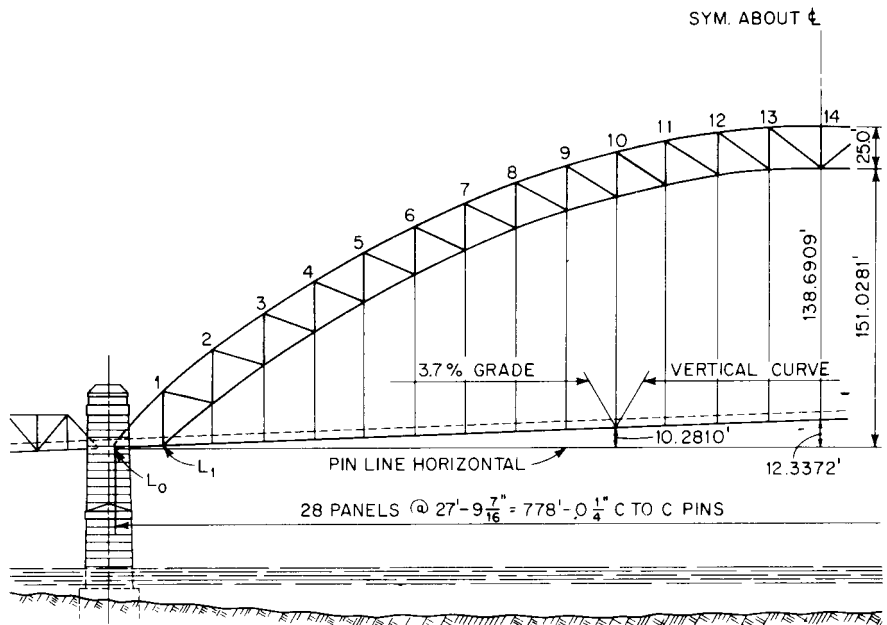
AVERAGE DEAD LOAD:	LB PER FT
Roadway, sidewalks, and railings .....	4,870
Floor steel and floor bracing .....	2,360
Arch trusses .....	4,300
Arch ties .....	2,100
Arch bracing .....	550
Hangers .....	360
Utilities and excess .....	600
TOTAL .....	15,140

SPECIFICATION LIVE LOADING: Allegheny County Truck & Street Car  
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED  
STRUCTURE: 1,790 lb per ft

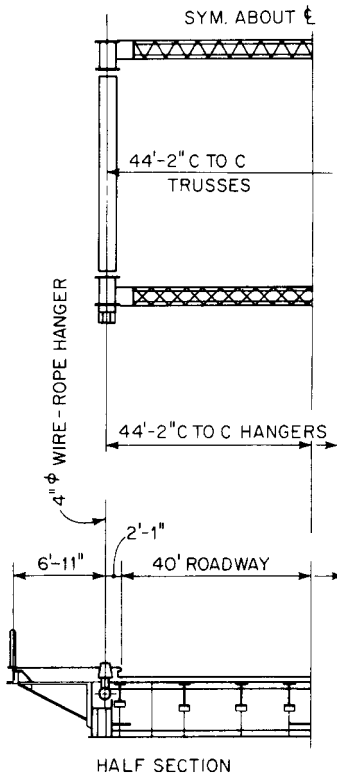
TYPES OF STEEL IN STRUCTURE:  
All main material in arch trusses and ties including splice material—silicon steel.  
Floor system and bracing ..... A7  
Hangers ..... Wire rope

OWNER: Pennsylvania Department of Transportation  
ENGINEER: Department of Public Works, Allegheny County  
FABRICATOR: American Bridge Division, U.S. Steel Corp.  
DATE OF COMPLETION: 1932





## ELEVATION

TRUSS MATERIAL AT  $\epsilon$  SPAN

## TOP CHORD

1 COV PL  $35" \times 11/16"$   
 4 L  $6" \times 6" \times 1/2"$   
 2 WEB PL  $34" \times 13/16"$   
 BATT PL  $40" \times 5/8"$  } BOTTOM  
 X LACING C6 x 13 } FLANGE

## BOTTOM CHORD

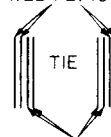
X LACING C6 x 13 } T & B  
 BATT PL  $40" \times 5/8"$  } FLANGE  
 4 L  $6" \times 6" \times 1/2"$   
 4 WEB PL  $36" \times 3/4"$   
 2 WEB PL  $36" \times 5/8"$   
 2 SIDE PL  $24" \times 1/2"$

## POSTS

4 L  $7" \times 3 1/2" \times 9/16"$   
 1 WEB PL  $18 1/2" \times 3/8"$

## DIAGONALS

4 L  $4" \times 4" \times 3/8"$   
 2 WEB PL  $18" \times 9/16"$   
 BATT PL  $18" \times 3/8"$  } T & B  
 SINGLE LACING } FLANGE  
 $2 3/4" \times 7/16"$

WEB PL  $40" \times 3/4"$ WEB PL  $40" \times 13/16"$ 

ALL SHOP AND FIELD WORK RIVETED

FIGURE 14.14 Details of West End-North Side Bridge.



FIGURE 14.15

FORT PITT BRIDGE

LOCATION: Pittsburgh, Pennsylvania, over the Monongahela River  
TYPE: Solid-ribbed, tied, through arch, 30 panels at 25 ft  
SPAN: 750 ft    RISE: 122.2 ft    RISE/SPAN = 1:6.2  
NO. OF LANES OF TRAFFIC: 4, each level of double deck  
HINGES: 2    DEPTH: 5.4 ft    DEPTH/SPAN = 1:139

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadways, slabs for sidewalks, railings and parapets, on both decks .....	16,100
Floor steel for roadway and sidewalks, on both decks .....	4,860
Floor bracing (truss bracing) .....	480
Arch ribs .....	5,480
Arch bracing .....	1,116
Arch hangers (included with rib and tie) .....	
Arch ties (trusses) .....	8,424
Miscellaneous—utilities, excess, etc .....	400
TOTAL .....	36,860

SPECIFICATION LIVE LOADING: HS20-S16-44  
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 2,500 lb per ft

TYPES OF STEEL IN STRUCTURE:		
Arch ribs and trussed ties .....	A242	64
	A7	36
Floor system .....	A242	90
	A7	10

OWNER: Pennsylvania Department of Highways  
ENGINEER: Richardson, Gordon and Associates  
FABRICATOR: American Bridge Division, U.S. Steel Corp.  
DATE OF COMPLETION: 1957

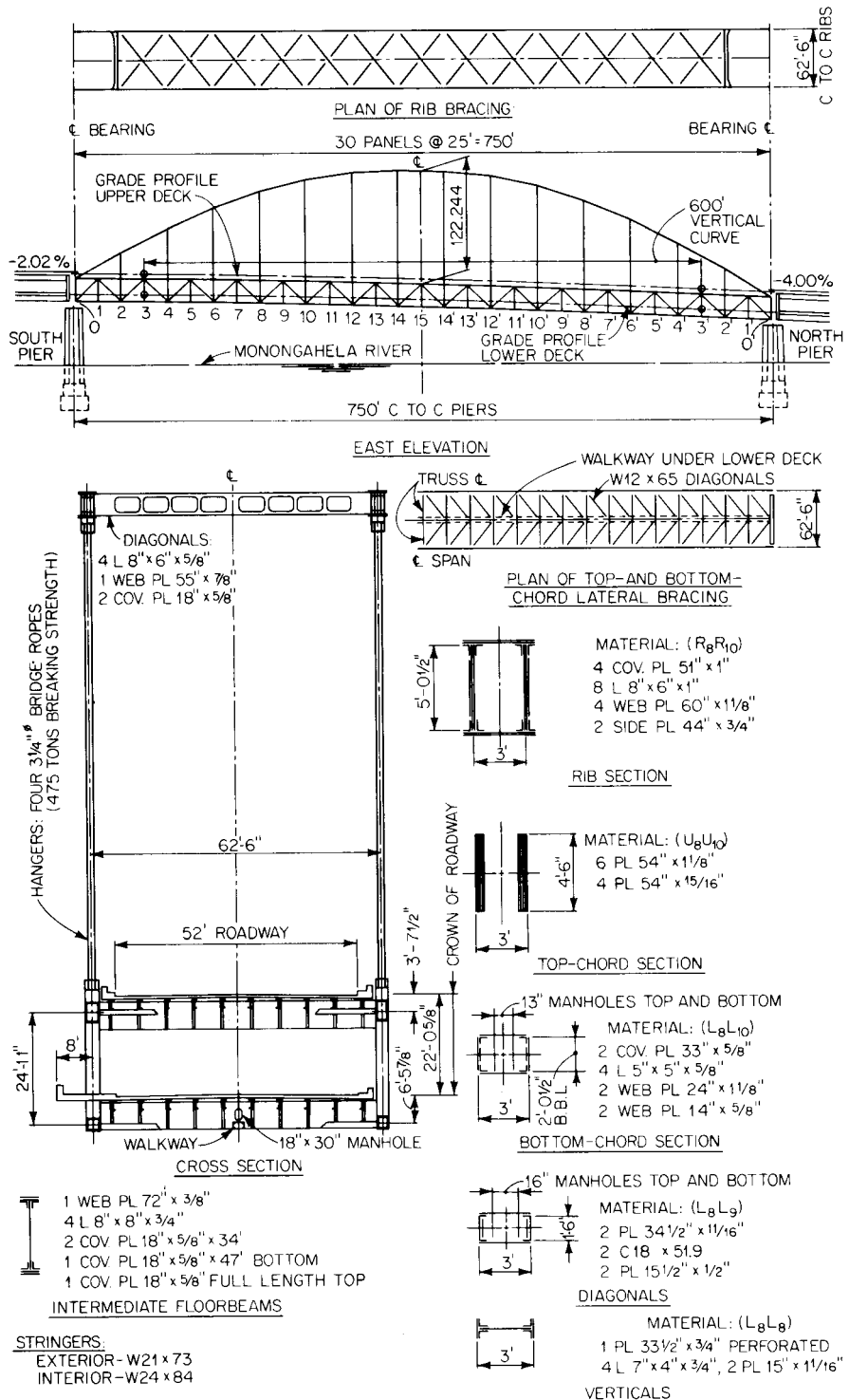


FIGURE 14.16 Details of Fort Pitt Bridge.



FIGURE 14.17

GLENFIELD BRIDGE

LOCATION: I-79 crossing of Ohio River at Neville Island, Pennsylvania

TYPE: Tied, through, solid-ribbed arch, 15 panels at 50 ft

SPAN: 750 ft      RISE: 124.4 ft      RISE/SPAN = 1:6

NO. OF LANES OF TRAFFIC: 6 plus 10-ft berms

HINGES: 0      CROWN DEPTH: 4 ft      DEPTH/SPAN = 1:187

AVERAGE DEAD LOAD:

	LB PER FT
Deck slab and surfacing for roadway .....	13,980
Railings and parapets .....	1,090
Floor steel for roadway .....	3,397
Floor bracing .....	392
Arch ribs .....	2,563
Arch bracing .....	1,639
Arch hangers .....	94
Arch ties .....	3,400
Miscellaneous—utilities, excess, etc. ....	589
TOTAL .....	27,144

SPECIFICATION LIVE LOADING: H20-S16-44

EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED

STRUCTURE: 1,920 lb per ft

TYPES OF STEEL IN STRUCTURE:

		%
Arch ribs and ties .....	A514	64
	A588	36
Ribs and bottom-lateral bracing .....	A36	100
Hangers .....	Wire rope	

OWNER: Pennsylvania Department of Transportation

ENGINEER: Richardson, Gordon and Associates

FABRICATOR: Bristol Steel and Iron Works, Inc. and Pittsburgh DesMoines Steel Co.

ERECTOR: American Bridge Division, U.S. Steel Corp.

DATE OF COMPLETION: 1976

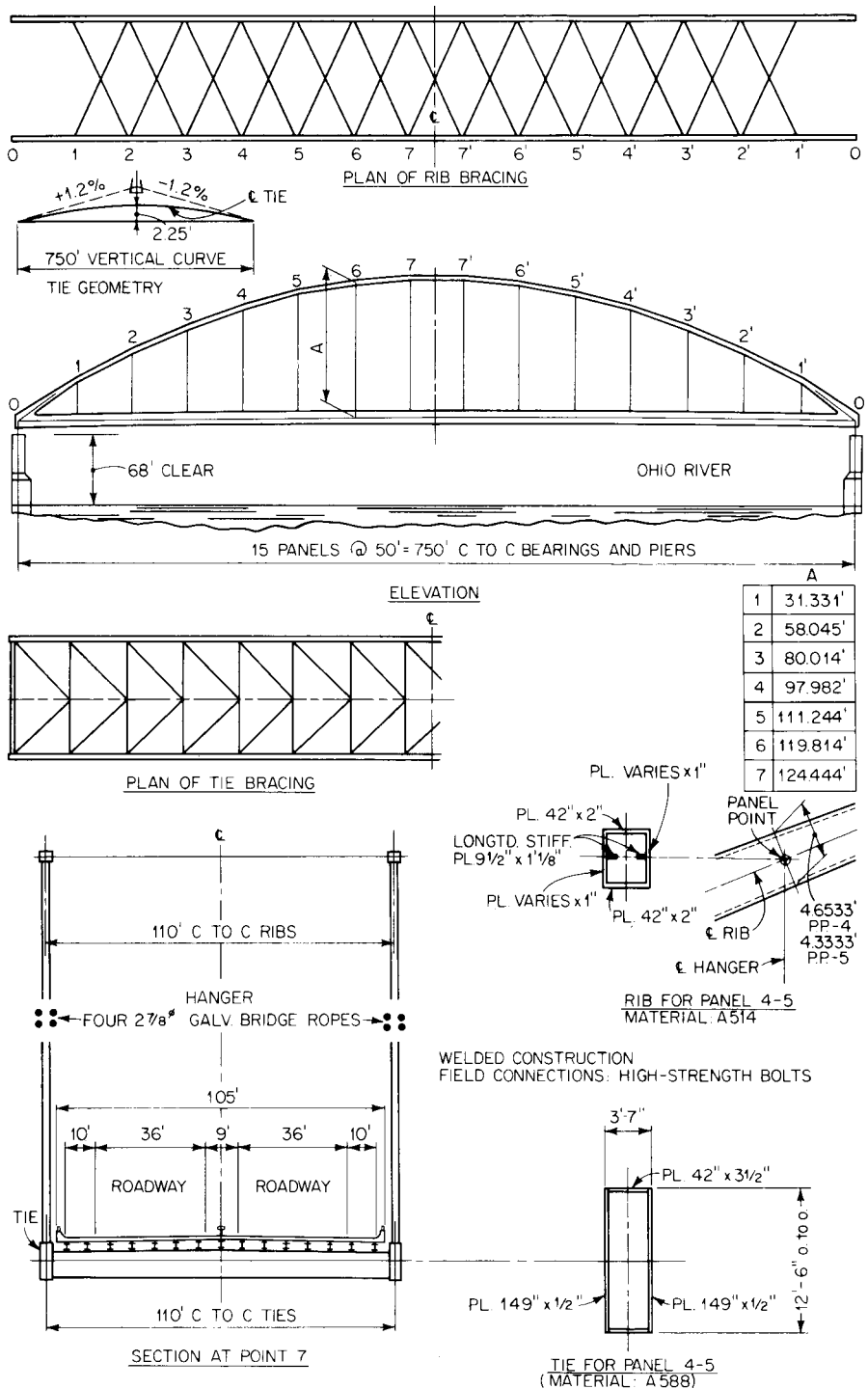


FIGURE 14.18 Details of Glenfield Bridge.

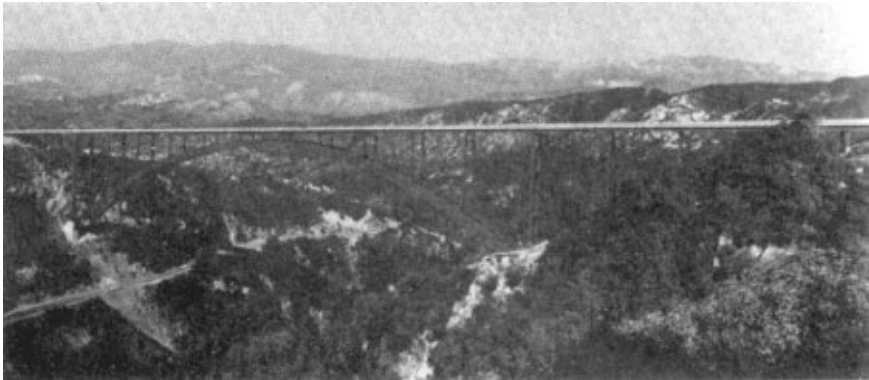


FIGURE 14.19

COLD SPRING CANYON BRIDGE

LOCATION: About 13.5 miles north of city limit of Santa Barbara, Calif.

TYPE: Solid-ribbed deck arch, 11 panels, 9 at 63.6 ft

SPAN: 700 ft      RISE: 119.2 ft      RISE/SPAN = 1:5.9

NO. OF LANES OF TRAFFIC: 2

HINGES: 2      DEPTH: 9 ft      DEPTH/SPAN = 1:78

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway .....	3,520
Railings and parapets .....	1,120
Floor steel for roadway .....	620
Floor bracing .....	75
Arch ribs .....	3,400
Arch bracing .....	530
Arch posts and bracing .....	210
TOTAL .....	9,475

SPECIFICATION LIVE LOADING: H20-S16-44

EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED STRUCTURE: 904 lb per ft

TYPES OF STEEL IN STRUCTURE:

Arch ribs .....	A373
Floor system .....	A373

OWNER: State of California

ENGINEER: California Department of Transportation

FABRICATOR: American Bridge Division, U.S. Steel Corp.

DATE OF COMPLETION: December, 1963

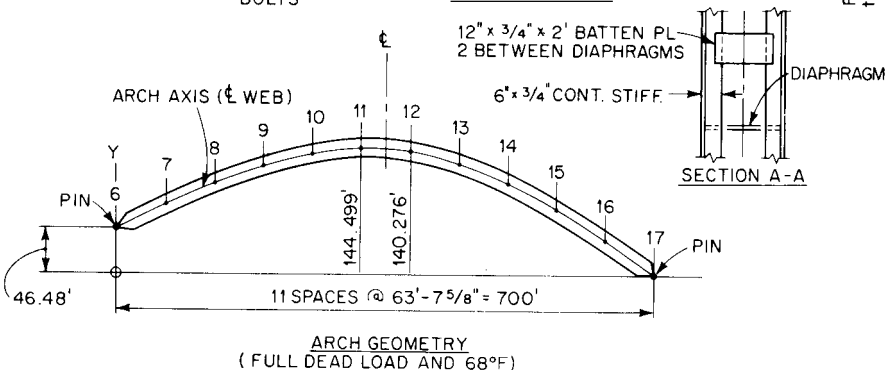
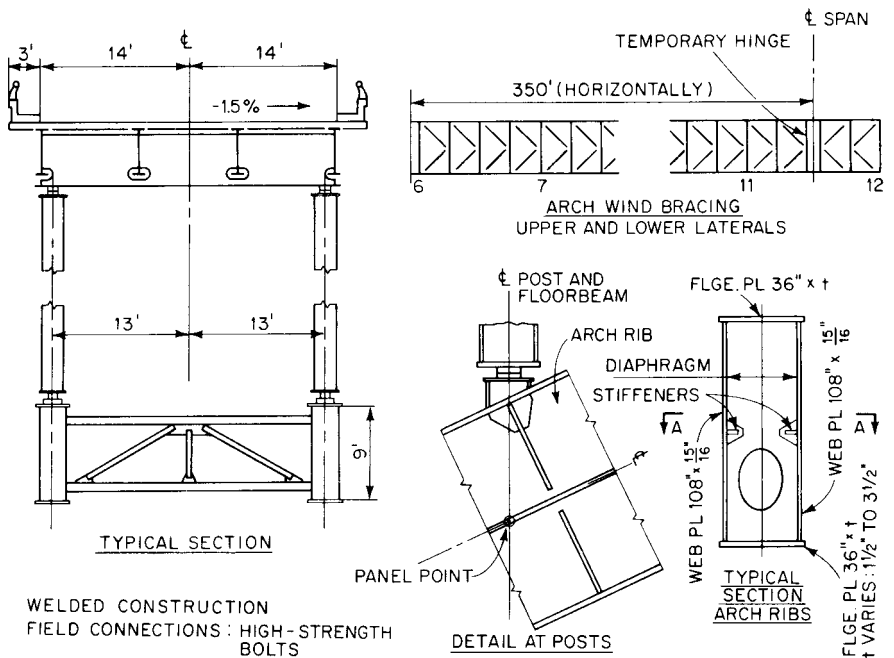
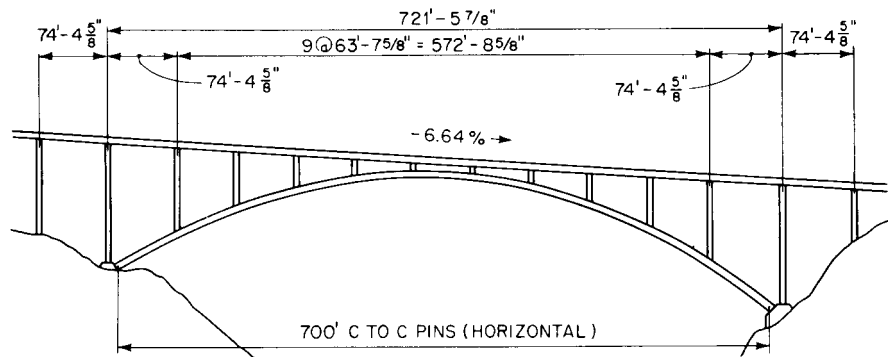


FIGURE 14.20 Details of Cold Spring Canyon Bridge.



FIGURE 14.21

BURRO CREEK BRIDGE

LOCATION: Arizona State Highway 93, about 75 miles southeast of Kingman, Arizona

TYPE: Trussed deck arch, 34 panels at 20 ft

SPAN: 680 ft      RISE: 135 ft      RISE/SPAN = 1:5.0

NO. OF LANES OF TRAFFIC: 2

HINGES: 2      CROWN DEPTH: 20 FT      DEPTH/SPAN = 1:34

Deck slab and surfacing for roadway .....	3,140
Slab for sidewalks .....	704
Railings and parapets .....	470
Floor steel for roadway .....	800
Floor bracing .....	203
Arch trusses .....	2,082
Arch bracing .....	580
Arch posts and bracing .....	608
TOTAL .....	8,587

SPECIFICATION LIVE LOADING: H20-S16-44

EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED

STRUCTURE: 1,420 lb per ft

TYPES OF STEEL IN STRUCTURE:

Arch trusses .....	A441	61
	A36	39
Other components .....	A36	

OWNER: Arizona Department of Transportation

ENGINEER: Bridge Division

FABRICATOR: American Bridge Division, U.S. Steel Corp.

DATE OF COMPLETION: Mar. 23, 1966



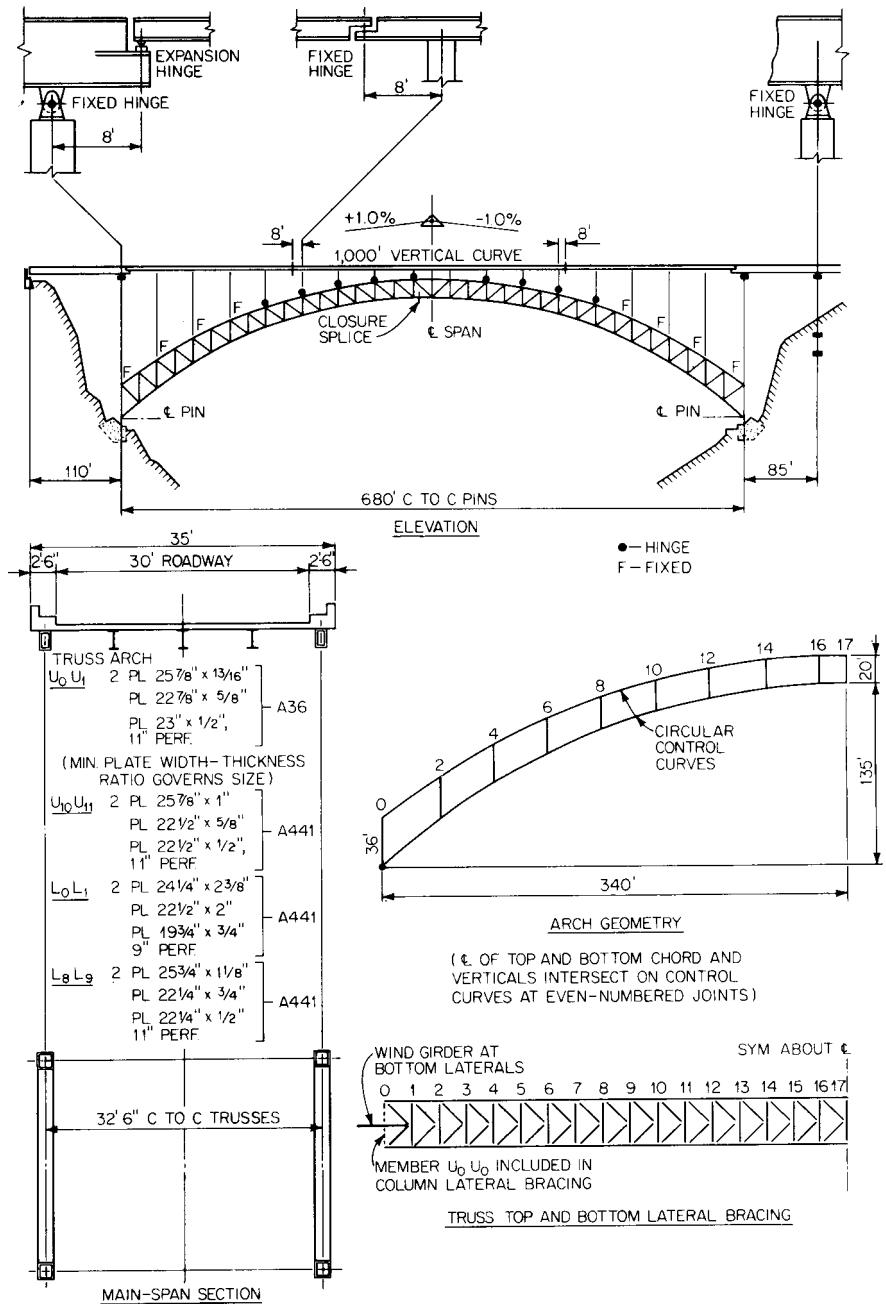


FIGURE 14.22 Details of Burro Creek Bridge.

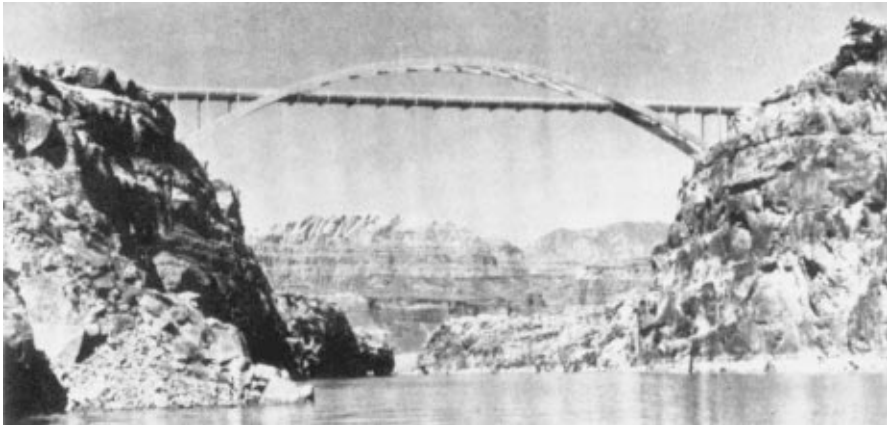


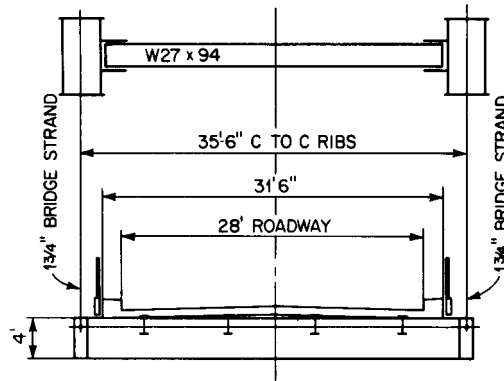
FIGURE 14.23

COLORADO RIVER ARCH BRIDGE

LOCATION: Utah State Route 95 over Colorado River, near Garfield-San Juan county line  
TYPE: Half-through, solid-ribbed arch, 21 panels, 19 at 27.5 ft  
SPAN: 550 ft      RISE: 90 ft      RISE/SPAN = 1:6.1  
NO. OF LANES OF TRAFFIC: 2  
HINGES: 0      DEPTH: 7 ft      DEPTH/SPAN = 1:79

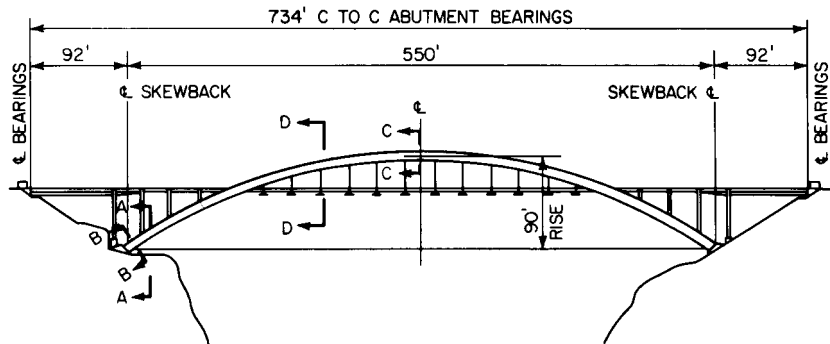
AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway .....	2,804
Railings and parapets .....	605
Floor steel for roadway .....	615
Floor bracing .....	60
Arch ribs .....	2,200
Arch bracing .....	370
Arch hangers and bracing .....	61
TOTAL .....	6,715

SPECIFICATION LIVE LOADING: HS20-44  
EQUIVALENT LIVE + IMPACT LOADING ONE EACH ARCH FOR FULLY LOADED  
STRUCTURE: 952 lb per ft  
STEEL IN THIS STRUCTURE: A36, except arch hangers, which are bridge strand.  
OWNER: State of Utah  
ENGINEER: Structures Division, Utah Department of Transportation  
FABRICATOR: Western Steel Co., Salt Lake City, Utah  
DATE OF COMPLETION: Nov. 18, 1966



SECTION D-D

ALL SHOP WORK WELDED  
 ALL FIELD CONNECTIONS:  $\frac{7}{8}$ "  $\Phi$  HIGH-STRENGTH BOLTS



ELEVATION

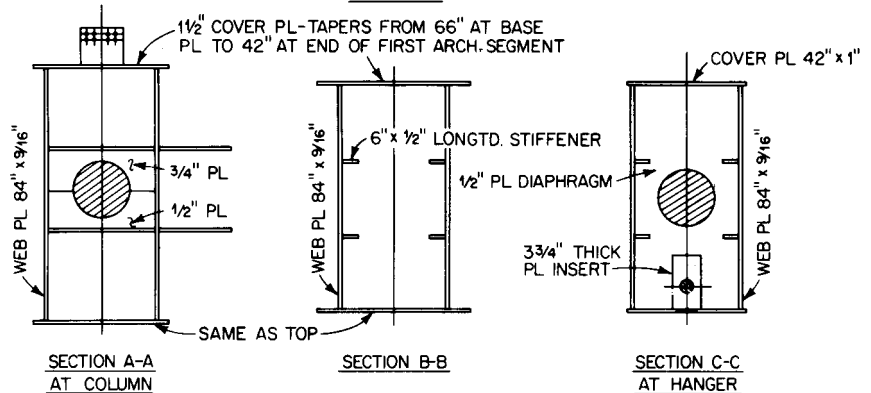


FIGURE 14.24 Details of Colorado River Arch Bridge.



FIGURE 14.25

SMITH AVENUE HIGH BRIDGE

LOCATION: Smith Avenue over Mississippi River in St. Paul, Minnesota

TYPE: Solid-ribbed, tied, deck arch, 26 panels at 40 ft

SPAN: 520 ft      RISE: 109.35 ft      RISE/SPAN = 1:4.8

NO. OF LANES OF TRAFFIC: 2

HINGES: 0      DEPTH: 8 ft      DEPTH/SPAN = 1:65

AVERAGE DEAD LOAD:	LB PER FT
Deck slab, sidewalks, railings and surfacing for roadway .....	9,370
Floor steel for roadway .....	920
Arch ribs .....	2,810
Arch bracing .....	360
Arch ties .....	200
Arch columns and bracing .....	300
TOTAL .....	13,960

SPECIFICATION FOR LIVE LOADING: HS20-44

EQUIVALENT LIVE + IMPACT LOADING FOR ARCH FOR FULLY LOADED

STRUCTURE: 2,250 lb per ft

TYPES OF STEEL IN STRUCTURE:

Arch ribs and ties .....	A588
Floor system .....	A588

OWNER: Minnesota Department of Transportation

ENGINEER: Stgar Roscoe Fausch/T. Y. Lin International

FABRICATOR: Lunda Construction

DATE OF COMPLETION: July 25, 1987

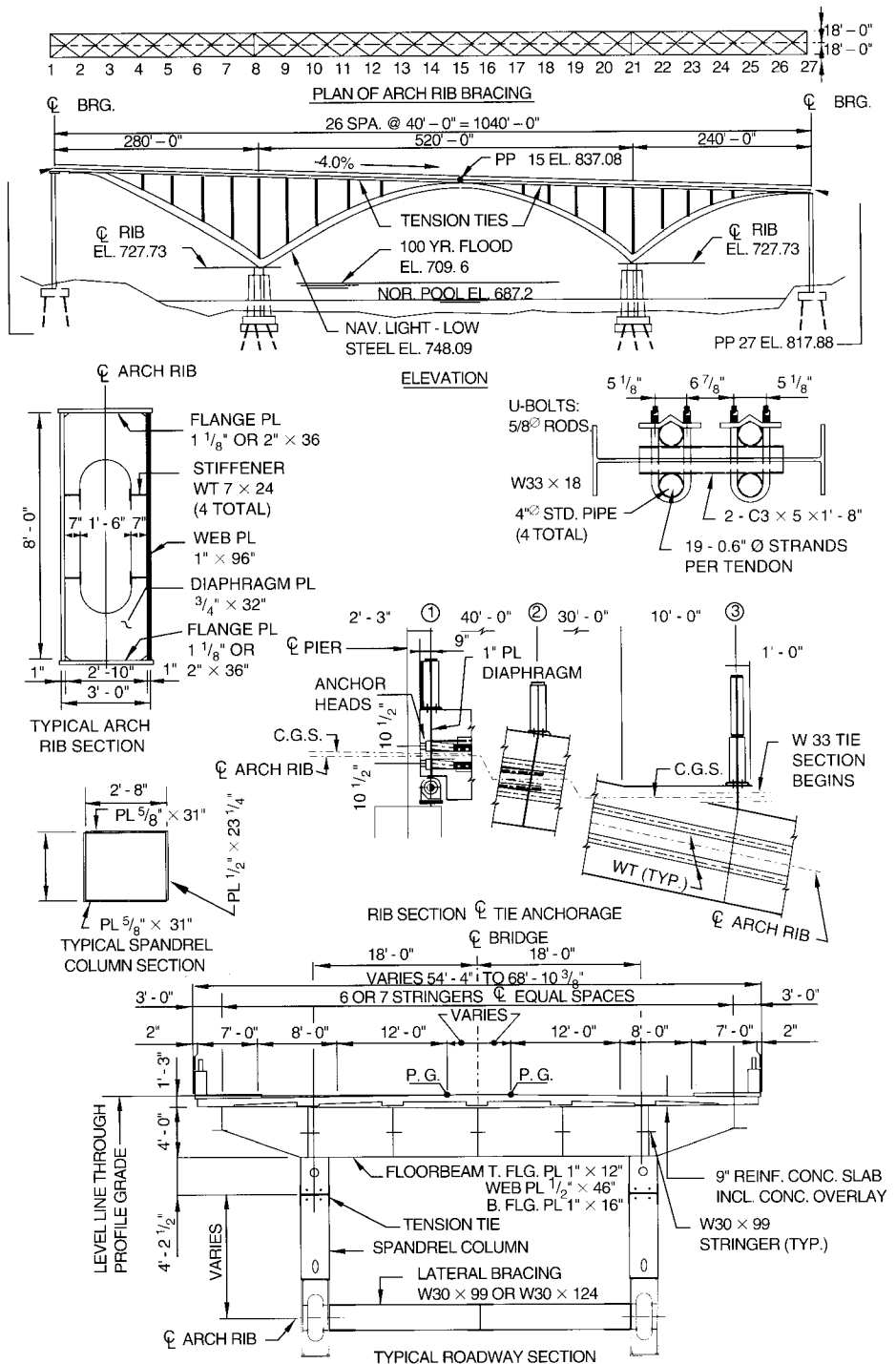


FIGURE 14.26 Details of Smith Avenue High Bridge.



FIGURE 14.27

LEAVENWORTH CENTENNIAL BRIDGE

LOCATION: Leavenworth, Kansas, over Missouri River  
TYPE: Tied, through, solid-ribbed arch, 13 panels at 32.3 ± ft  
SPAN: 420 ft      RISE: 80 ft      RISE/SPAN = 1:5.2  
NO. OF LANES OF TRAFFIC: 2  
HINGES: 0      DEPTH: 2.8 ft      DEPTH/SPAN = 1:150

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway .....	2,710
Railings and parapets (aluminum) .....	32
Floor steel for roadway .....	820
Floor steel for sidewalks .....	202
Floor bracing .....	116
Arch ribs .....	986
Arch bracing .....	420
Arch hangers and bracing .....	200
Arch ties .....	1,104
Miscellaneous—utilities, excess, etc. ....	110
TOTAL .....	6,700

SPECIFICATION LOADING: H20-S16-44  
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED  
STRUCTURE: 885 lb per ft

TYPES OF STEEL IN STRUCTURE:		
Arch .....	A7	25
	A242	75
Ties .....	A242	
Floor system and bracing .....	A7	
Hangers .....	A7	

OWNER: Kansas Department of Transportation and Missouri Highway and Transportation  
Department  
ENGINEER: Howard, Needles, Tammen & Bergendoff  
FABRICATOR: American Bridge Division, U.S. Steel Corp.  
DATE OF COMPLETION: April, 1955

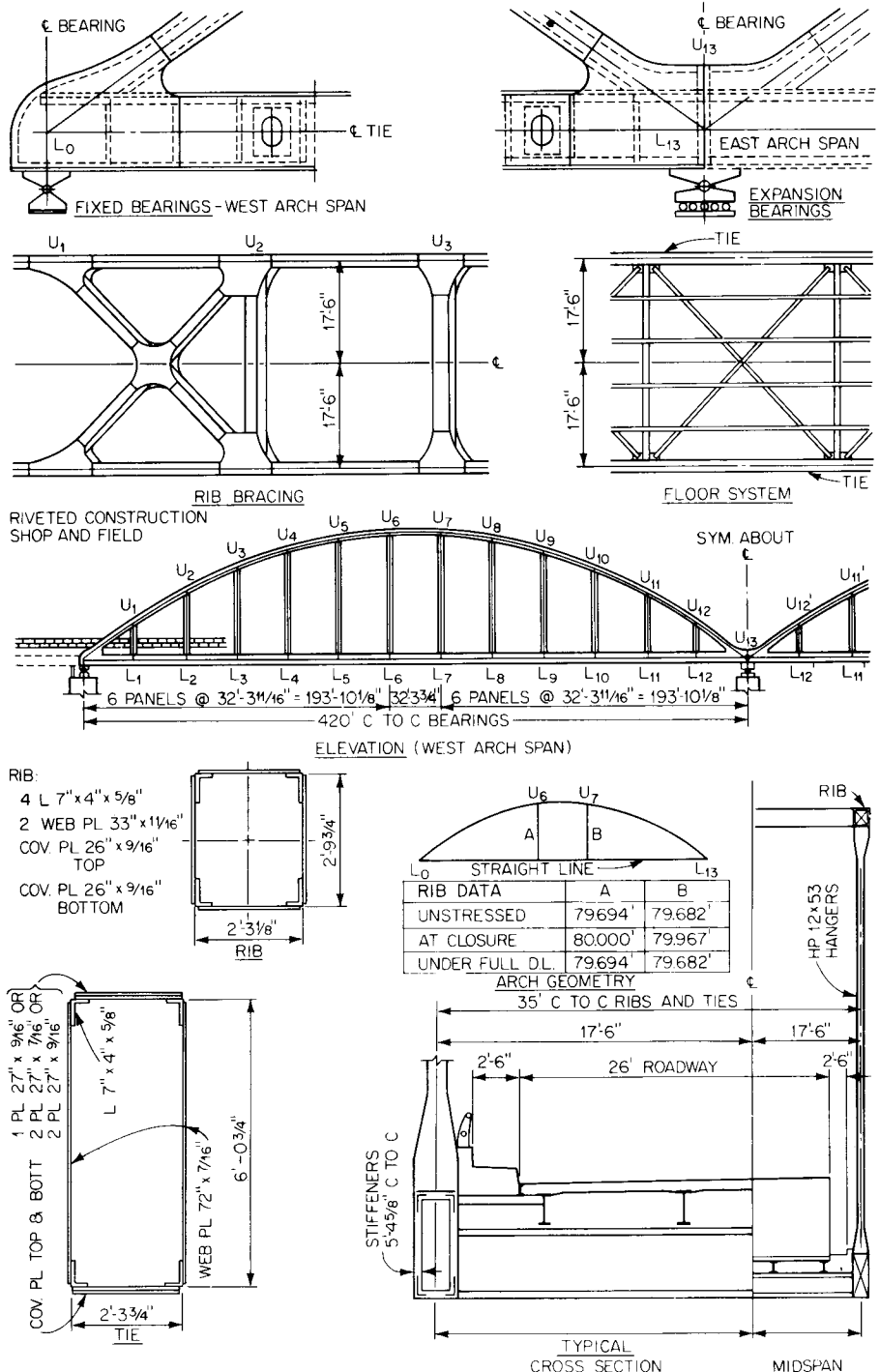


FIGURE 14.28 Details of Leavenworth Centennial Bridge.

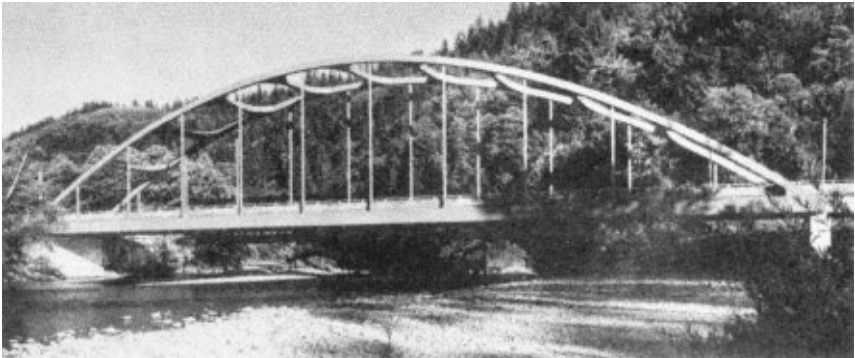


FIGURE 14.29

NORTH FORK STILLAGUAMISH RIVER BRIDGE

LOCATION: Cicero, Snobomish County, Wash.  
TYPE: Tied, through, solid-ribbed arch, 11 panels at 25.3 ft  
SPAN: 278.6 ft    RISE: 51 ft    RISE/SPAN = 1:5.5  
NO. OF LANES OF TRAFFIC: 2  
HINGES: 0    DEPTH: 2 ft    DEPTH/SPAN = 1:139

AVERAGE DEAD LOAD:	LB PER FT
Deck slab and surfacing for roadway .....	2,500
Railings and parapets .....	1,000
Floor steel for roadway .....	475
Floor bracing .....	59
Arch ribs .....	684
Arch bracing .....	400
Arch hangers or posts and bracing .....	83
Arch ties .....	799
TOTAL .....	6,000

SPECIFICATION LIVE LOADING: HS20  
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED  
STRUCTURE: 1,055 lb per ft  
TYPES OF STEEL IN STRUCTURE:

Arch ribs and ties .....	A36	28
	A440 and A441	72
Floor system .....	A36	63
	A440 and A441	37
Hangers .....	A36	

OWNER: Washington Department of Transportation  
ENGINEER: Bridges and Structures Division, Washington DOT  
FABRICATOR: Northwest Steel Fabricators, Vancouver, Wash.  
GENERAL CONTRACTOR: Dale M. Madden, Inc., Seattle, Wash.  
DATE OF COMPLETION: 1966



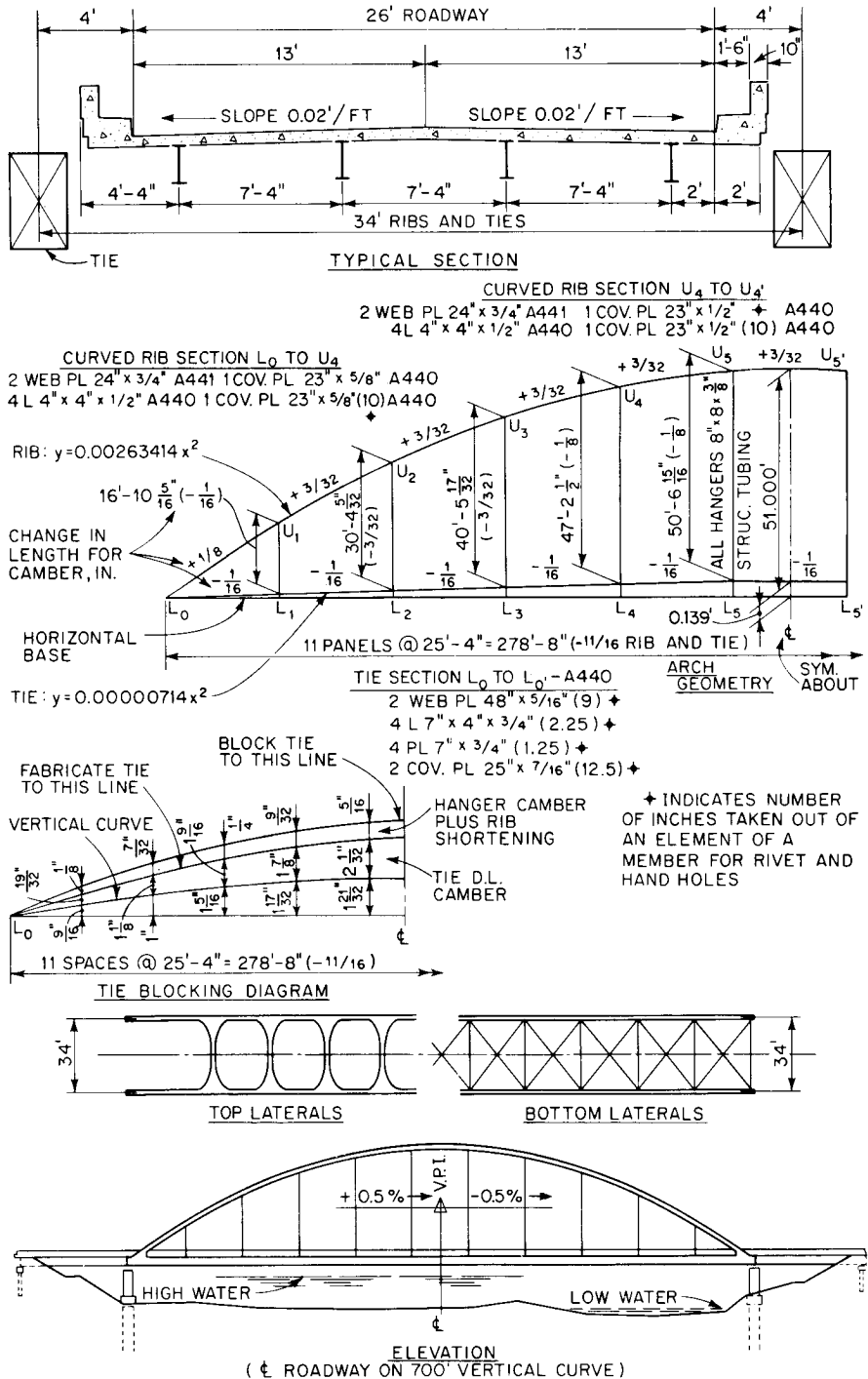


FIGURE 14.30 Details of North Fork Stillaguamish River Bridge.



FIGURE 14.31

SOUTH STREET BRIDGE OVER I-84		
LOCATION: South Street over Route I-84, Middlebury, Conn.		
TYPE: Solid-ribbed deck arch, 7 panels, 5 at 29 ft		
SPAN: 193 ft      RISE: 29 ft      RISE/SPAN = 1:6.7		
NO. OF LANES OF TRAFFIC: 2		
HINGES: 2      DEPTH: 3.3 ft      DEPTH/SPAN = 1:58		
AVERAGE DEAD LOAD:		LB PER FT
Deck slab and surfacing for roadway .....		4,000
Railings and parapets .....		500
Floor steel for roadway .....		560
Arch ribs .....		1,070
Arch bracing .....		230
Arch posts and bracing .....		20
Miscellaneous—utilities, excess, etc .....		50
TOTAL .....		6,430
SPECIFICATION LIVE LOADING: H20-S16-44		
EQUIVALENT LIVE + IMPACT LOADING ON EACH ARCH FOR FULLY LOADED		
STRUCTURE: 1,498 lb per ft		
TYPES OF STEEL IN STRUCTURE:		
Arch ribs .....		A373 98
		A242 2
Floor system .....		A373
OWNER: Connecticut Department of Transportation		
ENGINEER: Connecticut DOT		
FABRICATOR: The Ingalls Iron Works Co.		
DATE OF COMPLETION: 1964		

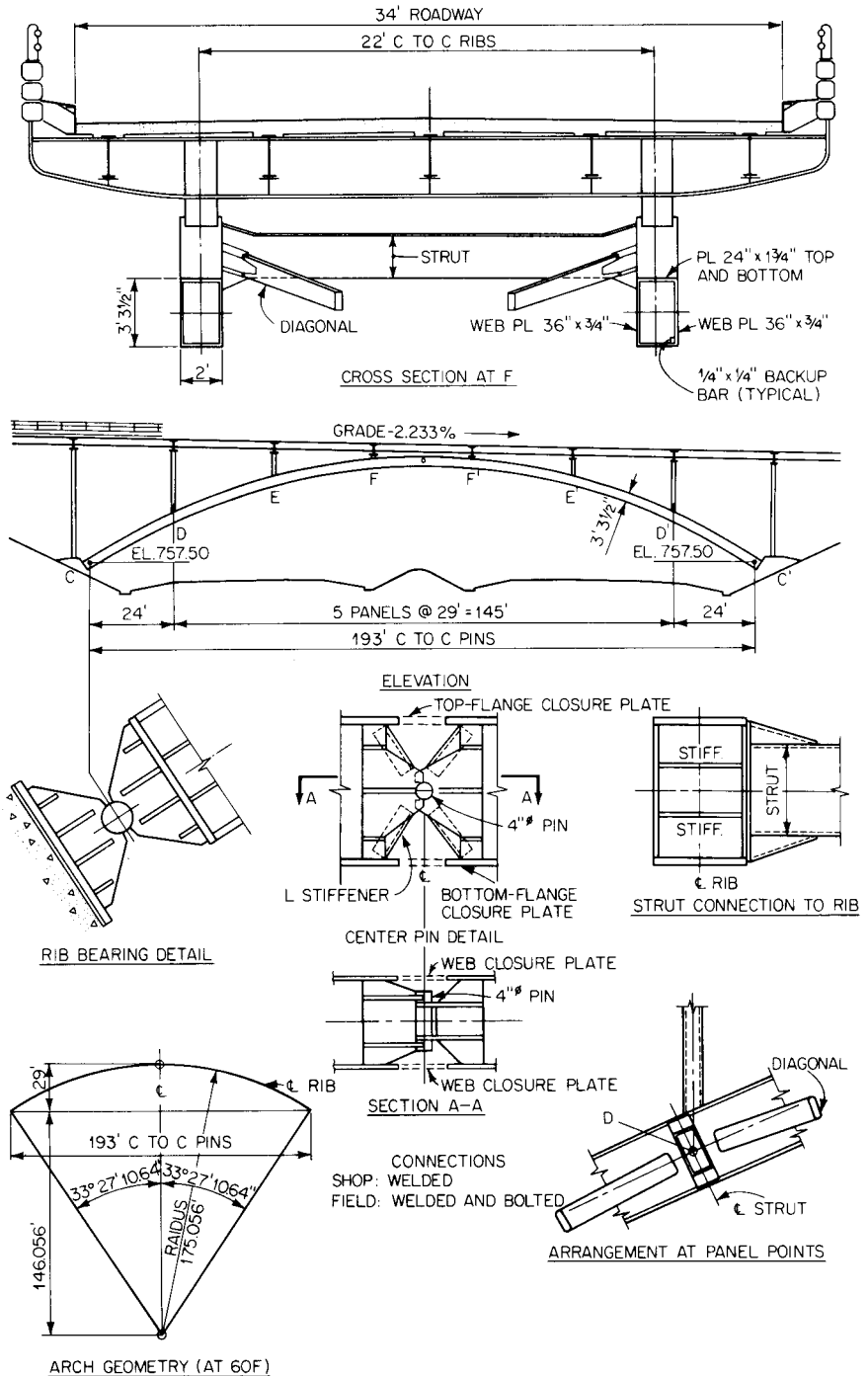


FIGURE 14.32 Details of South Street Bridge over I-84.

## 14.9 GUIDELINES FOR PRELIMINARY DESIGNS AND ESTIMATES

The usual procedure followed by most designers in preliminary designs of bridges involves the following steps:

1. Preliminary layout of structure
2. Preliminary design of floor system and calculation of weights and dead load
3. Preliminary layout of bracing systems and estimates of weights and loads
4. Preliminary estimate of weight of main load-bearing structure
5. Preliminary stress analysis
6. Check of initial assumptions for dead load

**Preliminary Weight of Arch.** The ratios given in Art. 14.6 can guide designers in making a preliminary layout with nearly correct proportions. The 16 examples of arches in Art. 14.8 also can be helpful for that purpose and, in addition, valuable in making initial estimates of weights and dead loads.

Equations (14.1) and (14.2), shown graphically in Fig. 14.33, were developed to facilitate estimating weights of main arch members.

For a true arch of low-alloy, high-strength steel (without ties), the ratio  $R$  of weight of rib to total load on the arch may be estimated from

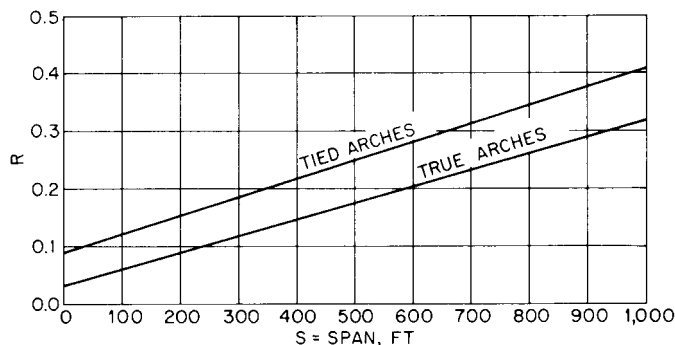
$$R = 0.032 + 0.000288S \quad (14.1)$$

where  $S$  = span, ft.

For a tied arch of low-alloy, high-strength steel, the ratio  $R$  of weight of rib and tie to total load on the arch may be estimated from

$$R = 0.088 + 0.000321S \quad (14.2)$$

This equation was derived from a study of seven of the structures in Art. 14.8 that are tied arches made of low-alloy, high-strength steels predominantly for ribs, trusses, and ties. Equation (14.1), however, is not supported by as many examples of actual designs and may give values on the high side for truss arches. Despite the small number of samples, both equations should give reasonably accurate estimates of weight for preliminary designs and cost esti-



**FIGURE 14.33** Chart gives ratio  $R$  of weight of rib, or rib and tie, to total load for arches fabricated predominantly with low-alloy steel.

mates of solid-ribbed and truss arches and for comparative studies of different types of structures.

With  $R$  known, the weight  $W$ , lb per ft, of arch, or arch plus tie, is given by

$$W = \frac{R(D + L)}{1 - R} \quad (14.3)$$

where  $D$  = dead load on arch, lb per ft, excluding weight of arch, or arch plus tie and  $L$  = equivalent live load plus impact, lb per ft, on arch when the structure is fully loaded.  $D$  is determined from preliminary design of bridge components other than arches and ties.

**Effect of Type of Steel on Arch Weights.** The following approximate analysis may be used to determine the weight of arch rib or arch rib and tie based on the weight of arch for some initial design with one grade of steel and an alternative for some other grade with different physical properties.

Let  $F_b$  = basic unit stress for basic design, ksi

$F_a$  = basic unit stress for alternative design, ksi

$D$  = dead load, lb per ft, excluding weight of rib, or rib and tie

$L$  = equivalent live load plus impact, lb per ft, for fully loaded structure

$W_b$  = weight of rib, or rib and tie, lb per ft for basic design

$W_a$  = weight of rib, or rib and tie, lb per ft, for alternate design

$P_b$  = total load, lb, carried by 1 lb of rib, or 1 lb of rib and tie, for basic design

$P_a$  = total load, lb, carried by 1 lb of rib, or 1 lb of rib and tie, for alternate design

The load supported per pound of member may be assumed proportional to the basic unit stress. Hence,

$$\frac{P_b}{P_a} = \frac{F_b}{F_a} \quad (14.4)$$

Also, the load per pound of member equals the ratio of the total load, lb per ft, on the arch to weight of member, lb per ft. Thus,

$$P_b = \frac{D + L + W_b}{W_b} = 1 + \frac{D + L}{W_b} \quad (14.5)$$

Similarly, and with use of Eq. (14.4),

$$P_a = 1 + \frac{D + L}{W_a} = \frac{P_b F_a}{F_b} \quad (14.6)$$

Solving for the weight of rib, or rib plus tie, gives

$$W_a = \frac{(D + L)F_b/F_a}{P_b - F_b/F_a} \quad (14.7)$$

Use of the preceding equations will be illustrated by application to the Sherman Minton Bridge (Figs. 14.11 and 14.12). Its arches were fabricated mostly of A514 steel. Assume that a preliminary design has been made for the floor system and bracing. A preliminary estimate of weight of truss arch and tie is required.

From the data given for this structure in Art. 14.8, the total load per arch, excluding truss arch and tie, is

$$D + L = \frac{15,580}{2} + 1755 = 9545 \text{ lb per ft}$$

From Eq. (14.2), or from Fig. 14.33, with span  $S = 797.5$  ft, if the arch had been constructed of low-alloy steel, the ratio of weight of rib and tie to total load would be about

$$R = 0.088 + 0.000321 \times 797.5 = 0.34$$

By Eq. (14.3), the weight of rib and tie, if made of low-alloy steel, would have been

$$W_b = 9545 \times \frac{0.34}{1 - 0.34} = 4900 \text{ lb per ft}$$

For the A514 steel actually used for the arch, an estimate of weight of rib and tie may be obtained from Eqs. (14.5) and (14.7). When these formulas are applied, the following basic unit stresses may be used:

Normal grades of low-carbon steel— $F = 18$  ksi

Low-alloy, high-strength steels— $F = 24$  ksi

A514 high-strength steel— $F = 45$  ksi

These stresses make some allowance for reductions due to thickness, reductions due to compression, and other similar factors. A check against a number of actual designs indicates that these values give about the correct ratios for the above grades of steel. Accordingly, the calculations for estimating weight of rib plus tie of A514 steel are as follows:

$$\frac{F_b}{F_a} = \frac{24}{45} = 0.53$$

By Eq. (14.5),

$$P_b = 1 + \frac{9545}{4900} = 2.95$$

Then, by Eq. (14.7), the weight of rib and tie is estimated at

$$W_a = 9545 \times \frac{0.53}{2.95 - 0.53} = 2090 \text{ lb per ft}$$

Use 2100 in preliminary design calculations.

Weight of truss arch and tie as constructed as  $\frac{1}{2}(3400 + 1040) = 2200$  lb per ft, checking the estimate within about 5%.

## 14.10 BUCKLING CONSIDERATIONS FOR ARCHES

Since all arches are subjected to large compressive stresses and also usually carry significant bending moments, stability considerations must be addressed. The American Association of State Highway and Transportation Officials (AASHTO) "Standard Specifications for Highway Bridges" contain provisions intended to ensure stability of structures.

For true arches, the design should provide stability in the vertical plane of the arch, with the associated effective buckling length, and also provide for moment amplification effects. For tied arches with the tie and roadway suspended from the arch, moment amplification in

the arch rib need not be considered. For such arches, the effective length can be considered the distance along the arch between hangers. However, with the relatively small cross-sectional area of the cable hangers, the effective length may be slightly longer than the distance between hangers.

For prevention of buckling in the lateral direction, a lateral bracing system of adequate stiffness should be provided. Effective lengths equal to the distance between rib bracing points are usually assumed. Special consideration should be given to arch-end portal areas. Lateral torsional buckling for open I-section ribs is much more critical than for box ribs and must be prevented.

Local buckling of web and flange plates is avoided by designs conforming to the limiting plate width-to-thickness ratios in the AASHTO standard specifications. If longitudinal web stiffeners are provided, additional criteria for their design are available (Art. 11.12).

## 14.11 EXAMPLE—DESIGN OF TIED-ARCH BRIDGE

The typical calculations that follow are based on the design of the 750-ft, solid-ribbed arch for the Glenfield Bridge (Figs. 14.17 and 14.18). The original design was in accordance with AASHTO “Standard Specifications for Highway Bridges,” 1965. The example has been revised in general in accordance with provisions in the 16th edition of the AASHTO specifications, inclusive of the 1998 “Interim Specifications,” for the service-load-design method (ASD) (Sec. 11). Conditions that do not meet current code provisions are noted.

The structure is a tied, through arch with 50-ft-long panels. The tie has a constant depth of 12 ft 6 in. The arch rib is segmental (straight between panel points) and tapers in depth from 7 ft at the springing line to 4 ft at the crown.

The tied arches are assumed to be fabricated so that dead loads, except member dead loads between panel points, are carried by axial stresses. The floor system is assumed to act independently. Thus, it does not participate in the longitudinal behavior of the arches. The following illustrates the design of selected components and some typical structural details.

### 14.11.1 Design of Floor System

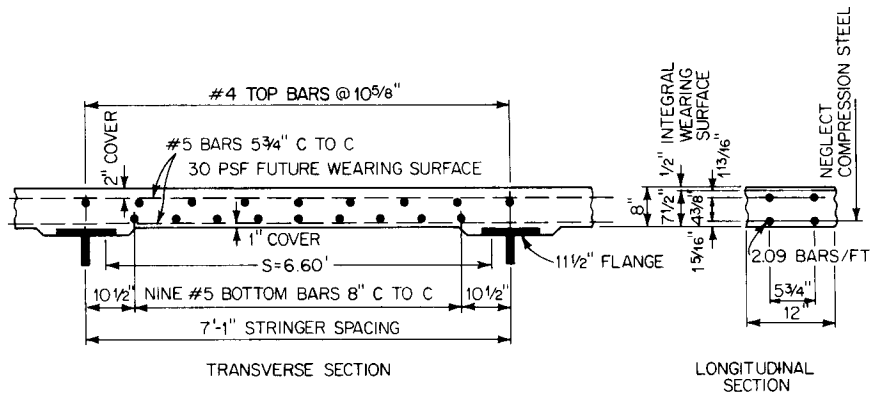
The floor system is designed for HS20-44 loading. See Art. 11.4.

**Slab Design.** Assumed cross sections of the roadway slab are shown in Fig. 14.34. Design is based on an allowable reinforcing steel stress  $f_s = 20$  ksi and an allowable compressive concrete stress  $f_c = 0.4f'_c = 1.2$  ksi, for 28-day strength  $f'_c = 3$  ksi and modular ratio  $n = 9$ . The effective slab span  $S = 7.08 - 0.96/2 = 6.60$  ft.

*Slab Dead Load, psf*

$$\begin{array}{rcl} \text{Concrete: } 150 \times 0.667 & = & 100 \\ \text{Future wearing surface} & = & \underline{30} \\ \text{Total} & = & 130 \end{array}$$

Calculations of bending moments in the slab take into account continuity. For dead load, maximum moment is taken to be  $M = wS^2/10$ , where  $w$  is the dead load, kips per ft<sup>2</sup>. For live load,  $M = 0.8P_{20}(S + 2)/32$ , where 0.8 is the continuity factor, and  $P_{20}$  is a wheel load, 16 kips for an H20 truck. The impact moment is obtained by applying a factor of 30% to the live-load moment.



**FIGURE 14.34** Concrete deck of tied-arch Glenfield Bridge (Fig. 14.18) is supported on steel stringers. (Current AASHTO standard specifications require that concrete cover over top reinforcing steel be at least 2½ in instead of the 2 in used for the Glenfield Bridge.)

*Slab Moments, ft-kips*

Dead load:	$0.130(6.60)^2/10$	= 0.566
Live load:	$0.8 \times 16(6.60 + 2)/32$	= 3.440
Impact:	$0.30 \times 3.440$	= 1.032
Total		= 5.038

For the area of main reinforcement,  $A_s = 0.65 \text{ in}^2/\text{ft}$ , the distance from the center of gravity of compression in the slab to center of gravity of tension steel is computed to be  $jd = 5.02 \text{ in}$ . For this moment arm, maximum stresses are computed to be  $f_c = 1.017 \text{ ksi}$  for the concrete and  $f_s = 18.8 \text{ ksi}$  in the steel, and are considered acceptable.

The percentage of the main reinforcement that must be supplied for distribution reinforcing in the longitudinal direction is computed from  $220/\sqrt{S}$ , but need not exceed 67%.

$$\frac{220}{\sqrt{6.60}} = 85.6 \quad \text{Use } 67\%$$

Required area of distribution steel =  $0.67 \times 0.65 = 0.44 \text{ in}^2/\text{ft}$ . No. 5 bars 8 in c to c supply an area of  $0.47 \text{ in}^2/\text{ft} > 0.44$ .

**Stringer Design.** The stringers are designed as three-span continuous, noncomposite beams on rigid supports 50 ft apart (Fig. 14.35). (While floorbeams do not provide perfectly rigid supports, the effects of their flexibility have been studied and found to be small.) The 101-ft-wide roadway is assumed to be carried by 15 stringers, each a  $W33 \times 130$ , made of A36 steel. Allowable stresses are taken to be  $F_b = 20 \text{ ksi}$  in bending and  $F_v = 12 \text{ ksi}$  in shear.

The dead load is considered to consist of three parts: the initial weight of stringer and concrete deck (Table 14.1a), the superimposed weight of median, railings, parapets, and future wearing surface (Table 14.1b), and a concentrated load at midspan from a diaphragm and connections (Fig. 14.35).

Superimposed dead load per stringer =  $4.124/15 = 0.275 \text{ kip per ft}$

Total uniformly distributed dead load =  $0.868 + 0.275 = 1.143 \text{ kips per ft}$

Concentrated dead load at midspan = 0.36 kips



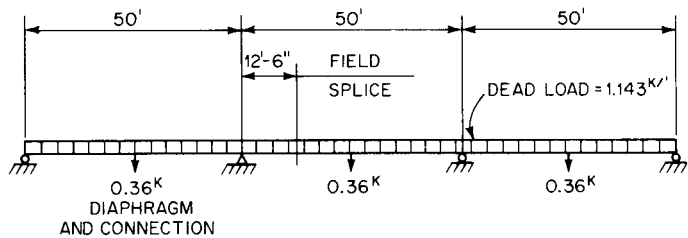


FIGURE 14.35 Stringers of Glenfield Bridge are continuous over three spans, with a splice in the center span.

For live load, the number of wheels to be carried by each stringer is determined from  $S/5.5$ , where  $S$  is the slab span, ft. Thus,

$$\text{Live-load distribution} = \frac{7.08}{5.5} = 1.288 \text{ wheels}$$

The impact fraction is computed from  $I = 50/(L + 125) \leq 0.3$ , where  $L$  is the loaded length of member, ft.

$$I = \frac{50}{50 + 125} = 0.286$$

The typical stringer may be analyzed by moment distribution, with a computer or with appropriate tabulated influence ordinates. Design moment and reaction for the end spans are given in Table 14.2.

For a  $W33 \times 130$  with section modulus  $S = 406 \text{ in}^3$ , the maximum bending stress is

$$f_b = \frac{M}{S} = 639 \times \frac{12}{406} = 18.9 \text{ ksi} < 20\text{—OK}$$

The maximum shear stress  $f_v$  in the web of the 33.10-in-deep beam is considerably less

TABLE 14.1 Dead Load per Stringer, kips per ft

(a) Initial dead load	
W33 $\times$ 130	= 0.130
8-in concrete slab: $0.150(8/12)7.08$	= 0.708
Concrete haunch	= 0.030
Total	0.868 kip per ft
(b) Superimposed dead load on 15 stringers	
Bridge median	= 0.030
Two railings: $2 \times 0.030$	= 0.060
Two parapets	= 1.004
Future wearing surface: $0.03 \times 101$	= 3.030
Total	4.124



38.3 in<sup>2</sup>, which has to be reduced by the percentage of flange area removed that exceeds 15%. Hole diameter =  $\frac{7}{8} + \frac{1}{8} = 1$  in.

*Hole Areas, in<sup>2</sup>*

Flange:  $1 \times 0.855 \times 2 = 1.710$  (two holes)

Web:  $1 \times 0.580 = 0.580$  (per hole)

The fraction of flange area removed in excess of 15% is:

$$1.710 / (11.51 \times 0.855) - 0.15 = 0.024$$

The adjusted gross moment of inertia is then calculated as follows:

$$\text{Flange reduction} = 11.51 \times 0.855 \times 0.024 = 0.24 \text{ in}^2 \text{ per flange}$$

$$\text{Reduced } I = 6710 - (2 \times 0.24 \times 16.122^2) = 6585 \text{ in}^4$$

AASHTO specifications require that splices be designed for the average of the calculated stress, or moment, and the allowable capacity of the member, but not less than 75% of the capacity. In addition, the fatigue stress range must be within the allowable.

With an allowable bending stress of 20 ksi, the moment capacity of the reduced section of the stringer is

$$M_c = \frac{F_b I}{c} = 20 \times \frac{6585}{16.55 \times 12} = 663 \text{ ft-kips}$$

Thus, the section must be designed for at least  $0.75 \times 663 = 497$  ft-kips.

The maximum moment at the splice is 198 ft-kips. Hence, the average moment for splice design is

$$M_{av} = \frac{1}{2}(198 + 663) = 431 \text{ ft-kips} < 497 \text{ ft-kips}$$

Consequently, the splice should be designed for 497 ft-kips.

The shear capacity of the web is

$$V_C = 12 \times 33.1 \times 0.580 = 230 \text{ kips}$$

Thus, the section could be designed for a shear of at least  $0.75 \times 230 = 173$  kips.

The maximum shear at the splice is 48.8 kips. Hence, the average shear for splice design is

$$V_{av} = \frac{1}{2}(48.8 + 230) = 139.4 \text{ kips} < 173$$

Design for the average shear capacity, however, would be unduly conservative, since web thickness at the splice is not governed by shear stress. Since both shear and moment are directly related to the load, it appears reasonable to use a design shear equal to the maximum shear increased by the same proportion as the design moment.

$$V = 48.8 \times \frac{497}{198} = 122.5 \text{ kips}$$

**Stringer Web Splice.** The size of splice plates required for the web (Fig. 14.37) is determined by the requirements for moment. The bolts are designed for shear and moment.

The moment to be carried by the bolts consists of two components. One is the moment due to the eccentricity of the shear.

$$M_v = 122.5 \times \frac{3.25}{12} = 33.2 \text{ ft-kips}$$

The other is the proportion of the 497-ft-kip design moment carried by the web. The web moment may be calculated by assuming that moments are proportional to moment of inertia. With a clear depth of 31.38 in between flanges, the web has a moment of inertia.

$$I_w = \frac{0.580(31.38)^3}{12} = 1494 \text{ in}^4$$

Consequently, the web moment is

$$M_w = 497 \times \frac{1494}{6585} = 112.8 \text{ ft-kips}$$

Thus, the web bolts must be designed for a moment of  $112.8 + 33.2 = 146.0$  ft-kips.

For determining bolt loads, the polar moment of inertia is first computed, as the sum of the moments of inertia of the bolts about two perpendicular axes (Fig. 14.37).

*Polar Moment of Inertia*

$$\begin{aligned} 2 \times 2 \times 270 &= 1080 \\ 9 \times 2(1.5)^2 &= \frac{41}{I_B = 1121 \text{ in}^2} \end{aligned}$$

The maximum bolt loads are resolved into horizontal and vertical components  $P_H$  and  $P_V$ , respectively.

$$P_H = \frac{Mc_1}{I_B} = \frac{146.0 \times 12 \times 12}{1,121} = 18.75 \text{ kips}$$

$$P_V = \frac{Mc_2}{I_B} = \frac{146.0 \times 12 \times 1.5}{1,121} = 2.34 \text{ kips}$$

Also, shear imposes a vertical load on the 18 bolts of

$$P_s = \frac{122.5}{18} = 6.81 \text{ kips}$$

Hence, the total load on the outermost bolt is the resultant

$$P = \sqrt{(6.81 + 2.34)^2 + 18.75^2} = 20.86 \text{ kips} > 18.0 \text{ kips}$$

Because of changes in the AASHTO specifications regarding splice design, the example indicates that the web bolts are overstressed by 16%. Actually, the bolts are adequate to carry the design loads, since the example is based on the higher “75% capacity” moment and shear values. For new construction, the design should be altered to eliminate the calculated overstress.

The web splice plates should be at least 27 in long to accommodate the 18 bolts in two rows of 9 each, with 3-in pitch (Fig. 14.37). If  $\frac{3}{8}$ -in-thick plates are selected, the area is more than adequate to resist the 122.5-kip shear. For resisting moment, the two plates supply a moment of inertia of

$$I = 2 \times 0.375 \left( \frac{27^3}{12} \right) = 1230 \text{ in}^4$$

Consequently, the maximum bending stress in the plates is

$$f_b = \frac{M_w c}{I} = 112.8 \times 12 \times \frac{13.5}{1230} = 14.85 \text{ ksi} < 20 \text{—OK}$$

The shear stress for maximum design shear is

$$f_v = \frac{P}{A} = \frac{122.5}{2 \times 27 \times 0.375} = 6.05 \text{ ksi} < 12 \text{ ksi—OK}$$

The two  $27 \times \frac{3}{8}$ -in plates are satisfactory for strength.

For fatigue, the range of moment carried by the web is

$$M_w = (198 + 196) \frac{1494}{6585} = 89.4 \text{ ft-kips}$$

The maximum bending-stress range in the gross section of the web splice plates then is

$$f_b = \frac{89.4 \times 12 \times 13.5}{1230} = 11.80 \text{ ksi}$$

Fatigue in base-metal gross section at high-strength-bolted, slip-critical connections is classified by AASHTO as category B. For a redundant-load-path structure and 2,000,000 loading cycles, the allowable stress range is 18 ksi > 11.80 ksi. Thus, the two  $27 \times \frac{3}{8}$ -in plates are satisfactory for fatigue.

**Stringer Flange Splice.** Each stringer flange is spliced with a  $\frac{1}{2}$ -in-thick plate on the outer surface and two  $\frac{9}{16}$ -in-thick plates on the inner surface, as indicated in Fig. 14.38. They are required to resist the design moment less the moment carried by the web splice. Thus, the splice moment is  $497 - 112.8 = 384.2$  ft-kips. With a stringer depth of 33.10 in and flange thickness of 0.86 in, the moment arm of the flange plates is about  $33.10 - 0.86 = 32.24$  in. Hence, the force, tension or compression, in each flange is

$$T = C = 384.2 \times \frac{12}{32.34} = 142.6 \text{ kips}$$

With an allowable stress of 20 ksi, the plate area required is

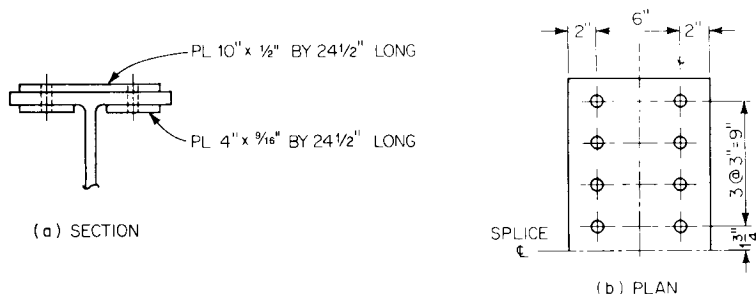


FIGURE 14.38 Flange splice for stringer.

$$A = \frac{142.6}{20} = 7.13 \text{ in}^2$$

Table 14.3 indicates the net area supplied by the flange splice plates.

The flange splice plates must be checked for fatigue. The range of live-load moment at the splice is  $198 + 196 = 394$  ft-kips. The stress range in the flanges then is

$$f = \frac{394 \times 12 \times 16.55}{6585} = 11.9 \text{ ksi} < 18 \text{ ksi}$$

The plates are satisfactory in fatigue.

The number of bolts required to transmit the flange force in double shear is  $142.6/18.0 = 7.92$ . The eight bolts supplied (Fig. 14.38) are satisfactory.

**Design of Interior Floorbeam.** Floorbeams, spaced 50 ft c to c, are designed as hybrid girders over the center 60 ft of their 110-ft spans. The web is made of A588 steel ( $F_y = 50$  ksi). Its depth varies from 109.3 in at centerline of ties to 116 in at centerline of bridge, to accommodate the cross slope of the roadway. For the flange, A514 steel ( $F_y = 100$  ksi) as well as A588 is used, to keep flange thickness constant over the full width of bridge.

The dead and live loads on a typical interior floorbeam are indicated in Fig. 14.39. To the live load, an impact factor of  $50/(110 + 125) = 0.213$  must be applied. For maximum moments and shears, however, live loads are reduced 25% because more than three lanes are loaded. Table 14.4 indicates the maximum moments and shears in the interior floorbeam.

The hybrid section is used in the region of maximum moment. Its properties required for bending analysis are given in Table 14.5.

**Flange Check.** Based on these properties, the bending stress for maximum moment is

$$f_b = \frac{M}{S} = \frac{22,353 \times 12}{6789} = 39.51 \text{ ksi}$$

The allowable bending stress is governed by the buckling capacity of the compression flange and strength of the web. To account for the latter, AASHTO standard specifications require application of a reduction factor  $R$  to the allowable buckling capacity [Art. 11.19 and Eq. (11.71)]. The allowable compressive stress for a homogeneous beam is

$$F_b = 0.55F_y = 0.55 \times 100 = 55 \text{ ksi}$$

For use in Eq. (11.71),

**TABLE 14.3** Net Area of Stringer Flange Plates, in<sup>2</sup>

0.500(10 - 2 × 1)	= 4.00
2 × 0.5625(4 - 1)	= 3.38
Total	= 7.38 > 7.13—OK

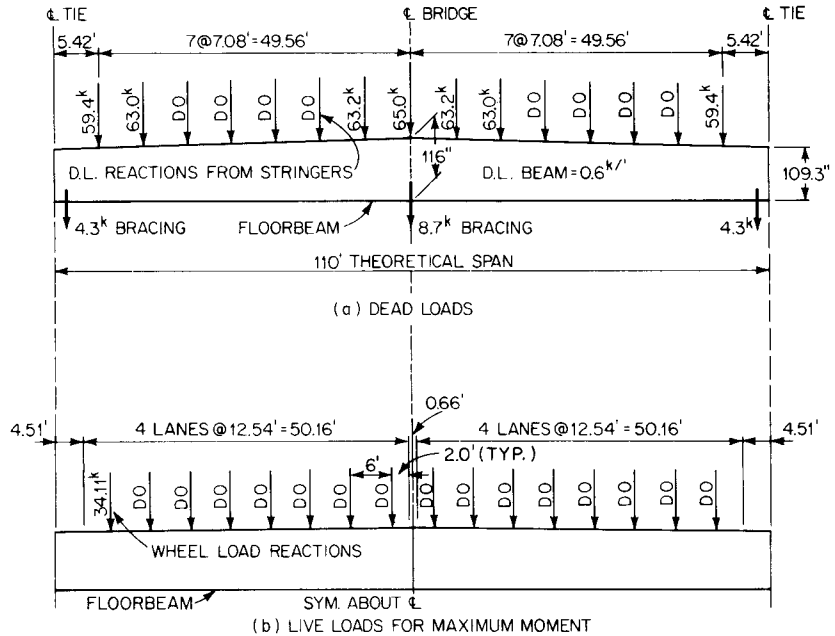


FIGURE 14.39 Loads on an interior floorbeam of Glenfield Bridge.

$$\alpha = \frac{F_{yw}}{F_{yf}} = \frac{50}{100} = 0.5$$

$$\beta = \text{ratio of web area to tension-flange area} = \frac{65.25}{48} = 1.36$$

$$\psi = \frac{c}{2c} = \frac{60}{120} = 0.5$$

Substitution in Eq. (11.71) yields  $R = 0.939$ . Hence, the allowable stress is

$$F'_b = RF_b = 0.939 \times 55 = 51.6 \text{ ksi} > 39.51$$

The flange is satisfactory.

TABLE 14.4 Maximum Moments and Shears in Interior Floorbeam

	Moment, ft-kips, at bridge center	Shear, kips, at tie
Dead load	14,691	510.7
Live load	6,316	209.4
Impact	1,346	44.6
Total	22,353	764.7

**TABLE 14.5** Properties of Hybrid Section of Floorbeam

Section	Steel	$F_y$	Area	$d$	$I$
Web: $116 \times \frac{1}{16}$	A588	50	65.25	. . .	73,167
Flanges: $2-24 \times 2$	A514	100	96.00	59.0	334,176
			161.25		407,343
Section modulus $S = 407,343/60 = 6,789 \text{ in}^3$					

The width-thickness ratio  $b/t$  of the compression flange may not exceed 24 or

$$\frac{b}{t} = \frac{103}{\sqrt{f_b}} = \frac{103}{\sqrt{51.6}} = 14.3$$

Actual width-thickness ratio is  $24/2 = 12 < 14.3$ —OK

**Longitudinal Stiffener for Floorbeam.** Web design assumes longitudinal stiffening. The effective bending stress in the web is  $f_b/R = 39.51/0.939 = 42.1$  ksi. Depth of web in compression is  $D_c = 116/2 = 58$  in, distance from inner surface of compression flange to stiffener is  $d_s = 116/5 = 23.2$  in,  $d_s/D_c = 0.4$ , and buckling coefficient is  $k = 5.17(D/d_s)^2 = 129$ . Web thickness for web depth  $D = 116$  in must be at least  $D/340 = 116/340 = 0.34$  in and

$$t_w = \frac{D\sqrt{f_b}}{128\sqrt{k}} = \frac{116\sqrt{42.1}}{128\sqrt{129}} = 0.52 \text{ in}$$

Use a  $\frac{1}{16}$ -in web plate,  $t_w = 0.563$  in.

Location of the longitudinal stiffener is determined by  $D/5 = 116/5 = 23.25$  in. Thus, it should be placed 1 ft 11 in from the bottom of the top flange of the floorbeam. Minimum moment of inertia required for the stiffener is

$$I = Dt^3 \left[ 2.4 \left( \frac{d_o}{D} \right)^2 - 0.13 \right] = 116(0.563)^3 \left[ 2.4 \left( \frac{85}{116} \right)^2 - 0.13 \right] = 23.9 \text{ in}^4$$

in which the spacing of transverse stiffeners at midspan is taken as  $d_o = 85$  in. Bending stress in the longitudinal stiffener, which is 35 in from the neutral axis, is

$$f_b = 39.51 \times \frac{35}{60} = 23.04 \text{ ksi}$$

Use A588 steel for the stiffener, with allowable stress of 27 ksi. If a stiffener width  $b' = 6$  in is assumed, the minimum thickness required is

$$t = \frac{b'\sqrt{F_{yf}}}{96} = \frac{6\sqrt{100}}{96} = 0.63 \text{ in}$$

Use a  $6 \times \frac{5}{8}$ -in plate for the longitudinal stiffener. Its moment of inertia exceeds the 23.9 in<sup>4</sup> required.

**Web Check.** Allowable web shear is  $F_v = 17$  ksi for A588 steel. Assume a thickness of  $\frac{1}{16}$  in for the 109.3-in web at the support, as indicated by preliminary design. Then, maximum shear stress at the support is



$$f_v = \frac{764.7}{109.3 \times 0.438} = 16.0 \text{ ksi} < 17$$

The  $109.3 \times 7/16$ -in web is satisfactory.

**Transverse Stiffeners for Floorbeam.** In accordance with AASHTO standard specifications, the maximum spacing for transverse intermediate stiffeners in longitudinally stiffened girders should not exceed 1.5 times the maximum subpanel depth,  $D_s = 4/5 D$ .

$$1.5(4/5)(109.3) = 131.2 \text{ in}$$

Stiffener spacing is determined by the shear stress computed from Eq. (11.25a). For this computation, assume that the end spacing for the stiffeners is  $d_o = 22$  in. Hence,  $d_o/D = 22/109.3 = 0.201$ . From Eq. (11.24d), for use in Eq. (11.25a),  $k = 5[1 + (1/0.201)^2] = 128.8$  and  $\sqrt{k}/F_y = \sqrt{128.8}/50 = 1.60$ . Since  $D/t_w = 109.3/0.438 = 250$ , which is less than  $190 \times 1.6$ ,  $C$  in Eq. (11.25a) is 1, and the maximum allowable shear is  $F'_v = F_y/3 = 50/3 = 16.67 \text{ ksi} > 16 \text{ ksi}$ —OK.

The occurrence of simultaneous shear and bending in a panel when shear is larger than 60% of the allowable shear stress  $F_v$  requires that the allowable bending stress be limited to

$$F_s = F_y(0.754 - 0.34f_v/F_v) \quad (14.8)$$

A check of the stresses indicates that the member sizes are adequate for this condition.

Transverse-stiffener design requires computation of a factor  $J$  for determining required moment of inertia, where  $J = 0.5$  or more:

$$J = 2.5 \left( \frac{D_s}{d_o} \right)^2 - 2 = 2.5 \left( \frac{87.4}{22} \right)^2 - 2 = 37.5$$

Hence, the moment of inertia required, with a stiffener spacing  $d_o = 22$  in, is

$$I = d_o t_w^3 J = 22(0.438)^3 37.5 = 69 \text{ in}^4$$

The minimum gross cross-sectional area  $A$  for intermediate stiffeners is determined from

$$A = Y[0.15BDt_w(1 - C)(f_v/F_v) - 18t_w^2] \quad (14.9)$$

where  $B = 1.0$  for stiffeners in pairs

$$Y = F_{yw}/F_{ys} = 50/36 = 1.39$$

$C = 1.0$  from a previous calculation

When  $C = 1.0$ , Eq. (14.9) yields a negative number, indicating that the moment of inertia of the stiffener controls the stiffener size. The stiffener width should be at least  $2 + D/30 = 2 + 116/30 = 5.86$  in and at least one-fourth the flange width, or  $24/4 = 6$  in.

For the first transverse stiffeners, a pair of stiffeners  $7 \times 1/2$  in is specified. These have a moment of inertia

$$I = 0.5(14)^3/12 = 114.3 \text{ in}^4 > 110.0 \text{ in}^4$$

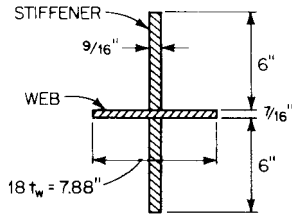
For the remaining intermediate transverse stiffeners,  $d_o = 42.5$  in is used. For computation of required  $I$ ,

$$J = 2.5(87.4/42.5)^2 - 2 = 8.57 > 0.50$$

For this value of  $J$ ,

$$I = 42.5(0.438)^3 8.57 = 30.6 \text{ in}^4$$

Use a 6-in-wide stiffener. Thickness must be at least  $\frac{1}{16}$  of this, or  $\frac{3}{8}$  in. Moment of inertia furnished by a pair of  $6 \times \frac{3}{8}$  in stiffeners is



$$I = \frac{0.375(12)^3}{12} = 54.0 \text{ in}^4 > 30.6 \text{ in}^4$$

Hence, a pair of  $6 \times \frac{3}{8}$ -in stiffeners is satisfactory. Use A36 steel.

A check of stiffeners between the first and second stringers indicates required  $I = 53.5 \text{ in}^4 < 54.0$ .

FIGURE 14.40 Bearing stiffeners for floorbeam.

**Bearing Stiffeners for Floorbeam.** Stiffeners must be provided under the stringers. Use a pair of A36 stiffener plates. They act, with a strip of web  $18t_w = 7.88$  in long, as a column transmitting the stringer reactions to the floorbeam (Fig. 14.40). Minimum stiffener thickness permitted, assuming a 6-in wide plate, is

$$t = \frac{b'}{12} \sqrt{\frac{F_y}{33}} = \frac{6}{12} \sqrt{\frac{36}{33}} = 0.523 \text{ in}$$

Use  $6 \times \frac{9}{16}$ -in plates. Moment of inertia furnished is

$$I = \frac{0.563(12.44)^3}{12} = 90.2 \text{ in}^4$$

Area of the equivalent column is

$$A = 7.88 \times 0.438 + 2 \times 6 \times 0.563 = 10.20 \text{ in}^2$$

The radius of gyration thus is

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{90.2}{10.20}} = 2.97 \text{ in}$$

For a length of about 115 in (maximum stiffener depth at stringers), the column then has a slenderness ratio  $KL/r = 115/2.97 = 38.7$ . The allowable compressive stress for this column is

$$F_a = 16.98 - 0.00053(KL/r)^2 = 16.98 - 0.00053(38.7)^2 = 16.19 \text{ ksi}$$

Actual compressive stress under a maximum stringer reaction of 127 kips is

$$f_a = \frac{127}{10.20} = 12.4 \text{ ksi} < 16.19 \text{ ksi}$$

The bearing stiffeners provide an effective bearing area, outside the flange-to-web weld, of

$$A_b = 2 \times 0.563(6 - 0.5) = 6.19 \text{ in}^2$$

The stringer reaction imposes a bearing stress of  $127/6.19 = 20.5$  ksi. Since this is less than the 29 ksi permitted, the  $6 \times \frac{9}{16}$  in stiffeners are satisfactory.

**Flange-to-Web Welds for Floorbeam.** These are made the minimum size permitted for a 2-in flange,  $\frac{9}{16}$ -in fillet welds. To check these welds, the properties of the floorbeam at the support given in Table 14.6 are needed. Shear at the top of the web is

$$v = \frac{VQ}{I} = \frac{764.7 \times 2,675}{345,436} = 5.92 \text{ kips per in}$$

Thus, each of the two fillet welds connecting the web to the flange is subjected to a unit shear

$$f_v = \frac{5.92}{2 \times 0.707 \times 0.3125} = 13.4 \text{ ksi}$$

This is less than the 15.7 ksi allowed by AASHTO specifications for welds made with E70XX welding electrodes. For fatigue the maximum shear range is  $209.4 + 44.6 = 254.0$  kips. The shear stress range at the top of the web is

$$v = 254.0 \times 2675/345,436 = 1.97 \text{ kips per in}$$

The shear stress on the throat of the fillet weld is

$$f_v = 1.97/(2 \times 0.707 \times 0.3125) = 4.46 \text{ ksi}$$

This detail falls into AASHTO fatigue stress category F and, for a redundant-load-path structure and 2,000,000 loading cycles, the allowable stress range is 9 ksi  $>$  4.46 ksi. Thus, the  $\frac{5}{16}$ -in welds are satisfactory.

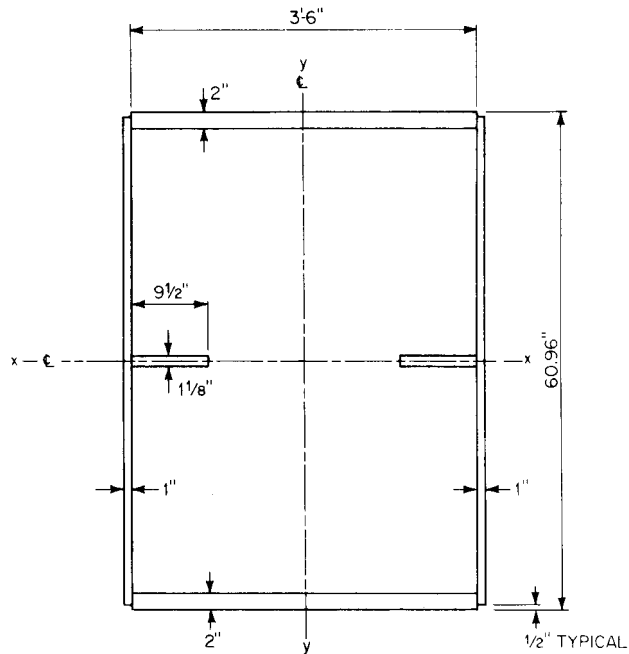
### 14.11.2 Design of Arch Rib

Arch, tie, and hangers were analyzed by computer with the system assumed acting as an indeterminate plane frame. The live load was taken as a moving load of 1.92 kips per ft, without a concentrated load or impact. For bridges of this type, which are outside the range of the AASHTO specifications, the choice of live loading is subject to the judgement of the designer.

The design procedure for the arch rib will be illustrated by the calculations for a rib section 54.78 ft long at panel point  $U_3$  (Fig. 14.18). The assumed cross section, of A514

**TABLE 14.6** Properties of Floorbeam at Support

Section	Area	$d$	$I$
Web: $109\frac{3}{8} \times \frac{7}{16}$	47.8	...	47,704
Flanges: $2-24 \times 2$	96.0	55.69	297,732
			345,436
At top of web, $Q = 24 \times 2 \times 55.69 = 2675 \text{ in}^3$			



**FIGURE 14.41** Arch-rib cross section.

steel, is shown in Fig. 14.41. The section properties given in Table 14.7 are needed. From the computer analysis, the loads on the arch section are:

	Thrust, kips	Moments, ft-kips
Dead load	-8430	367
Live load	<u>-686</u>	<u>1285</u>
Total	-9116	1652

**TABLE 14.7** Properties of Arch Section

Section	Area	Axis <i>x-x</i>			Axis <i>y-y</i>		
		<i>d<sub>y</sub></i>	<i>I<sub>o</sub></i>	<i>I<sub>t</sub></i>	<i>d<sub>x</sub></i>	<i>I<sub>o</sub></i>	<i>I<sub>t</sub></i>
2—59.96 × 1	119.92	...	35,928	35,928	21.5	...	55,433
2—42 × 2	168.00	29.48	...	146,004	...	24,696	24,696
2—9½ × 1½	21.38				16.25	160	5,806
	<u>309.30</u>			<u>181,932</u>			<u>85,935</u>

Radius of gyration  $r_y = \sqrt{85,935/309.30} = 16.67$  in

$$r_x = \sqrt{181,932/309.30} = 24.24 \text{ in}$$

Slenderness ratio  $L/r_y = 54.78 \times 12/16.67 = 39.4$

$$L/r_x = 54.78 \times 12/24.24 = 27.1$$

Section modulus  $S_x = 181,932/(60.96/2) = 5969 \text{ in}^3$

The arch rib is subjected to both axial compressive stresses and bending stresses. For buckling in the vertical plane, the allowable compressive stress is

$$F_a = \frac{F_y}{2.12} \left[ 1 - \frac{(KL/r)^2 F_y}{4\pi^2 E} \right] = \frac{100}{2.12} \left[ 1 - \frac{(39.4)^2 100}{(4\pi^2) 29,000} \right] = 40.77 \text{ ksi}$$

The allowable bending stress for A514 steel is  $F_b = 55$  ksi. The axial compressive stress in the rib is

$$f_a = P/A = 9,116.0/309.30 = 29.47 \text{ ksi}$$

The bending stress in the rib is

$$f_b = M/S_x = 1,652 \times 12/5,969 = 3.32 \text{ ksi}$$

The combined axial and bending stresses are required to satisfy the equation

$$f_a/F_a + f_b/F_b \leq 1.0 \quad (14.10)$$

Substitution of the preceding stresses in Eq. (14.10) yields

$$29.47/40.77 + 3.32/55.0 = 0.72 + 0.06 = 0.78\text{—OK}$$

[Equation (14.11), p. 14.64, should be used instead of the simpler Eq. (14.10) but the difference in result is trivial for this example.]

**Plate Buckling in Arch Rib.** Compression plates are checked to ensure that width-thickness ratios  $b/t$  meet AASHTO specifications. However, the arch rib does not fall into the limits of applicability of the AASHTO equations because

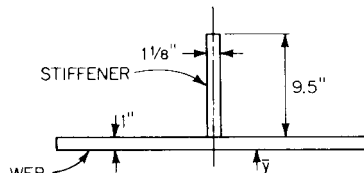
$$f_b/(f_a + f_b) = 3.32/(29.47 + 3.32) = 0.10 < 0.20$$

Hence, the check is performed as follows: Because stiff longitudinal stiffeners will be attached to the web (Fig. 14.42), assume that a node will occur at the web middepth. Thus, the  $D/t_w$  ratio is checked for a solid web based on the clear distance between the longitudinal stiffener and flange:  $D/t_w = 27.92/1 = 27.92$ . Since the stress gradient is small for this case, the  $D/t_w$  limit will be based on the total stress ( $f_a + f_b$ ) rather than the axial stress alone. Therefore, the limit is

$$D/t_w \leq 158/\sqrt{f_a + f_b} \leq 158/\sqrt{29.47 + 3.32} = 27.61 \approx 27.92$$

Since the limit is exceeded by only about 1%, the depth-thickness ratio is acceptable.

The width-thickness ratio for the flange plates between webs is limited to a maximum of



**FIGURE 14.42** Stiffener on arch-rib web.

$$\begin{aligned}
 b/t_f &= 134/\sqrt{f_a + f_b} \leq 47 \\
 &= 134/\sqrt{29.47 + 3.32} = 23.5 < 47
 \end{aligned}$$

The width-thickness ratio of the flange plates is

$$b/t_f = 42/2 = 21 < 23.5\text{—OK}$$

**Longitudinal Stiffener in Arch Rib.** The required moment of inertia of the longitudinal stiffener (Fig. 14.42) about an axis at its base, parallel to the web, is

$$\begin{aligned}
 I_s &= 0.75Dt_w^3 = 0.75 \times 56.96(1)^3 \\
 &= 42.7 \text{ in}^4
 \end{aligned}$$

The stiffener provides a moment of inertia of

$$I = bh^3/3 = 1.125(9.5)^3/3 = 321.5 \text{ in}^4 > 42.7\text{—OK}$$

The width-thickness ratio of the stiffener is governed by

$$\begin{aligned}
 b'/t_s &= 51.4/\sqrt{f_a + f_b/3} \leq 12 \\
 &= 51.4/\sqrt{29.47 + (3.32/3)} = 9.3 < 12
 \end{aligned}$$

The width-thickness ratio of the stiffener is

$$b'/t_s = 9.5/1.125 = 8.4 < 9.3\text{—OK}$$

It is not necessary to check the arch rib for fatigue, since it is not subject to tensile stresses.

### 14.11.3 Design of Tie

Design procedure for the tie will be illustrated for a section at panel point  $L_3$  (Fig. 14.18). The assumed cross section, of A588 steel, is shown in Fig. 14.43. The tie is subjected to combined axial tension and bending. In this case, the axial stress is so large that no compression occurs on the section due to bending. The allowable stress for A588 steel in axial tension or bending is 27 ksi.

The section properties given in Table 14.8 are needed.

From the computer analysis of the arch-tie system, the loads on the tie section are

	Axial tension, kips	Moment, ft-kips
Dead load	7,696	471
Live load	418	16,436
Total	8,114	16,907

Based on the preceding properties, the maximum combined stress is

$$f = \frac{P}{A} + \frac{Mc}{I} = \frac{8,114}{443} + \frac{16,907 \times 12}{24,710} = 18.32 + 8.21 = 26.53 \text{ ksi} < 27$$

Since the tie is a tension member, fatigue should be investigated. It was checked as a non-redundant-load-path structure and found to be acceptable. Thus, the tie section is satisfactory.

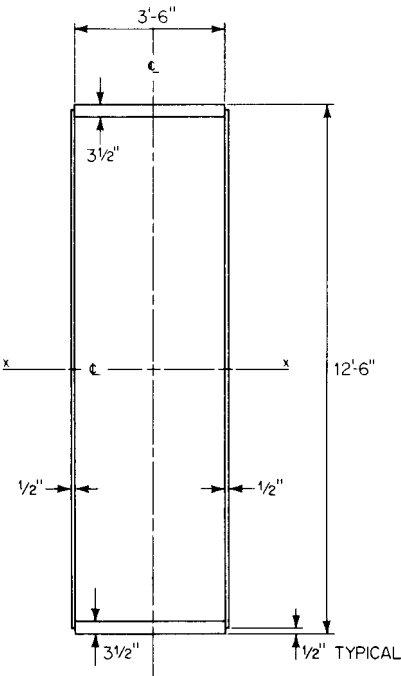


FIGURE 14.43 Tie cross section.

14.11.4 Design of Hangers

All hangers consist of four 27/8-in dia bridge ropes (breaking strength 758 kips per rope). With a safety factor of 4, the allowable load per rope is 189 kips.

From the computer analysis of the arch-tie system, the most highly stressed hanger is  $L_4U_4$  (Fig. 14.18). It carries a 630-kip dead load and 99.5-kip live load, for a total of 729.5 kips. This is carried by four ropes. Thus, the load per rope is

$$P = \frac{729.5}{4} = 182.4 \text{ kips} < 189\text{—OK}$$

The live-load stress range for the hangers is small and considered acceptable. If a lower safety factor is used, or a larger live-load stress range is present, a more detailed fatigue

TABLE 14.8 Properties of Tie Section

Section	Area $A$	$d$	$Ad^2$	$I_o$	$I_t$
PL 2—149 $\times$ 1/2	149.0			276,000	276,000
PL 2—42 $\times$ 3 1/2	294.0	73.25	1,577,000		1,577,000
	443.0				1,853,000
Section modulus $S_x = 1,853,000/75 = 24,710 \text{ in}^3$					

investigation should be made. Also, provisions must be made to eliminate any possible aerodynamic vibrations of the hangers and details must be adequate for corrosion protection.

#### 14.11.5 Bottom Lateral Bracing

The plan of the bracing used in the plane of the tie is shown in Fig. 14.18. Figure 14.44 shows the section used for the diagonal in the panel between  $L_0$  and  $L_1$ . Steel is A36.

Because of lateral wind, the axial load on the 73-ft-long diagonal is 295 kips. The member also is subjected to bending due to its own weight. The section properties given in Table 14.9 are needed.

Weight of the member is

$$w = 34.18 \times 0.0034 = 0.12 \text{ kip per ft}$$

This produces a maximum bending moment, at midspan, of

$$M = \frac{wL^2}{8} = \frac{0.12(73)^2}{8} = 79.9 \text{ ft-kips}$$

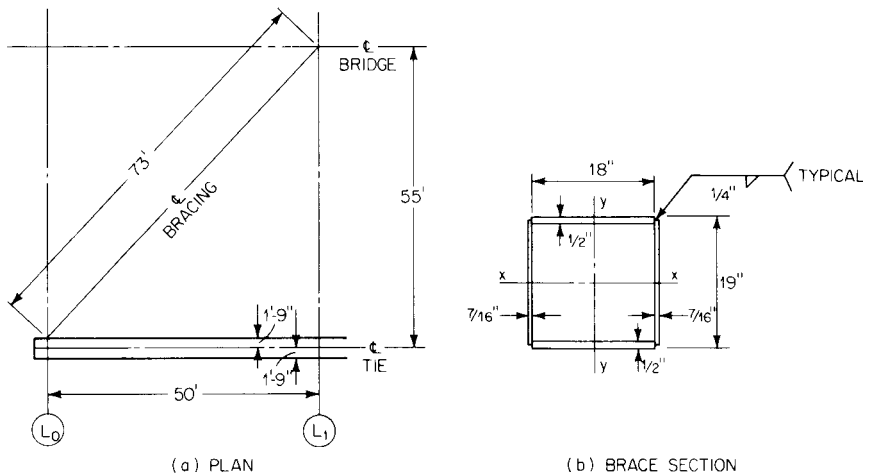
Maximum dead-load stress produced then is

$$f_b = \frac{79.9 \times 12}{211} = 4.54 \text{ ksi}$$

Wind will induce an axial compressive stress in the diagonal of

$$f_a = \frac{295}{34.18} = 8.63 \text{ ksi}$$

Members subjected to combined axial compression and bending must satisfy



**FIGURE 14.44** Diagonal brace in the plane of the ties.



**TABLE 14.9** Properties of Diagonal in Bottom Lateral Bracing

Section	Area $A$	Axis $x$ - $x$				Axis $y$ - $y$			
		$d_x$	$I_o$	$Ad_x^2$	$I_x$	$d_y$	$I_o$	$Ad_y^2$	$I_y$
PL 2—18½ × 7/16	16.18	...	462	...	462	9.22	...	1375	1375
PL 2—18 × ½	18.00	9.25	...	1540	1540	...	486	...	486
	34.18				2002				1861

Distance  $c$  to  $c$  connections =  $73 - 2 = 71$  ft

Radius of gyration  $r_x = \sqrt{2002/34.18} = 7.65$  in

$$r_y = \sqrt{1861/34.18} = 7.38 \text{ in}$$

Effective length factor  $K = 0.75$  (truss-type member connections)

Slenderness ratio  $KL/r_y = 0.75(71 \times 12)/7.38 = 86.6 < 140$

Slenderness ratio  $KL/r_x = 0.75(71 \times 12)/7.65 = 83.5 < 140$

Section modulus  $S_x = 2002/9.5 = 211 \text{ in}^3$

$$\frac{f_a}{F_a} + \frac{C_{mx}f_{bx}}{(1 - f_a/F'_{ex})F_{bx}} + \frac{C_{my}f_{by}}{(1 - f_a/F'_{ey})F_{by}} \leq 1.0 \quad (14.11)$$

$$\text{where } F'_e = \frac{\pi^2 E}{FS(K_b L_b / r_b)^2}$$

$FS$  = safety factor = 2.12

$C_m$  = coefficient defined for Eq. (6.67) (may be taken conservatively equal to unity)

The allowable axial stress for structural carbon steel ( $F_y = 36$  ksi) is

$$F_a = 16.98 - 0.00053(KL/r_y)^2 = 16.98 - 0.00053(86.6)^2 = 13.0 \text{ ksi}$$

The allowable bending stress is  $F_b = 20$  ksi. For dead-load bending about the strong axis,

$$F'_e = \frac{\pi^2(29,000)}{2.12(83.5)^2} = 19.36 \text{ ksi}$$

When wind stresses are combined with dead-load stresses (Group II loading), the allowable stresses may be increased 25%. Substitution of wind-load and dead-load stresses in Eq. (14.11) gives

$$\frac{8.63}{1.25 \times 13.0} + \frac{1.0 \times 4.54}{(1 - 8.63/19.36)20 \times 1.25} = 0.53 + 0.33 = 0.86 < 1.0 \text{—OK}$$

**Plate Buckling in Lateral Brace.** Compression plates are checked to ensure that width-thickness ratios  $b/t$  meet AASHTO specifications. The compressive stress is taken as  $f_a = (8.63 + 4.54)/1.25 = 10.54$  ksi.

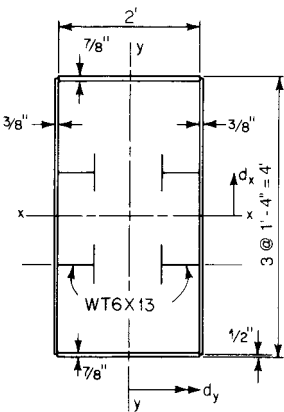


FIGURE 14.45 Section for brace between arch ribs.

Width-Thickness Ratios

	1/2-in top plate	7/16-vertical plate
Actual $b/t$	$18/0.5 = 36$	$18.5/0.438 = 42.3$
Allowable $b/t$	$126.5/\sqrt{f_a} = 39 < 45 \text{ max}$	$158/\sqrt{f_a} = 48.7 < 50 \text{ max}$

The brace section is satisfactory.

14.11.6 Rib Bracing

The plan of the structural carbon steel bracing used for the arch rib is shown in Fig. 14.18. Figure 14.45 shows the section used for a brace in the first panel of bracing.

Rib bracing is designed to carry its own weight, wind on ribs and rib bracing, and an assumed buckling shear from compression of the ribs. Loads on the first-panel brace are given in Table 14.10, and section properties are computed in Table 14.11. The maximum bending stress produced by total load is

$$\begin{aligned} f_b &= \frac{1120 \times 12}{1284} + \frac{134.5 \times 12}{652} \\ &= 10.5 + 2.5 = 13.0 \text{ ksi} \end{aligned}$$

TABLE 14.10 Loads on Brace Between Arch Ribs

	$P$ , kips	$M_x$ , ft-kips	$M_y$ , ft-kips
Dead load	...	1120	67.5
Wind	58.7	...	67.0
Buckling	266.0		
Total	324.7	1120	134.5

**TABLE 14.11** Properties of Rib Brace

Section	A	Axis x-x				Axis y-y			
		$d_x$	$I_o$	$Ad_x^2$	$I_x$	$d_y$	$I_o$	$Ad_y^2$	$I_y$
PL 2—47 × 3/8	35.2	...	6490	...	6,490	12.19	...	5230	5230
PL 2—24 × 7/8	42.0	23.56	...	23,300	23,300	...	2020	...	2020
4—WT 6 × 13	15.3	8.00	35	979	1014	7.14	47	780	827
	92.5				30,804				8077
Radius of gyration $r_x = \sqrt{30,804/92.5} = 18.2$ in									
$r_y = \sqrt{8,077/92.5} = 9.34$ in									
Unsupported length = 58.7 ft									
Effective length factor $K = 0.75$ (truss-type member connections)									
Slenderness ratio $KL/r_x = 0.75 \times 58.7 \times 12/18.2 = 29.0$									
Slenderness ratio $KL/r_y = 0.75 \times 58.7 \times 12/9.34 = 56.6$									
Section modulus $S_x = 30,804/24 = 1284$ in <sup>3</sup>									
$S_y = 8,077/12.38 = 652$ in <sup>3</sup>									

The total axial stress is

$$f_a = 324.7/92.5 = 3.5 \text{ ksi}$$

For combined stresses with wind, allowable stresses may be increased 25%. Axial and bending loads are evaluated for combined stresses with Eq. (14.11) with  $C_m = 1$ .

$$F'_{ex} = \frac{\pi^2(29,000)}{2.12(29.0)^2} = 160.5 \text{ ksi}$$

$$F'_{ey} = \frac{\pi^2(29,000)}{2.12(56.6)^2} = 42.1 \text{ ksi}$$

$$F_a = 16.98 - 0.00053(56.6)^2 = 15.3 \text{ ksi}$$

$$F_b = 20.0 \text{ ksi}$$

$$\begin{aligned} & \frac{3.5}{1.25 \times 15.3} + \frac{1.0 \times 10.5}{(1 - 3.5/160.5)20.0 \times 1.25} + \frac{1.0 \times 2.5}{(1 - 3.5/42.1)20.0 \times 1.25} \\ & = 0.18 + 0.43 + 0.11 = 0.72 < 1.0 \text{—OK} \end{aligned}$$

**Plate Buckling in Brace.** Compression plates are checked to ensure that width-thickness ratios  $b/t$  meet AASHTO specifications. Compressive stress is taken as  $f_a = (3.5 + 13.0)/1.25 = 13.2$  ksi.

*Width-Thickness Ratios*

	<sup>3</sup> / <sub>8</sub> -in Web	<sup>7</sup> / <sub>8</sub> -in Flange
Actual $b/t$	$16/0.375 = 42.7$	$24/0.875 = 27.4$
Allowable $b/t$	$158/\sqrt{f_a} = 43.5 < 50 \text{ max}$	$126.5/\sqrt{f_a} = 34.8 < 45 \text{ max}$

The brace section is satisfactory.