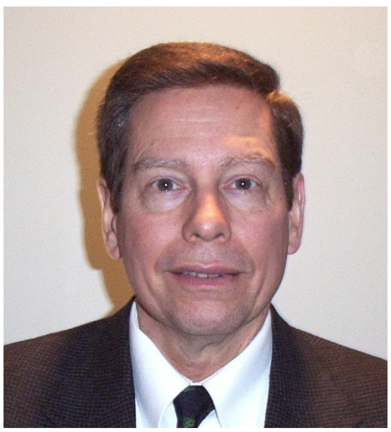
DESIGN, FABRICATION AND ERECTION ISSUES FOR LONG SPAN STEEL TIED ARCHES: A CASE STUDY



ERNST PETZOLD, P.E.

BIOGRAPHY

Mr. Petzold was Project Manager for the preliminary and final design of the Route 364 Missouri River Bridge described in this paper. He has served as Designer and Project Manager on various other projects throughout the U.S.; design studies for Mississippi River bridges at St. Paul, MN, Dubuque and Burlington, IA, and Moline, IL, seismic evaluation of the Poplar Street bridge and double-decked approaches in St. Louis, MO, rehabilitation / replacement study for a Missouri River bridge at Chamberlain, SD and the engineering study for a steel box girder across the Carquinez Strait in California, to name a few.

Mr. Petzold has been with Jacobs for 31 years, is a certified Senior Project Manager and a Jacobs Fellow, and is a registered Professional Engineer in numerous states. He holds both BS and MS degrees in civil engineering from Washington University in St. Louis and a MBA from St. Louis University. He is a member of the American Society of Civil Engineers, American Railway Engineering Association, Heavy Movable Structures (Movable Bridges Affiliate), and the Post-Tensioning Institute.

SUMMARY

Steel tied arch bridges are again becoming more common in the United States as better concepts of how to provide redundancy in the structural system have been developed and accepted. This type of bridge fell out of favor in the late 1970's when several welded tie girders developed potentially catastrophic cracks.

The paper reviews various design related issues with steel tied arches, including the important one of redundancy. Overall structural response is discussed along with specific detailing for the deck system, tie and arch rib, hangers and laterals. Fabrication issues are briefly touched on. Several methods for erection are discussed including the use of falsework and tiebacks for erection on-alignment and the use of off-alignment techniques.

The recently constructed Route 364 bridge across the Missouri River with its 188 meter long steel tied arch navigation span is used as a case study example of the application of some of the issues presented in the paper.

DESIGN, FABRICATION AND ERECTION ISSUES FOR STEEL TIED ARCHES: A CASE STUDY

Ernst Petzold

INTRODUCTION

The Route 364 Missouri River Bridge, located in the St. Louis, Missouri metropolitan area, includes a major steel tied arch navigation span. This paper will review some of the issues associated with designing, fabricating and erecting steel tied arches in general. A case study format is used, with the discussion of the issues being followed by a summary of how the particular item was dealt with on the Route 364 bridge.

The Missouri River Bridge is part of a larger project (the Page Avenue Extension) constructed by the Missouri Department of Transportation (MoDOT) to provide an additional route for commuters between St. Louis County and St. Charles County. The project, to the northwest of St. Louis City, will help to relieve heavy congestion on the current I-70 corridor. In addition to the Missouri River Bridge, the route includes another major structure, the Creve Coeur Lake Memorial Park Bridge. Located in an urban park the latter bridge crosses the southern tip of one of the few remaining ox-bow lakes of the Missouri River (Figure 1).

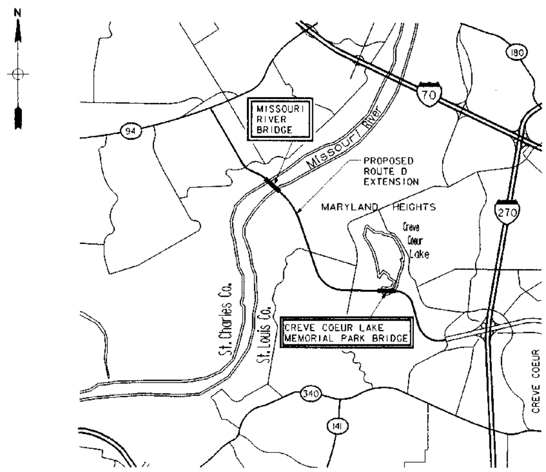


Figure 1 Project Location

Both bridges carry ten lanes of traffic on dual side-by-side structures (five lanes of traffic with full shoulders on each structure). The Missouri River Bridge, which also includes provisions for a bicycle/pedestrian path, is 987 meters long and extends from west of the Howard Bend Levee in St. Louis County to the bluff on the St. Charles County side of the River. In addition to the 188 meter long tied arch navigation span, the bridge includes multiple steel plate girder and concrete precast girder approach spans (Figure 2).

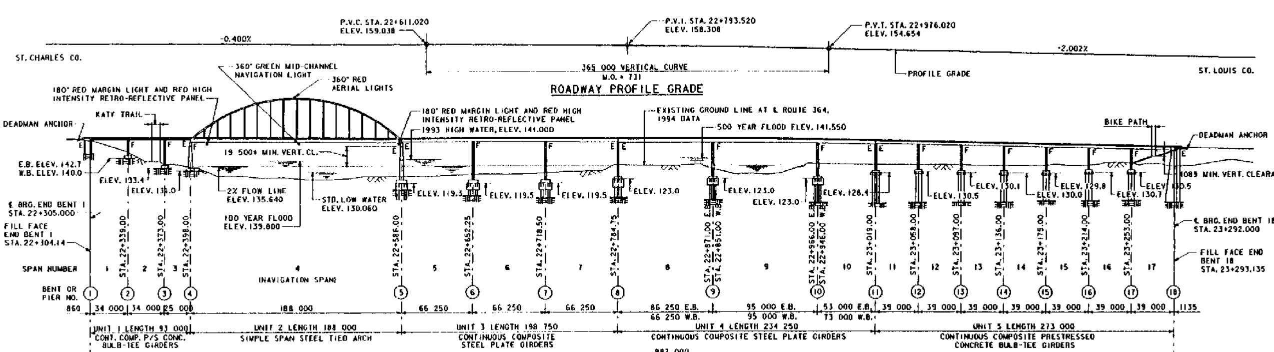


Figure 2 Bridge Elevation

Preliminary design began on the project in early 1994 with final design commencing in late 1995. The project was constructed in three construction contracts with a total bid price of approximately $116 million ($195 per square foot of roadway area). Contractors were Fred Weber, Inc. and Midwest Foundations, Inc. for substructures and Edward Kraemer & Sons, Inc. for the superstructure. The tied arches were fabricated by

PDM in Eau Claire, Wisconsin. The plate girder approach spans were fabricated by Stupp Bridge in Bowling Green, Kentucky. Construction was completed in late 2003 and the project was formally opened for traffic on December 14, 2003.

DESIGN

Redundancy

From the standpoint of bridge structures, redundancy refers to the ability of a structure to redistribute loads carried by a particular member (or system of members) in the event of the failure of that particular member or system such that the overall integrity of the bridge is maintained, that is, the structure does not collapse. As noted in FHWA Technical Advisory T5140.4 (1), "…the tied arch structure…is one of the most nonredundant structures, relying entirely on the capability of two tie girders to accommodate the total thrust imposed by the arch ribs." This particular advisory, issued in 1978, resulted from several tied arch bridges experiencing lamellar tearing in the hanger connections to the arch rib and also cracking at weld details in the tie girders. The purpose of the advisory was to alert owners to the problems that had occurred in tied arch bridges and to "…emphasize the need for a thorough evaluation of alternate designs which provide more redundancy." It should be noted that the bridges referenced in the advisory remained in service for an unknown, but likely significant, time period before the damage was noted and the bridges closed for repair. There were certain characteristics of these bridges that contributed to the initial cracking and its propagation. Fully welded tie girders, as used on these bridges, can potentially allow a crack, once formed, to propagate throughout the entire cross section. The tie girders for at least one of the bridges were fabricated with electroslag welding, which proved at its then-level of development, to be more susceptible to cracking than other welding technologies. In certain zones rather thick plates were used in constructing the ties. Since thick plates may not be as well consolidated in the rolling process as thin plates, there may exist inclusions in such plates that can be the source of crack initiation. In any event, these problems and the resulting advisory highlighted the need to provide redundancy in structural bridge systems.

For about a 20-year period after these problems first appeared (late 1970's to late 1990's) few steel tied arch bridges were proposed. The reasons for this are partly due to concerns over providing redundancy in the system and partly due to the emergence of the cable stayed bridge form. In the last 5 to 10 years, however, there has been a reemergence of the tied arch form for major bridges as designers have learned to design economical systems that deal with the previously identified concerns.

Overall System – Bowstring vs. Moment-Tie

Tied arch bridges can consist of a single span or be configured as continuous span systems. The single span bridges are almost always through arches with the tie girder at the deck level. The continuous systems usually consist of three spans with a center full arch and the flanking spans being half arches. These bridges can be arranged as half-through arches or as deck arches. This paper will deal mostly with the single span variety with some occasional references to the continuous tied systems (Figure 3).

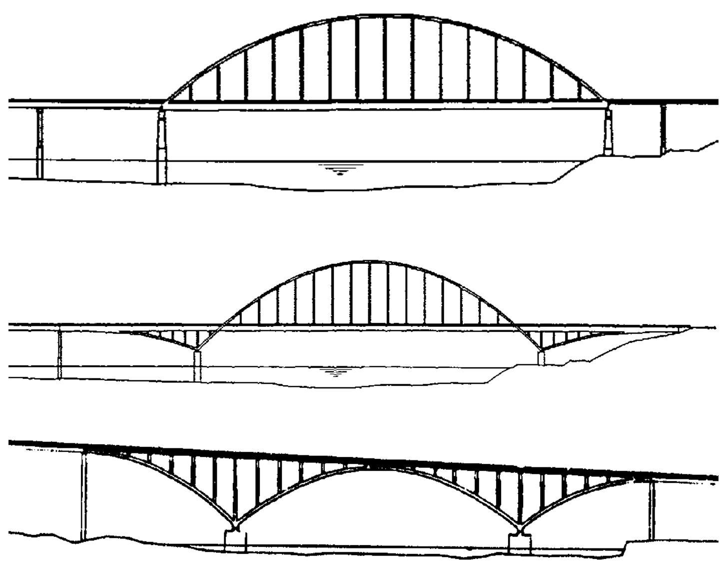


Figure 3 Basic Arch Bridge Configurations

From the standpoint of external statics the single span tied arch behaves in a determinant manner and reacts on the supporting substructure as if it were a simply supported beam. Internally, however, the system is indeterminate with the behavior being dependent on the ratio of the tie stiffness to the rib stiffness. In the classic bowstring arch the tie is predominantly a tension member with minimal bending stiffness. In this system the vertical loads are carried almost exclusively by the arch rib. The resulting proportions of the rib and lateral bracing are similar to what they would be if the system were in fact a "true" arch using a compression thrust block instead of a tension tie. Many older steel tied arches are of the bowstring type, perhaps due to the more direct correlation of the analysis techniques for this system with those of a true arch. As the stiffness of the tie is increased relative to that of the arch rib it begins to function as a beam and to participate in carrying vertical loads to the supports (this type of tied arch will be referred to as a "moment-tie" type in this paper). Accordingly, the demands on the arch rib are reduced allowing its size to be reduced significantly as compared to that of a true arch of comparable span length. Taken to the extreme, of course, these "moment-tie" tied arches would simply be beams with vestigial arches.

While the two systems differ somewhat structurally, there are no great advantages of one system over the other. Economics may favor the moment-tie types particularly where wide bridges are involved due to the smaller rib and bracing members. From an appearance standpoint the resulting rather large arch rib and accompanying large lateral bracing members of the bowstring arch may be at a disadvantage compared to the thinner ribs and lighter bracing of the moment tie systems. Spreng has investigated the structural efficiency of tied arches and has developed recommendations for tie and arch proportions using a definition of efficiency as the ratio of "structural output" over "structural input" (2). He defines "output" as the loading supported when the stress reaches the allowable level and "input" as the self-weight of the arch.

One significant overall design parameter for arch design is the rise-to-span ratio. Spreng has investigated this parameter and concluded that for loading over half of the bridge a relatively flat arch with a rise-to-span ratio of 1:7 works well. Conversely, for load over the entire span, a steeper arch with a ratio of 1:3 is the more optimal. For tied arch highway bridge structures where the dead load is more-or-less uniform over the length of the bridge, but the loaded length and position of the live load varies, the optimal ratio is usually considered to be in the range of 1:4 to 1:6. Designing within this range provides a reasonably efficient system in terms of the magnitude of the thrusts and moments to be carried in both the rib and the tie and the resulting amount of material required. The resulting geometry is also usually visually pleasing, providing an arch profile that is neither too high nor too flat.

The Route 364 tied arch was configured as a moment-tie type mainly to minimize the arch rib dimensions and produce a lighter overhead system. The relative proportions between the rib and the tie were such that the capacity of the tie is matched by approximately 50 percent tension demand and 50 percent bending demand. The resulting system has a tie that is roughly twice the depth of the rib. In the absence of other constraints, a rise-to-span ratio of 1:5 was used.

Deck Systems

Over time the deck or floor systems of tied arches (and trusses also) have evolved into transverse floor beam systems supporting longitudinal stringers, which in turn support a transversely spanning mild reinforced concrete deck slab. And while other deck systems are, of course, possible, the majority of truss and arch bridges use this basic arrangement. The floor beams are usually plate girders with a depth-to-span ratio of about 1:8. Panel lengths are in the range of 40 to 50 feet, which provides for efficient truss framing and reasonable arch hanger sizes. This panel length also works well with the use of rolled beams for stringers. The stringers may or may not be made composite with the deck slab they support, although a noncomposite design will usually be the most economical. This economy results since the stringer is sized for the noncomposite negative moment over the floorbeams and due to its short span length it is not typically economical to vary the stringer section in the positive moment area, so the same section is used throughout. Thus the stringer section alone is more than adequate to carry the positive moments and there is no reason to effect a composite connection.

Where structural depth is a concern the stringers may be framed into the floorbeams. Various stringer-to-floorbeam connection types have been used in this situation ranging from a full moment connection with upper splice plates to something more akin to a semi rigid connection. Alternatively, where structural depth is not a particular issue the stringers can be made continuous over the floorbeams, being supported on small bearings on the floorbeam top flange (Figure 4). The latter arrangement is usually the more economical of the two options.

Figure 4

Historically, designers have gone to great lengths to insure that the deck system does not participate with the main longitudinal support system, whether an arch or another type of system. This has been accomplished by placing transverse relief joints in the deck slab at about every fourth or fifth panel point and by using an appropriate mix of fixed and expansion bearings to support the stringers. These devices allow the deck to expand and contract independently of the arch both during construction (when the slab is placed) and in service when loaded by live load. The major drawback of such systems is the maintenance required by these relief joints themselves and the effect of the inevitable deck drainage that passes through them onto the steel below. On the Route 364 arches these deck relief joints were eliminated. And while still structurally separated from the arch itself the deck acts as one large continuous member instead of as a series of segmented slabs. In the system used the three center stringers have a fixed bearing at every floorbeam. The floorbeam top flanges are rather narrow and this fixity insures lateral buckling stability of these flanges. Given the length of the floorbeams (85 feet+) the resulting stresses in the top flange due to any resulting lateral bending are quite small. The center stringer is connected to the lower lateral system via a longitudinal truss at the center of the span that gathers all longitudinal forces to this point. All other stringers are supported on expansion bearings. It is important that stringers nearest the floorbeam-tie connection do not laterally restrain the floorbeam top flange. Such restraint can lead to fatigue cracking in the floorbeam top flange to web weld caused by differential longitudinal movement between the tie and the stringer system.

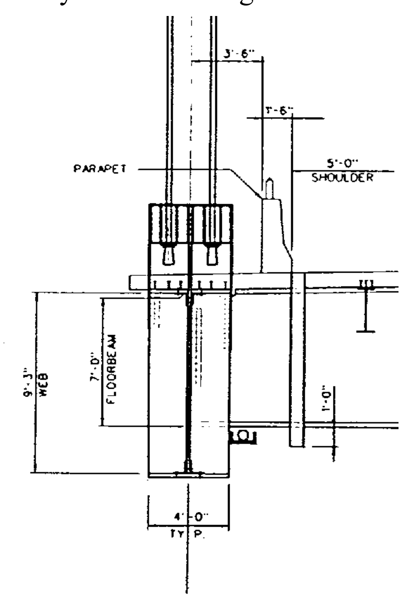


Figure 5 Concrete Deck Made Composite with Tie Girder

As an alternative to the traditional approach, some designers have structurally connected the floor system to the main longitudinal system (the tie girder in the case of a tied arch) to force the deck to participate with it. By so doing the overall stiffness of the system is increased and the size (depth) of the tie girder may be somewhat reduced as axial tension and, to a perhaps lesser extent, bending moments, are shared by the deck. These arrangements have been used in combination with orthotropic steel decks such as in the Port Mann, the Gorinchem and the Fremont tied arch bridges (3, 4). In these cases the steel deck participates in carrying both dead and live load effects arising in the arch system. This structural connection between the deck and the tie girder has not typically been used with concrete decks although the new US 20 Bridge at Dubuque, Iowa has been designed in this way (5). The basic concept is shown in Figure 5. For concrete decks, in order to limit the tension in the deck, it will be advantageous to make the deck connection to the tie using a closure pour after the majority of the deck has been placed. In this way the deck participates only in resisting the residually applied dead loads and the live load.

Tie Girder

In designing the tie girder there are three major areas of consideration: 1) the basic configuration of the tie, 2) the method of joining the individual elements of the tie and 3) how redundancy is to be built into the tie. These considerations are interrelated, particularly as regards items 2 and 3.

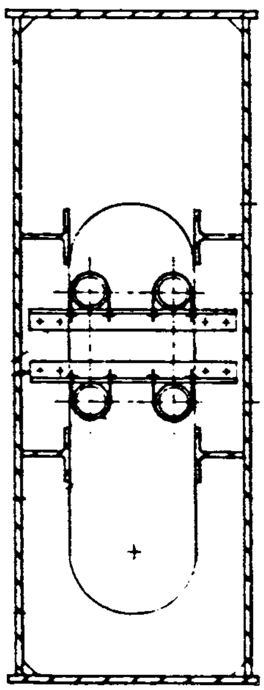


Figure 6 Post-tensioned Tie

In the broadest sense, two cross sectional arrangements are possible, a closed section, usually in the form of a rectangular box, and an open section, usually in the form of an "I" girder. Each has its advantages. Since, as discussed below, the arch rib is usually a box section, the use of a box section for the tie girder can simplify the tie-rib connection at the arch knuckle. Regarding fabrication, the three-plate design of the "I" girder may make it easier and more economical to fabricate than a box section. Structurally, the idealized end condition of the floorbeams as simply supported, may be better realized when a torsionally flexible tie girder is used instead of a torsionally stiff one, such as a box.

How the individual elements of the tie are joined and how redundancy is built into the structure are highly interrelated. Simply supported spans, such as the typical tied arch span, cannot redistribute loads to other spans in the event of the failure of a member. Alternate load paths within the single span of the tied arch are also not usually available, particularly for wide bridges, since the floor system and the upper lateral systems typically cannot redistribute a significant amount of load from a failed member on one side to the other side of the bridge. For these reasons, most redundancy strategies rely predominantly on the use of internal redundancy. That is, the tie girder is configured in such way that should a portion of it fail, the loads can be redistributed internally around the failed area and the possibility of the entire member failing is minimized. Internal redundancy has been provided in tie girders through various means. These include the use of post-tensioning strands (Figure 6) and joining the individual components by bolting instead of by welding.

An interesting tie concept has recently been used for an 85 meter long tied arch bridge in Des Moines, Iowa. The tie is constructed of high performance post-tensioned concrete. The numerous post-tensioning strands provide the desired amount of redundancy. The bridge uses steel arch ribs and steel composite floorbeams (6).

For the Route 364 tied arch span, it was decided to use a bolted tie or a tie that combined bolting and welding in such a way that internal redundancy was achieved. Since the tie girder fabrication is a definite high-dollar item, multiple tie girder configurations were developed and various fabricators and erectors were consulted to gather information regarding overall constructability (Figure 7). As a result of this study a fully bolted four-sided box was chosen for the tie girder (Figure 8).

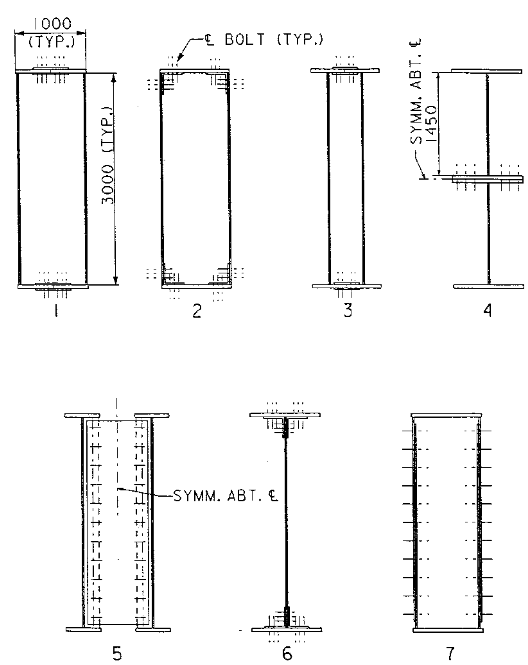


Figure 7 Alternative Tie Girders

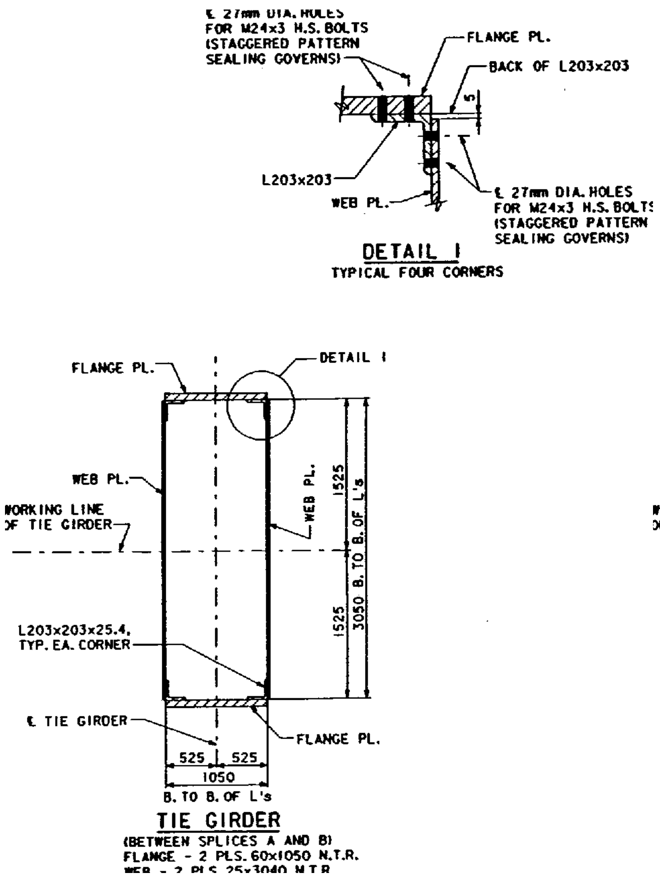


Figure 8 Final Tie Girder

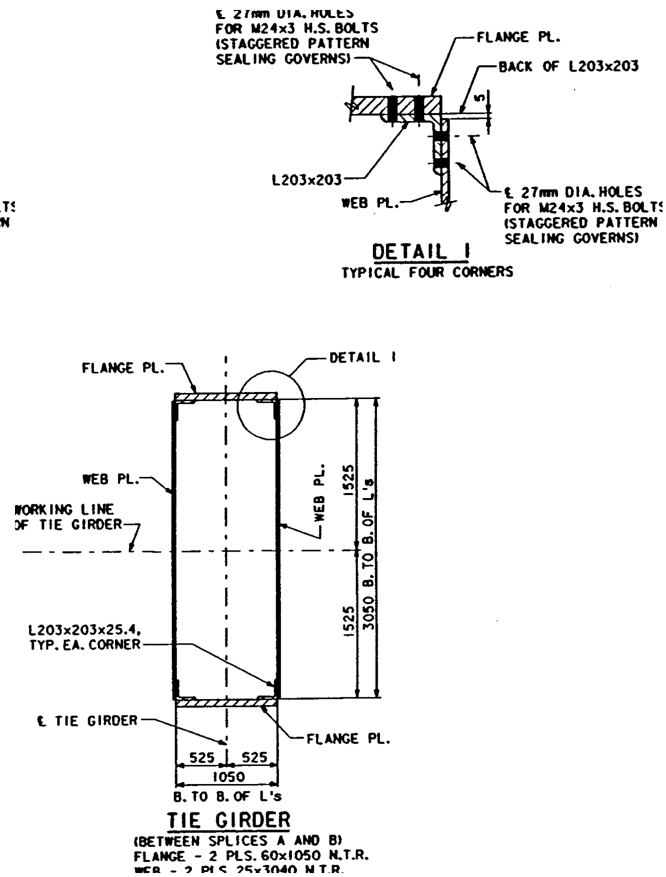


Figure 9 Arch Rib

Arch Rib

Since the rib carries a significant amount of compressive force, insuring its overall stability against buckling is a primary design concern. For this reason the ribs of most large tied arches are of closed box section construction. Redundancy is not a great concern since cracks are unlikely to occur and they would not readily propagate in any event, so fully welded designs are typical. From the standpoint of fabrication cost, it may be best to proportion the box dimensions and the individual plate thicknesses such that longitudinal stiffeners are not required. This can typically be accomplished with moment-tie tied systems where the rib is normally slender and the local buckling of the plates, assuming support at the box corners only, is not too limiting. For the larger and stiffer rib of bowstring arches, local buckling of the side plates may dominate the design unless a longitudinal stiffener (or stiffeners) is used. The rib for the Route 364 tied arch used a fully welded design without longitudinal stiffeners (Figure 9).

Hangers

Hangers of early tied arches were often of structural steel construction using rolled or built-up sections. Current practice is to use high strength bridge strand or wire rope. The hangers terminate at cast or forged steel sockets of various designs usually based on traditional practice and details. Hanger lengths must be adjustable during construction, and for that reason, one of the sockets, usually the lower one, is designed to allow variation in the hanger length. Each individual hanger consists of a group of strands or ropes; typically two or four are used.

Historically, tied arch hangers have been vertical, although diagonal or network systems are also possible. These network hanger systems are sometimes referred to as Nielsen systems after O. F. Nielsen who pioneered their use. The diagonal hangers stiffen the overall tied arch system, particularly in the region of the rib-tie connection and improve the overall response of the arch to concentrated and unsymmetrical loading. The result is that for any loading arrangement the forces are distributed by the hangers to the arch such that the line of thrust deviates little from the centerline of the arch and small moments result in both the rib and the tie. As compared to vertical hanger systems, the network system puts a higher demand on the hangers, making them larger and a more dominant feature of the bridge. The hangers near the end of the bridge may attempt to go into compression under certain loading cases, which situation needs to be handled by appropriate design considerations. The improved response of network system arches results in smaller demands on the arch rib and tie with a subsequent reduction in material for these components as compared to a similar tied arch with vertical hangers. For these reasons a network arch may be more economical. Due to the higher forces in the hangers of a network arch, details similar to those for the cables of a cable-stayed bridge may be utilized for these components.

For the Route 364 arch vertical hangers were used. This decision was based more on aesthetics than on engineering concerns. During the design development phase a design committee had responsibility to provide overall project oversight. Based on visualizations developed of both systems, they overwhelming favored the vertical hangers. Since there are twin roadways, there are four planes of hangers in the bridge. In this situation the vertical hangers tend to provide a more pleasing look from all vantage points as compared to diagonal systems.

Upper Lateral System

Various upper lateral configurations are possible and have been used, including "X" and "K" bracing and Virendeel moment frames. Structurally, the braced systems are the most efficient. However, since the upper lateral system is a major visual aspect of the bridge, the specific choice of system to be used is sometimes based on other than design efficiency considerations. The individual members of the upper lateral system are usually relatively substantial built up members although lattice-type members have been used. Again, aesthetics may play a part in this decision.

In some cases the lateral system has been connected to both flanges of the arch rib. This can lead to large bracing members. For the Route 364 tied arch the lateral members were proportioned to be shallower than the arch rib. An internal diaphragm at the lateral connection to the rib provided a positive connection between the flanges of the rib and the lateral system. The end result is a slender bracing system (Figure 10).

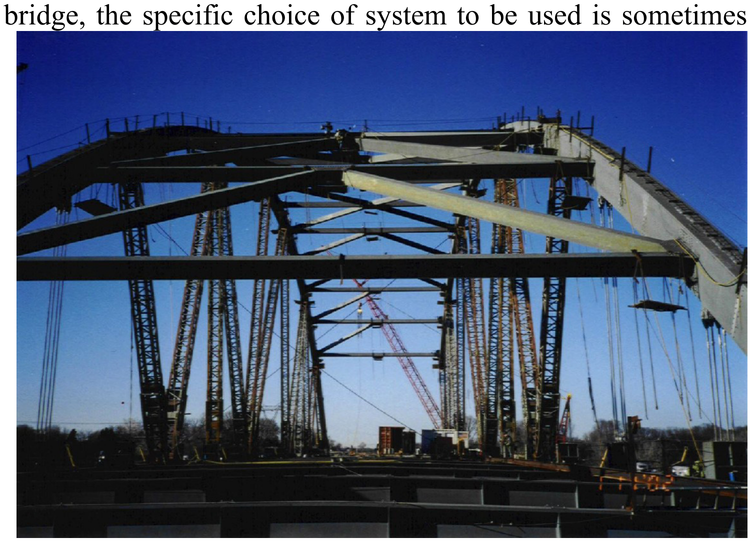


Figure 10 Upper Laterals for Route 364

Extreme Event Loads

Two of the extreme cases for the Route 364 project are described here; they dealt with redundancy and seismic concerns. To verify redundancy of the tie girder the structure was analyzed considering that one of the flange plates of the tie had fractured and was not effective in carrying load. The loading used for this situation was the full dead load and one lane of live load per arch. The stresses were shown to remain within the elastic limit of the tie material.

St. Louis is in a moderate seismic zone (SPC B). Given the evolving nature of seismic codes and the high costs associated with seismically retrofitting a major bridge, however, it was decided to incorporate several features to enhance the seismic response of the bridge. One of these involved the tied arch span. The tied arch is a simple span with a fixed bearing at one end and an expansion bearing at the other. To minimize the chance of the expansion end of the span coming off the supporting pier and also to force that pier to participate in resisting longitudinal seismic forces, shock transmission units were used at the expansion end of the arch. Two 500 kip units were used at each tie girder. These devices develop minimal force under thermal and other long-term inputs but effectively lock-up under shock loads.

Camber

The arch is detailed to provide the plan geometry after all dead load is applied. To accomplish this, the arch members are cambered for length. This cambering induces a favorable set of moments in the arch frame that are opposite in sign to the dead load moments, thus minimizing the total moments in the frame. To insure that these moments are present in the assembled arch, it is important that the designed geometry is achieved in the erected structure. This is accomplished by minimizing the amount of reaming that is permitted during assembly. For one closure joint of the Route 364 arches, the Contractor was not initially able to obtain the desired geometry. Subsequent analysis of the arch with a discontinuity in the geometry corresponding to the angular difference in the two sides of the splice led to the conclusion that the joint would have to be reassembled. To minimize such rework it is also possible during design to include a misalignment moment in the frame to account for minor erection misalignments.

It is noted that lateral members and stringers are not typically cambered, but are fabricated to the final dead load geometry of the bridge. By this means these members are prevented from participating in carrying dead load forces. As a result, the installation of these members can be somewhat complicated by the fact that they do not “fit” the bridge geometry at the time of erection. For this reason “K” bracing is often easier to erect than “X” bracing since the lateral flexibility of the midpoint connection of the “K” (at the center point of a strut or floorbeam) can facilitate the installation.

FABRICATION

From a designer’s standpoint, particularly in light of the discussion above dealing with camber, one of the more important issues related to fabrication is the manner in which the splices are fabricated. The contract documents for Route 364 required that the connections of the tie and rib be reamed or drilled full size while assembled in the shop in their correct angular relationship. Once the initial assembly was completed, subsequent members were to be assembled to at least one adjacent member that had been reamed or drilled full size in a previous assembly. As an alternative, the contract documents allowed the use of numerically controlled (N/C) drilling equipment. The fabricator was allowed to drill full size into unassembled pieces subject to specific requirements. These requirements included a full-length check assembly for the arch and tie of one arch along with tolerances for hole alignment. The arch check assembly did not require the fabricator to close the arch frame but only to verify that the theoretical angular relationships and overall dimensional geometry were achieved.

Figure 11 Arch Check Assembly

The fabricator elected to exercise the N/C drilling option. The check assembly was successfully completed (Figure 11) and the arches went together in the field with few problems.

ERECTION

Tied arches present somewhat of an erection challenge since unlike a truss the basic arrangement is more difficult to make self-supporting when partially erected. Also, as with any single span bridge of significant length, the use of temporary measures to support the span while it is being built are almost always required. In general, the erection of the single span tied arch can be classified into two cases: 1) those bridges that are built in their final location, and 2) those that are built off-alignment and then moved into their final position.

Figure 12 Arch Erection with Falsework

For those bridges built in their final locations two basic systems have been used. One method uses temporary falsework to support the partially completed arch (Figures 12 and 13). In the case of a navigable span, this falsework will need to be positioned such that the reduced navigation opening is acceptable to the users of the waterway and any regulatory agencies, such as the US Coast Guard.

A method of erecting the arch in its final position that eliminates the falsework in the navigation span is the use of temporary erection towers and a system of tiebacks to support the arch (Figure 14). Depending on the particular erection sequence it may be necessary to provide temporary stiffening of the arch using a set of rigid diagonals until the arch is closed. The advantage of erecting the arch in it final location is that each piece is handled only once. A possible disadvantage, if falsework is used, is the risk that it might be struck and damaged by either marine vessels or floating debris while the structure is relying on that falsework for overall stability.

Figure 13 Arch Erection with Falsework

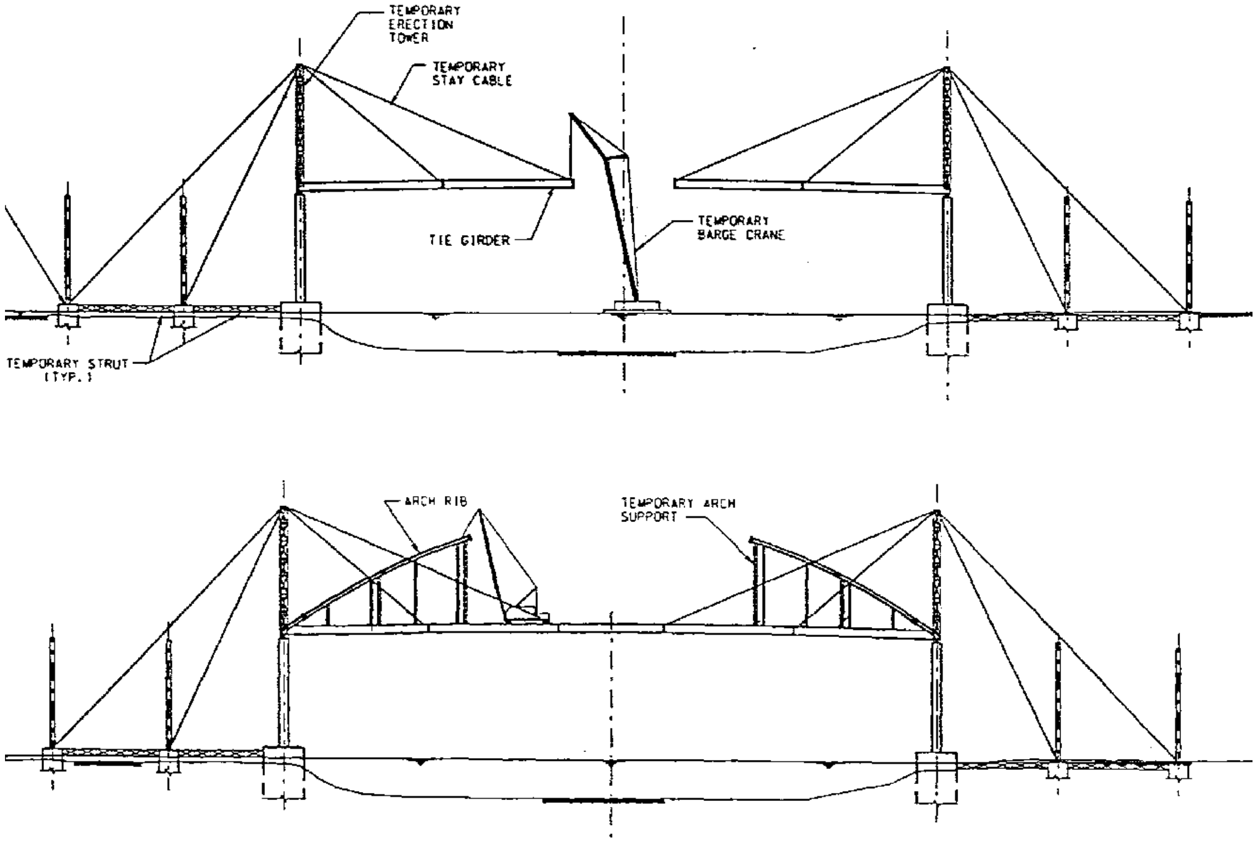


Figure 14 Arch Erection with Tiebacks

The second general method of erection involves building the arch span at a location other than the final position and then moving it into position once it is fully assembled. This method may be appropriate where sufficient space is available in the waterway to allow for the preassembly and where the water depth is adequate for the float-in operation. The obvious disadvantage of this method is that each piece is handled twice, once to erect it and again when the entire arch is moved. An advantage is that the construction is moved away from the navigable opening and that the erection can take place in a relatively more protected area.

Continuous span tied arches are typically erected in their final position. Falsework may be required to erect the side spans, but the center span is almost always erected using free cantilevering with the side spans used as anchor spans.

Figure 15 Route 364 Arch - Initial Erection on Approach Piers

The Route 364 arches were erected by the float-in method (Figures 15 and 16). The Missouri River carries a good bit of floating debris and the erector was leery of using falsework having experienced problems at other projects along the river. The method employed had an interesting twist, however. Instead of erecting the arches on extensive temporary falsework, the

Figure 16 Route 364 Arch - Float In Operation

Contractor elected to use the side span piers for erection. These piers had been designed for barge impact and provided a safe erection platform. The piers required only minor strengthening. The only major difficulty with this scheme was that the side span approaches could not be completed until the arches were both moved into their final positions.

SUMMARY

The paper has reviewed various topics associated with the design of steel tied arch bridges, using a recently constructed bridge to illustrate their application. Steel tied arches remain an attractive, economical and structurally viable option for major bridges.

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