Practical Steel Tub Girder Design
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Abstract
This paper addresses the entire design process for a steel tub girder bridge and offers a list of pertinent references for each phase. The paper presents preliminary design considerations, including appropriate applications for steel tub girders; preliminary girder sizing and spacing guidelines; framing plan layout considerations; and suggestions related to preliminary (approximate) design. Also discussed are issues related to final, detailed design, including available analysis tools; specifics on design of numerous components of a steel tub girder bridge; suggestions on steel tub girder detailing; and special considerations for construction of a steel tub girder bridge.

While this paper is not a stand-alone design manual for steel tub girders, it endeavors to cover the entire design process in outline form in one document, while providing an extensive list of references the designer can consult. Because nearly every steel tub girder bridge is unique in terms of configuration, site conditions, local fabrication and construction preferences, and myriad other factors, the paper typically will outline issues and considerations related to specific questions, rather than attempting to provide hard and fast rules, in the hope of enabling designers to make better-informed final decisions for their projects.
INTRODUCTION
Steel tub girder use is becoming more commonplace in modern infrastructure design. They offer advantages over other superstructure types in terms of span range, stiffness and durability—particularly in curved bridges. In addition, steel tub girders have distinct aesthetic advantages due to their clean, simple appearance (Figure 1). However, steel tub girder design is in many ways more complex than steel plate girder design, especially for construction loading stages.

This paper addresses the entire design process for a steel tub girder bridge and offers a list of pertinent references for each phase. The paper presents preliminary design considerations, including appropriate applications for steel tub girders; preliminary girder sizing and spacing guidelines; framing plan layout considerations; and suggestions related to preliminary (approximate) design. Also discussed are issues related to final, detailed design, including available analysis tools (the M/R Method, grid analysis, and three-dimensional finite element analysis); specifics on design of numerous components of a steel tub girder bridge; suggestions on steel tub girder detailing; and special considerations for construction of a steel tub girder bridge.

While this paper is not a stand-alone design manual for steel tub girders, it endeavors to cover the entire design process in outline form in one document, while providing an extensive list of references the designer can consult. Because nearly every steel tub girder bridge is unique in terms of configuration, site conditions, local fabrication and construction preferences, and myriad other factors, the paper typically will outline issues and considerations related to specific questions, rather than attempting to provide hard and fast rules, in the hope of enabling designers to make better-informed final decisions for their projects.

Reference 1 and the anticipated Reference 2 also provide extensive, current guidance on the topic of steel tub girder design.

STEEL TUB GIRDER APPLICATION ISSUES
There are many reasons to choose steel tub girders—which offer several distinct advantages over steel plate girders and other superstructure types—for a project. However, steel tub girders are not a panacea. Designers should carefully consider each bridge on a case-by-case basis to determine if steel tub girders are an appropriate superstructure choice. Several keys issues to evaluate include:

- Span ranges—Steel tub girders generally are economical in middle span ranges, from about 150’ to 500’.
- Curvature—As closed cell structures, the torsional strength and rigidity of tub girders make them excellent choices for curved structures with tight radii of curvature.
- Aesthetics—Their smooth uncluttered form and the reduced number of girders give steel tub girder bridges a clean, simple appearance.
• Durability/Maintainability—Since many elements of tub girders are located inside the box section, protected from the elements, they exhibit greater durability.

• Economy—Tub girders are not inexpensive, but in appropriate applications offer economically competitive solutions to tough design situations. Furthermore, as the use of tub girders increases, their unit cost should decrease, making them more cost effective for a wider range of situations.

References 1, 3, 4, 5 and 6 offer good discussions on many of these key issues.

DEPTH, WIDTH AND SPACING OF TUB GIRDERS

Depth-to-Span Ratios

The traditional rule of thumb for steel bridge girder depth of L/25 (with L measured between points of contraflexure) is a good starting point for steel tub girders. The L/25 guideline is mentioned in Section 12 of the latest version of the AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges (7) and Section 2.5.2.6.3 of the AASHTO LRFD Bridge Design Specifications (8). However, designers should not be afraid to exceed this ratio; tangent steel tub girders have approached L/35 and still met all code requirements for strength and deflection control. Keep in mind, the minimum recommended web depth for tub girders is 5', as cited by several writers (4, 9). Note that steel tub girder depth affects more than just commonly considered implications, such as vertical clearance below a bridge. Because most tub girders use sloped webs (typically sloped at 1H:4V), web depth also directly affects bottom flange width (discussed later).

Tub Girder Width and Spacing

As a lower bound, at least one writer suggested a minimum width of 4' be maintained for tub girders (9) to allow sufficient room for workers to complete fabrication of various internal girder details. As an upper bound, there are no hard and fast rules. Upper limits on tub girder width (and tub girder spacing) often are controlled indirectly by three central issues:

• Bottom Flange Width—Extremely wide bottom flanges likely will require the addition of longitudinal stiffeners and/or transverse stiffeners for stability (see further discussion below).

• Deck Transverse Span—Wide girder spacings can offer some significant opportunities for taking advantage of some of the inherent efficiencies of tub girders and reduce total steel weight, reduce construction efforts, and lower initial construction costs. However, some owners (e.g., state departments of transportation) are uncomfortable with wide girder spacings, both within each tub girder and between girders. One concern is constructability. Wider deck spans are more difficult to form, and some owners have very economical deck details, including permanent metal deck forms or stay-in-place precast concrete deck panels, which may not be suited for wider girder widths and spacings. Another issue is the ability of wide-span decks to accommodate certain overload conditions. Finally, redecking while maintaining traffic may be quite difficult on bridges with wider girder widths and spacings.

• Transportation Limits—Note that tub girder widths above approximately 12' present transportation difficulties. Widths above 14' may require provisions for a longitudinal bottom flange splice so the tub can be transported in two sections.

The ratio of the width of an individual tub girder to the spacing between tub girders also should be considered in accordance with Article 10.39.1.1 of the AASHTO Standard Specifications for Highway Bridges (10) or with Article 6.11.2.3 of the AASHTO LRFD Bridge Design Specifications (8). However, exceeding these limits may be acceptable, provided a more
Refined analysis that more rigorously quantifies live load distribution is performed. Designers should take care to consider all implications if they choose to exceed these limits.

Overall width of a bridge also is a controlling factor in setting individual tub girder widths and spacings. In many narrower bridges carrying only one lane of traffic plus shoulders (such as direct connector ramp structures in multilevel interchanges), total out-to-out deck width may be as narrow as 28' or less. In longer spans requiring deep girders, two tub girders with webs sloped at 1H:4V may end up with overly narrow bottom flange widths. Single tub girder bridges have been successfully used in these situations, but beware that using a single tub girder may result in an excessively wide bottom flange as well as design, detailing and construction complexities associated with providing two bearings per tub girder. Also, during construction, care must be taken to ensure that a single tub girder is sufficiently braced at all stages of erection to avoid the potential for an unstable system. See Reference 1 for more information.

Designers are advised to evaluate these considerations early on, before even preliminary structural design calculations are performed.

ANALYSIS AND DESIGN METHODS
Final design of steel tub girder bridges is a detailed and intensive process. Care taken during the preliminary design phase will pay dividends many times over later on. Thorough consideration of design, detailing and construction issues up front will result in better, more efficient and easier-to-construct bridges. Before beginning to run numbers, designers are advised to consider numerous framing plan issues discussed later.

Full discussion of each level of analysis mentioned below is beyond the scope of this paper. The reader is instead directed to References 1 and 11 for more detailed treatment of this topic.

Tangent Tub Girder Design Techniques and Tools
Straight bridges containing multiple tangent tub girders can be designed using a single girder model. Design can be facilitated using any of several commonly available tangent girder programs, some of which are developed specifically for tub girder design. Others normally are only applicable to plate girder or rolled beam design, but can provide sufficient design information to shortcut some tedious analysis steps.

For shear design, designers must remember that all tub girders carry torsion, which increases effective design shear in one web. If a single girder model is used, torsional moments can be approximated by hand. If a multiple girder computer analysis model is used, torsional moments should be available from the analysis and additional web shears can be derived.

In addition, because most tub girder webs are sloped, the designer must account for the increase in resultant web shear due to web slope, as well as the increase in web depth along the slope.

Approximate/Preliminary Curved Girder Design
In slightly curved bridges, it is helpful to know that using the straight girder methods previously mentioned in this paper would result in a design very close to the final design where torsion is considered and resulting cross-section sizes are good candidates for the trial member sizes in the final design model. In cases where the curvature is more severe and would affect primary bending moments, an approximate analysis can still be performed using straight girder methods as the basis, but with the straight girder results amplified using factors to account for curvature.
found in Reference 12. When analyzing a curved bridge using tangent girder methods, developed span lengths of the tub girders should be used.

**Effects of Curvature, M/R Method**
Designers must remember that tangent girder programs do not account for load shifting and torsional effects caused by horizontal curvature. It is up to designers to account for these effects in order to correctly quantify design loads on a tub girder.

In general terms, an overturning moment (moment about the longitudinal axis of the bridge) occurs in curved bridges, since the centers of gravity of each girder, and of the bridge as a whole, are offset from a chord line drawn between support points of each span. When girders act independently of each other (for example, self-load of an individual girder, or deck placement when no intermediate external diaphragms are provided), this effect results in torsion in each tub girder. These torsional moments can be determined using the M/R Method, an approximate, hand calculation method for torsional analysis of single curved tub girder bridges developed by Tung and Fountain (5, 12 and 13).

The M/R Method can be applied to effectively calculate the torsional moment in single girders or in girders under construction before adjacent tub girders are tied together by intermediate diaphragms and slab. Because cross-sectional rotations also can be obtained using the M/R Method, it is possible that the method can be applied to multiple girders connected by intermediate diaphragms to solve the load shifting by iterations. However, the process would be too complicated without additional computational aids. More effective methods to consider the interaction of the girders are grid analysis and Three-Dimensional Finite Element Analysis (3-D FEA).

**Grid Analysis**
Multiple girders and diaphragms can be modeled in a grid analysis; most commercial programs for curved tub girder design are based on this method. A two-dimensional grid is used to define the overall geometry, with line elements used to model the girders and diaphragms. Most programs distribute live loads to girders using the lever rule.

Grid analysis is popular because building a grid model is relatively simple and fast. However, designers are cautioned that the many simplifying assumptions required to model a complex structure in a 2-D grid model requires experience and good judgment. Also, grid analysis does not directly account for all related deformations, including bending, shear, torsion, axial deformation and cross-sectional distortion.

For final design of curved or severely skewed tub girders, a grid analysis is the minimum level of analysis that should be considered.

**3-D FEA**
3-D FEA is a method in which girder plates are modeled using plate/shell elements, while a concrete slab is modeled using either plate/shell elements or solid brick elements. Bracing members also are included in the analysis, usually modeled with either truss or beam elements.

3-D FEA methods are intended to create highly inclusive and accurate models that can replace all other analysis methods. However, building, running and post-processing a 3-D FEA model requires significant effort and may result only in marginal improvement in refinement of the analysis.
Three-dimensional analysis programs and services exist that include code-checking procedures for main girders. These are powerful analysis tools if used properly; however, design procedures still require other supplemental structural analysis, as demonstrated in the design example in the current AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges (7).

**GIRDER DESIGN**
The girder design process for tub girders should begin with development of a framing plan that includes consideration of all of the many parts of a tub girder (Figure 2). Many decisions made early in the design process can have significant impacts later on; some basic issues are presented below. Structural analysis as discussed above must be performed to derive the load effects, which are subsequently checked with the code provisions.

Tub girder bridges must always be designed considering construction sequence. Some structural members, such as lateral bracing, are provided only for construction purposes. Their design is consequently controlled by construction loading. The analysis must therefore be performed on the partially completed structure, simulating the sequence of construction loading. Total stresses are the sum of those generated due to loads on the complete bridge and those locked-in during construction.

Numerous guides and design examples are available that specifically discuss girder design and code provisions in detail (3, 5, 6, 7, 10, 14 and 15).

Designers are cautioned that narrow tub girder bridges with only one or two girders in their cross-section may be considered fracture-critical. However, recent revisions to the AASHTO LRFD Bridge Design Specifications (8) allow for demonstration of redundancy by means of rigorous analysis.

**Bottom Flanges**
Bottom flange thickness and b/t ratios may seem like minor details better left to final design. However, designers of steel tub girders are well-advised to consider this issue up front while developing a framing plan, because many potentially complex and costly ramifications may result from the simple choice of bottom flange width and thickness.

The AASHTO Standard Specifications for Highway Bridges (10) suggest that for b/t ratios greater than 45, “longitudinal stiffeners be considered” and that b/t ratios greater than 60 are not permitted for compression flanges. The AASHTO LRFD Bridge Design Specifications (8) replace these provisions with more general guidance in Article 6.11.8.2.3 and its associated commentary. The b/t limit of 45 for consideration of longitudinal stiffeners is intended to be a rule of thumb; with a b/t above 45, it may be more economical to add a longitudinal stiffener to the bottom flange to increase bottom flange capacity without thickening the flange. Because they are costly additions and their use may result in undesirable fatigue details, careful consideration...
should be given before adding longitudinal stiffeners. Engineers often find it more economical to simply thicken the bottom flange in lieu of using longitudinal stiffeners.

Even in positive moment regions, there are lower bound limits for bottom (tension) flange thickness. As stated in Preferred Practices for Steel Bridge Design, Fabrication, and Erection prepared by the Texas Steel Quality Council (16):

“For wide bottom flanges, plate distortion during fabrication and erection can be a problem. Designers should check with fabricators when using bottom tension flange plates less than 1” thick to determine whether practical stiffness needs are met. In no case should bottom tension flanges be less than 1/2” thick. Another suggested guideline is that the bottom tension flanges have a b/t ratio of 80 or less.”

Other writers have suggested a maximum b/t ratio of 120 for bottom flanges in tension (17).

For extremely wide and/or slender bottom flanges, transverse stiffeners may be required. Bottom flange transverse stiffeners serve several purposes, including bracing bottom flange longitudinal stiffeners and stiffening bottom flanges for torsional shear stresses. Again, care should be taken before adding transverse stiffeners to bottom flanges, since they will be costly and may result in constructability and fatigue problems if not carefully detailed.

Some detailing guides, such as Preferred Practices for Steel Bridge Design, Fabrication, and Erection (16), provide more detailed suggestions regarding tub girder bottom flange thickness and b/t ratios. These issues also are discussed in the Commentary to Section 10.4.2.4 of the AASHTO Guide Specification for Horizontally Curved Steel Girder Highway Bridges (7). Designers should carefully review the issues presented in these documents and seek guidance from local steel bridge fabricators with tub girder experience regarding relative cost issues before making hasty assumptions regarding bottom flange thickness.

Webs

Although tub girders carry vertical shear similar to plate girders, they also carry torsional shear stress. Tub girders are very efficient in carrying torsion, so this generally does not present a significant design challenge. The shear flow can be obtained from the torsional moment using the formula \( q = \frac{T}{2A} \), where \( A \) is the area enclosed by the tub girder webs, flanges and slab (or lateral bracing, if investigating girder prior to deck hardening). However, designers should remember that the webs will carry more shear than what might be predicted by an approximate, tangent girder analysis, and thus increased thickness or additional transverse stiffeners may be required. Recent experience has shown that providing a reasonable number of transverse stiffeners is currently more economical than providing either a thinner web with extensive transverse stiffeners or a thicker web without.

It should be noted that all tub girders have torsion; even tangent tub girders will be subject to some level of torsion from a variety of causes. Some potential sources of torsion in tangent (and curved) tub girders include:

- Skew—Skew increases torsion in tub girders, because web span positions relative to various load points are no longer symmetrical from one web to the next.
- Asymmetrical Non-Composite Loading—External girders in particular can be subject to asymmetrical loading during deck placement, since overhang widths often are not equal to the tributary deck width between adjacent girders and at phased construction lines. This effect can be controlled/reduced by use of intermediate external diaphragms and/or lateral bracing.
Asymmetrical Live Loading—The Commentary for Article 9.7.2.4 of the AASHTO LRFD Bridge Design Specifications (8) offers a good discussion of this issue. Because tub girders have very high torsional stiffness, they can develop torsional loading during asymmetrical application of live load. Plate girders do not experience this phenomenon, since they are very flexible torsionally and twist or rotate to accommodate the deck behaving as a simple span. For tub girders, this phenomenon is more important for deck design (it can control the design of the deck slab), but it is worth noting here as another source of torsion in tangent and curved tub girders.

Top Flanges
Top flanges of tub girders are designed primarily to carry the girder bending stresses. Additional longitudinal stresses due to torsion exist, as the flanges also serve as members of the truss system used for lateral bracing. Top flanges also are subjected to significant lateral bending stresses. These lateral bending stresses can be generated by horizontal girder curvature (6), sloping webs (18), and temporary supporting brackets for slab overhangs (7). In addition, forces from lateral bracing systems may represent a major source of lateral flange bending stresses and should also be considered in design. See the recent article by Fan and Helwig (18) for a more detailed discussion.

Design provisions (7) suggest that lateral bending of top flanges can greatly affect the portion of capacity allocated to bending stresses. Increasing top flange width is generally more effective for resisting the lateral bending stresses than increasing top flange thickness. Sufficient top flange width also is necessary to provide room for connection of lateral bracing members. However, the recommended b/t ratio for top flanges in tension or compression should be followed carefully.

Critical design stages for top flanges often occur during construction prior to the deck curing, when the flanges are laterally braced only at the K-frame locations. Lateral bending stresses due to live load effects can be neglected in the capacity check when top flanges are embedded in the hardened concrete with shear studs, except in areas where shear studs are not provided. However, curved tub girders typically are provided with shear studs along the entire girder length to achieve the desired torsional rigidity from a closed cross-section.

In addition, top flanges also are subjected to erection loads, as most contractors lift steel girder sections by clamping the top flanges. Ideally, local stresses in the top flanges, including stresses in the weld between the flanges and web, should be checked for these erection loads.

DIAPHRAGMS AND BRACING
Internal Intermediate Diaphragms
Internal intermediate diaphragms are provided in tub girders to control cross-sectional distortion, which introduces additional stresses in box girders and should be minimized. Cross-sectional distortion is caused by torsional loads that do not act on boxes in the same pattern as the St-Venant shear flow, which is uniformly distributed around the perimeter of the tub girder cross-section. Unfortunately, most torsional loads on box girder bridges—such as eccentric vertical loads and curvature-induced torsion—fall in this category and will introduce distortion. Because a true 3-D FEA analysis using plate/shell elements will give total stresses induced by bending, torsion and distortion, a separate distortional analysis is not needed. However, all design programs based on grid analysis using line elements are unable to predict distortional behavior of box girders.
Distortional stresses can be neglected in design, if a sufficient number of internal diaphragms with adequate stiffness are provided. Research efforts in the early 1970s resulted in guidelines for spacing and stiffness requirements of internal diaphragms. While there might be conceptual deficiencies in these previous investigations, experience in the past three decades has proven that the guidelines are, in general, conservative. Those studies focused primarily on X-type cross frames. Most cross frames in modern box girder bridges are K-frames, allowing better access for construction and inspection. With very narrow boxes, consideration of X-frame or Z-frame configurations might be warranted. Until more studies on K-frames are available, previous data for X-type cross frames can be judiciously used for K-frames. It should be noted that the current AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges (7) and AASHTO LRFD Bridge Design Specifications (8) have omitted these previously published guidelines.

Spacing of internal diaphragms is part of the framing process, and should be determined by also considering factors such as the angle and length of lateral bracing members, as discussed below. The stiffness requirement often results in very small sizes of cross frame members, which typically are structural angles. Members are sometimes selected based on handling considerations; cross frame members must also be checked to meet strength requirements. Large brace forces can develop inside cross frames, particularly when they are spaced relatively far apart in curved bridges. Approximate design equations have recently been developed to estimate these forces (19). Strength verification should be conducted on diaphragms with controlling (largest) brace forces, usually located in the largest bending moment regions. The above-mentioned guides (5, 7, 19, 20 and 21) offer good discussions on sizing and spacing issues.

Internal cross frames also act as braces to prevent girder top flanges from buckling. However, the flange stability issue has not been well addressed in previous research. Current design guidelines based on distortional stresses likely will continue to be the primary approach in years to come.

Internal cross frames should be detailed to minimize fatigue concerns. Stiffeners to which bracing members are attached should be connected to girder flanges to reduce local distortion.

**External Intermediate Diaphragms**

It is widely known that intermediate diaphragms in curved I-shaped plate girder bridges are primary structural members and are always needed. Intermediate diaphragms prevent individual girders from rotating independently under torsional loads and tie the girders together such that torques are resisted by a structural system with a large rotation arm and are thus much stiffer than individual girders. Individual I-shaped plate girders have low torsional stiffness and can develop significant rotation if not restrained.

Tub girders, on the other hand, possess larger rotational stiffness and are much more capable of carrying external torques individually. In finished bridges, when tubs are fully closed and the concrete deck effectively ties the girders together, transverse rotations of tubs are expected to be small and external intermediate diaphragms typically are not warranted. During construction, however, the rotational rigidity of tub girders is not as high. Steel tubs at this stage are quasi-closed, and their rotational behavior is not fully understood or predictable. Research recently performed at the University of Texas will provide more insight into this behavior; publication is expected by fall 2005 (2). The main reason for concern is the consequence of transverse rotations. Because a girder’s two top flanges are spaced apart but still rotate together,
resulting differential deflections at the top flanges would be large, even with a small girder rotation. Reasonable fit-up may become impossible and differential slab thickness will result at the top flange locations if girder rotation is not controlled.

Consequently, external intermediate diaphragms are used to control differential displacement and rotation of individual tub girders during slab placement. In tangent tub girders, external intermediate diaphragms can sometimes be omitted or minimized if individual tub rotation and differential displacements are expected to be relatively small and manageable. External intermediate diaphragms often utilize a K-frame configuration, with depth closely matching girder depth for efficiency and simplification of supporting details.

The necessity of external diaphragms has been a debated topic for years, and tub girder bridges have been built with and without external diaphragms. Problems with slab placement have occurred on some curved girders without external intermediate diaphragms.

Many previous and current research investigations report that external intermediate diaphragm forces are small and concluded that they are not needed. Yet girder rotations are frequently observed in curved bridges. For example, bolt holes do not align during installation. Many engineers and contractors tend to be conservative with this issue and like the additional stiffness and rotation control that external intermediate diaphragms bring to structures. It is likely they will continue to be used unless strong evidence suggests otherwise, or a convenient, reliable tool becomes available to predict girder rotation.

There also is ongoing debate regarding external intermediate diaphragm removal after the deck has cured. Some advocate removal of these diaphragms for aesthetic reasons. However, care should be taken in evaluating the effects of removing "temporary" external intermediate diaphragms; at a minimum, five issues should be addressed:

- **Safety**—Removing temporary external intermediate diaphragms can be difficult and potentially hazardous, due to falling debris concerns or if members are carrying significant load. Crane access is limited once the deck is in place.
- **Deck Stresses**—Loads carried by temporary external intermediate diaphragms will shift to the deck when the diaphragms are removed, adding to deck stresses and potentially leading to deck cracking if this effect is not carefully evaluated in the deck design.
- **Future Redecking**—In locations where future redecking of the bridge is likely (for example, to address deck deterioration in regions where the heavy use of deicing salts is routine), consideration should be given to retaining external intermediate diaphragms, since they would likely be required during redecking for the same reasons they were required during the original construction.
- **Traffic Control**—If lane closures are required to remove external diaphragms, salvage value of removed diaphragms will be far less than costs to close lanes.
- **Fatigue**—Leaving external diaphragms in place may present fatigue concerns, since connection of diaphragm bottom chords to the girder bottom flanges is problematic and connection to the web with stiffener backups may result in a poor fatigue detail.

In Texas, external K-frames currently are provided every two to three internal K-frames, and typically are removed after slab construction.

**Internal Diaphragms at Supports**

Pier and end diaphragms generally are full-depth plate girder sections. Particular care should be taken in detailing end diaphragms for constructability, because the presence of abutment backwalls, other girders or pier cap stems will limit access to one side of the diaphragm during
erection. Note that pier diaphragms (and sometimes end diaphragms) require access manholes for future inspection.

Internal diaphragms at supports are designed as deep beams subjected to 1) bending loads, which are the shear forces from the girder webs; and 2) torsion-induced shear flow around the perimeter of the diaphragms, due to torsional moment reactions on box girders.

Internal diaphragms typically consist of a vertical plate. Top flanges can be provided for the diaphragm to increase the bending capacity of the diaphragms, as pointed out above, and also can provide support for the slab at girder ends. Bearing stiffeners can be attached to diaphragms and are designed as columns subjected to an axial load equal to the reaction. Access holes should be provided for inspection purposes; in determining the location and size of the holes, it is important to consider the stress flow due to bending and torsional moments.

Diaphragms are supported by one or two bearings. Two-bearing supports provide better torsional resistance and induce less bending stress in the internal diaphragms. However, two-bearing supports are not often recommended, due to width limitations of bottom flanges and high demand for construction accuracy. Single supports are more widely used, and the torsional resistance—as well as the distributional reactions—resulting from the large bearing contact area often is conservatively neglected. Bearing design is discussed in more detail in Reference 1.

**External Diaphragms at Supports**

If dual bearings or other measures (such as anchor bolts or shims) under the girder are able to prevent transverse rotation, external diaphragms could, theoretically, be stress-free. If single-point support is used, however, torsional rotations at the ends of the girders must be resisted by external diaphragms.

Large torsional reactions may occur at the girder supports, leading to the use of solid plate girders for diaphragms in many curved bridges. These solid plate girder diaphragms are subjected to bending and shear as torsion in individual girders is resolved via a force-couple between the supports of adjacent girders. The bending moments and shear forces in the diaphragms can be determined by assuming that external and internal diaphragms are combined to form a beam supported at the bearing locations. The loads on the beam are moments equal to the resisting torques for girders.

Since these moments are acting at the supports of the beam while the middle span portions of the beam represent the external diaphragms, this approach typically yields bending moments on external diaphragms that are smaller than resisting torsional moments on the girders. However, the accuracy of this model could be compromised by potential eccentricities of the bearing supports and other uncertainties. Alternately, it is reasonable and usually conservative to directly use the torsional reactions of the box girders as the design moments for the external diaphragms. Shears on the diaphragms can be evaluated using the same model \((J)\). Torsional reactions at girder supports should be available from global bridge analysis results. The largest torsion is very likely to occur during the construction stage.

The loads in these diaphragms may suggest the use of a moment connection to the girder (where the diaphragm flanges are connected to the girder); the connection may be designed in a similar manner to the splice design for the girder.

Torsional moments in straight girders are significantly smaller, so a plate girder may not be warranted for external diaphragms; a truss-type diaphragm may be sufficient in these cases.
Avoid skews in tub girder bridges if at all possible—especially at girder ends—since skewed end diaphragms are enormously complicated to detail, fabricate and erect. Pier diaphragms can be detailed as pairs of perpendicular diaphragms to avoid problems; note however that end moments in these types of paired perpendicular diaphragms may be high since they may be subject to significant racking due to differential deflection of adjacent girders along the skew.

Top Flange Lateral Bracing
Top flange lateral bracing is required in tub girders to form the “fourth side” of the box until the slab is in place. The previously mentioned guides (5, 7, 18, 20 and 21) offer good discussions on sizing and spacing issues. This bracing typically is formed with WT or angle sections, often configured in a single diagonal (Warren truss) or double diagonal (X-bracing truss) arrangement.

The purpose of providing a lateral bracing system is to increase the torsional stiffness of tub girders. However, the relationship between torsional stiffness and lateral bracing systems (including connection) has not been well studied. Analytical formulas are available to calculate the thickness of an equivalent plate converted from the horizontal truss, although there is no experimental verification on the stiffness aspect of this method. Rotation of tub girders also is likely to occur if bolted connections of the lateral bracing members experience slip. A better understanding of the quasi-closed tub girder stiffness would impact the analytical method and the use of external diaphragms.

Design of a top lateral bracing system consists of three major steps:

1. Framing—The angle between diagonal lateral bracing members and the top flange, $\alpha$, is an important design parameter. Ideally, an $\alpha$ around 45 degrees is desired. A small $\alpha$ leads to larger cross frame spacing and fewer bracing panels and connections. However, larger brace forces and girder stresses result from a small $\alpha$. Engineers should plan the framing using both structural and economical considerations.

   Diagonal lateral bracing members are framed into the intersection of the girder top flange and internal diaphragm. For narrow girders, the guidelines discussed earlier may suggest a cross frame spacing much larger than the girder width, which would result in a small $\alpha$. Therefore, location of the internal diaphragms should be planned with consideration of lateral bracing.

   An alternative is to divide the spacing between two consecutive cross frames into more than one lateral bracing panel. Single lateral members (typically angle sections) are used at those dividing lines. Attaching two more members to the single lateral to form a vertical K-frame does not significantly increase steel weight, but does add cost, due to increased fabrication. Note that doing so also makes the structure stiffer, and brace forces in both the lateral bracing members and internal diaphragms may be slightly larger.

   It is suggested to keep the plane of the top flange lateral bracing reasonably close to the plane of the top flanges, which increases the torsional efficiency of tub girders and avoids excessive bending loads in web stiffeners.

2. Analysis—Torsional moments induced by dead loads and construction loads will result in brace forces in lateral bracing members. These forces can be derived from the St.-Venant shear flow at girder cross-sections, assuming the horizontal truss acts as an imaginary plate.

   Lateral brace members, transverse struts in internal diaphragms, and top flanges form a geometrically stable horizontal truss. It is therefore possible to take axial forces representing the top flange component of girder bending moments and calculate member forces in the lateral
bracing due to box girder bending. Design equations have been developed to evaluate this component of force for different truss types (18).

Vertical loads on the top flanges also induce brace forces due to web slope. However, the majority of these forces are resisted directly by lateral struts. Thus, the total forces in lateral bracing members are essentially the sum of torsional and bending components. Because bending forces are a function of member size, trial truss member sizes must be assumed before the force calculation. External diaphragms also contribute load to the bracing system. If a 3-D FEA program is available, total forces can be obtained directly from the analysis.

3. Member Size Selection and Connection Detailing—Once forces are determined, brace members can be designed as beam columns. Depending on the connection details, bending moments resulting from any eccentricity must be considered. The effective length factor (k) used in design of brace members should reflect end connection conditions.

MISCELLANEOUS DETAILS
Many other details associated with steel tub girder design have unique aspects. These details include:

- Flange-to-web welds
- Shear studs
- Bearings
- Bridge decks
- Field splices
- Steel detailing
- Material selection
- Painting

The reader is directed to Reference 1 for in-depth discussions of these details.

CONCLUSION
Steel tub girders often are an excellent choice for modern highway bridge superstructures. They offer advantages over other superstructure types in terms of span range, stiffness and durability—particularly in curved bridge applications. Steel tub girders also offer distinct aesthetic advantages, due to their clean, simple appearance. The increased use of tub girders in the future could address many demands for improved bridge aesthetics, efficiency, and durability.

However, the layout, design, detailing, fabrication and erection of steel tub girder bridges is in many ways different and often more complicated than for steel plate girder bridges. Designers are advised to understand and evaluate all these issues very early in the design process.

Designers should also realize that in many cases, there are no hard and fast rules associated with these issues. Each tub girder project is unique and a solution that worked for one bridge might not necessarily work for another. Instead, it is imperative that the designer of a tub girder bridge explore all options on a case-by-case basis, often in consultation with local fabricators and/or erectors.

A full understanding and careful consideration of all these issues will result in a more successful tub girder bridge project.
REFERENCES


