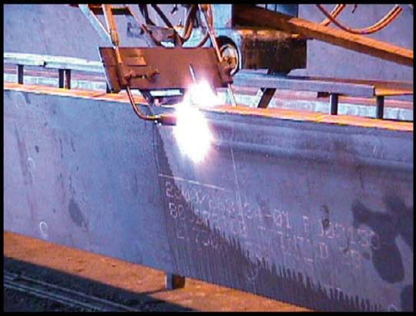
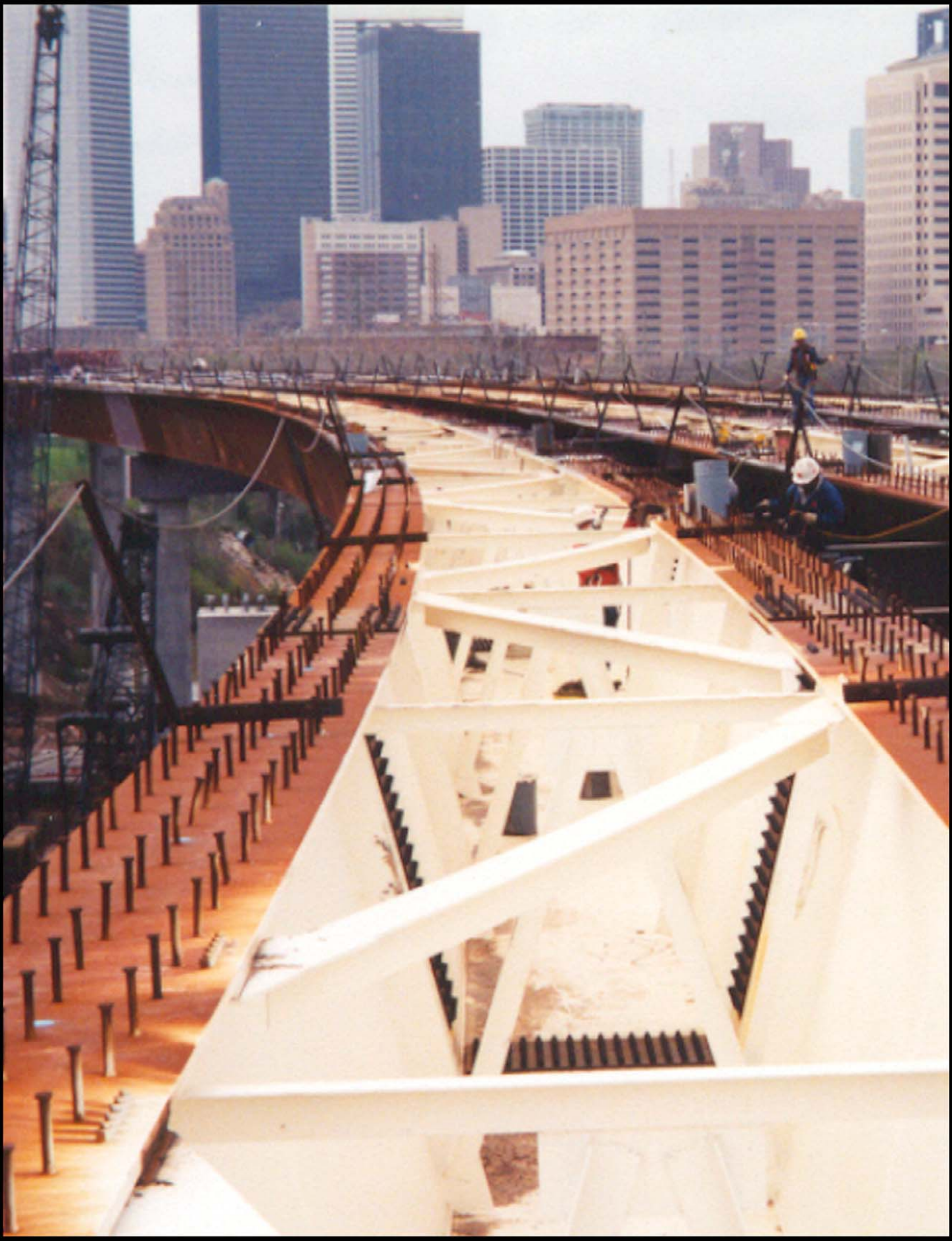
PRACTICAL STEEL TUB GIRDER DESIGN





NATIONAL STEEL BRIDGE ALLIANCE

PRACTICAL STEEL TUB GIRDER DESIGN

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The National Steel Bridge Alliance has published this document in its continuing effort to enhance state-of-the-art steel bridge design and construction in the United States.

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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. Steel tub girder design is in many ways more complex than steel plate girder design, especially for construction loading stages. Yet there is no single, comprehensive source of information on steel tub girders. Instead, bits and pieces of layout, design, detailing and construction guidance are scattered among a broad and disconnected collection of design specifications, guidebooks, textbooks, articles and formal design examples, many of which are out of print or otherwise difficult to obtain. Designers faced with the task of preparing plans for a steel tub girder bridge often have to start with an extensive – sometimes frustrating – literature search, hoping to find enough advice from among several sources to guide them from preliminary design through final detailing.

A much anticipated tub girder publication is forthcoming from the University of Texas and the University of Houston (25) but was not available at the time of publication of this book. Another much anticipated publication is the upcoming 2005 Interim Revisions to the AASHTO *LRFD Bridge Design Specifications* (26), in which the design provisions for straight and curved steel girder bridges (both I girders and tub girders) are "unified" into a single design specification document.

This book addresses the entire design process for a steel tub girder bridge and offers a list of pertinent references for each phase, including suggestions on how to find some hard-to-locate documents. The book 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. The book also discusses issues related to final, detailed design, including various available analysis tools (the M/R Method, grid analysis, and three-dimensional finite element analysis); specifics on design of the numerous components of a steel tub girder bridge (girders, internal and external diaphragms, lateral bracing members, stiffeners, bearings, deck, field splices, etc.); suggestions on steel tub girder detailing; and special considerations for construction of a steel tub girder bridge.

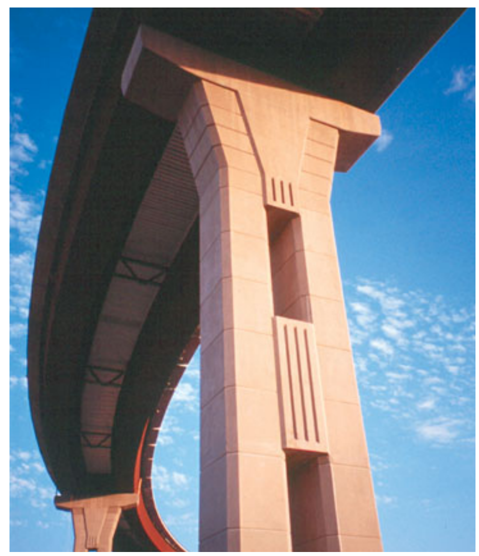


Figure 1: A bold statement in modern highway bridge design, steel tub girders offer advantages in span range, ability to accommodate curvature, and aesthetics.

While this book is not a stand-alone steel tub girder design manual, it endeavors to cover the entire design process in outline form in one document, while providing an extensive list of references that 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 book will typically outline issues and considerations related to specific questions rather than attempting to provide hard and fast rules, in the hopes of leaving designers better informed to make their own final decisions for their project. Finally, some recent steel tub girder bridge experience in Texas is presented, with key issues and lessons learned highlighted.

There are many reasons to choose steel tub girders, which offer many distinct advantages over steel plate girders and other superstructure types...

There are many reasons to choose steel tub girders, which offer many distinct advantages over steel plate girders and other superstructure types. 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 are listed below.

Figure 2: Steel tub girders under construction for a new Automated People Mover system. Tight curvature and clean appearance were key factors in choosing steel tub girders.

SPAN RANGES

Steel tub girders can potentially be more economical than steel plate girders in long-span applications, due to the increased bending strength offered by their wide bottom flanges and thanks to less field work associated with handling fewer pieces. However, their use in long-span applications should be evaluated with due consideration given to increased fabrication costs of tub girders, particularly if bottom flange longitudinal stiffeners or other complicated details are required. Also, lifting weights may be higher with tub girders. Spans in excess of 500 feet have been successfully constructed; for instance, in October 1998 the West Virginia Department of Transportation completed the Lower Buffalo River Bridge carrying WV 869 over the Kanawha River. That bridge features a five-span tangent tub girder unit with a 525-foot-long center span, the third longest tub girder span in the United States.

In short span ranges, steel tub girders can be an economical solution, particularly if considerations such as aesthetic preferences preclude other structure types. However, it is increasingly difficult to design efficient steel tub girders for span lengths shorter than those requiring the rule-of-thumb 5-foot minimum web depth (minimum depth needed to provide accessibility for future inspection). This limit works out to be below approximately 150 feet for simple spans and 200 feet for continuous spans. One writer has suggested a desirable lower span length limit of 120 feet (1). In lower span ranges, and/or when curvature is slight, other shapes may prove to be more efficient than steel tub girders.

Figure 3: For slight curvature and spans below approximately 120 feet, other superstructure types, such as the curved steel plate girders in this shallow depth curved grade separation bridge, may be a better choice than steel tub girders.

CURVATURE

While steel tub girder use is not limited to curved structures, they do offer definite advantages in curved bridge applications. Torsional stiffness of tub girders is many times greater than that of plate girders, resulting in superior transverse load distribution characteristics. Tub girders are also extremely efficient in carrying torsional loads found in curved bridges, requiring far fewer diaphragms between girders. Steel tub girders accommodate extremely tight radii of curvature and have

Steel tub girders accommodate extremely tight radii of curvature and have been used in this role from their earliest applications...

been used in this role from their earliest applications. Among the earliest uses of steel tub girders in the United States were two bridges built on horizontal radii of 150 feet in Massachusetts in the early 1960s (2). Later, a series of steel tub girders were constructed at the Dallas/Fort Worth International Airport in the late 1970s on 175-foot horizontal radii. On the down side, the complexity of fabricating trapezoidal box shapes to accommodate vertical curvature, horizontal curvature, super-elevation transitions, and/or skews is challenging and contributes to a cost premium versus other structure types.

Figure 4: Steel tub girders built on horizontal radii as tight as 175 feet were constructed at the Dallas/Fort Worth International Airport in the late 1970s.

AESTHETICS

One of the reasons most often cited for using steel tub girders is aesthetics. Bracing, stiffeners, utilities, and other components are typically hidden within the box, resulting in a smooth, uncluttered form. Because a single tub girder can essentially take the place of two plate girders, the number of visible components is minimized, again leading to a reduction in visual clutter.

DURABILITY/MAINTAINABILITY

Steel tub girders offer advantages in durability and maintainability. The steel surface area exposed to the environment is greatly reduced, since half of the web and flange surfaces are enclosed. In addition, elements routinely subjected to debris buildup (and thus corrosion and deterioration) in plate girders – such as bottom flanges, diaphragms, and bracing members – are protected in tub girders. Furthermore, reduced numbers of diaphragms result in reduced painting costs. Tub girders are also easier to inspect, since much inspection is performed from inside the tub, which serves as a protected walkway. However, it should be noted that bridges with only one or two tub girders (such as direct connector ramp structures) may be considered fracture critical, requiring additional inspection.

ECONOMY

Steel tub girders are more costly to fabricate than plate girders. Their geometry is complex, making it difficult to fabricate and assemble individual pieces in the shop and at bridge sites. It takes highly skilled workers to fabricate and erect steel tub girders, resulting in a premium on labor costs even if material costs are competitive with plate girder alternates. Individual tub girders are also bigger and heavier than plate girders, increasing equipment costs for erection.

Figure 5: Simple lines, smooth surfaces, and fewer parts visible; the clean appearance of tub girders offers distinct aesthetic advantages.

However, well-conceived and carefully detailed steel tub girders offer some economic advantages over plate girders. The need for fewer diaphragms reduces fabrication costs. Furthermore, fewer

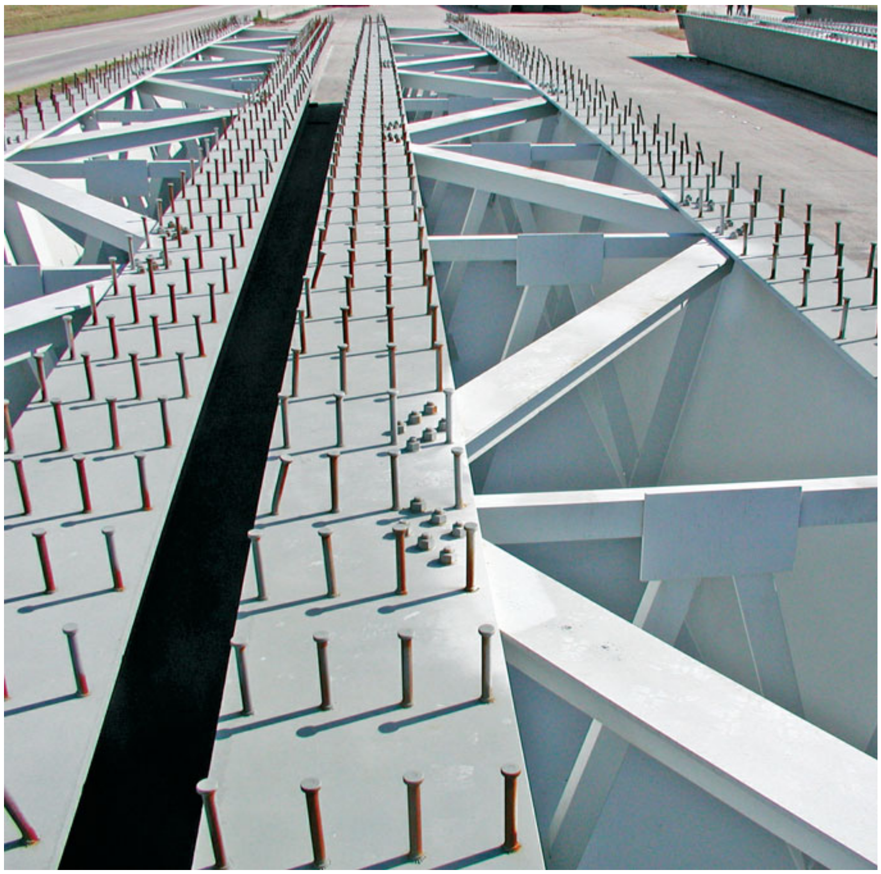
Figure 6 (near right): Modern highway construction involves working around many constraints; steel tub girders can help solve tough problems.

diaphragms and fewer girders versus plate girder alternates can speed erection, a critical advantage in urban highway reconstruction projects where traffic interruptions (and associated contractor “lane rental” penalties) dramatically affect construction costs and pose economic impacts to the community.

While steel tub girders are not inexpensive, when efficiently designed, carefully detailed, and used in the right application, they represent a reasonably economical, aesthetically pleasing solution that will compete favorably with other superstructure types.

APPLICATION ISSUES REFERENCES

There are several good references that discuss steel tub girder application issues. The article “Why Steel Box Girders?” (2) offers a concise overview of many key issues. Both “Section 12, Beam and Girder Bridges,” of the *Structural Steel Designer’s Handbook* (1) and “Section 21, Curved Steel Box Girder Bridges,” of the *Structural Engineer’s Handbook* (3) touch on some of these issues as well. The introduction to Volume II, Chapter 7, “Composite Box Girder Load Factor Design,” of the United States Steel (USS) *Highway Structures Design Handbook* (4), also provides a succinct discussion of many key issues.



As an upper bound, there are no hard and fast rules...

DEPTH-TO-SPAN RATIOS

The traditional rule of thumb for steel bridge girder depths of L/25 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* (5). 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 feet, as cited by several writers (2, 6). Note that steel tub girder depth affects more than just the 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 (as will be discussed later).

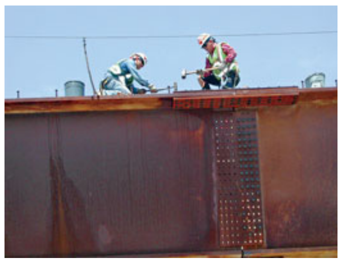
TUB GIRDER WIDTH AND SPACING

As a lower bound, at least one writer has suggested a minimum width of 4 feet be maintained for tub girders (6) 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 also on tub girder spacing) are often controlled indirectly by three central issues:

- Bottom Flange Width – Extremely wide bottom flanges will likely require the addition of longitudinal stiffeners and/or transverse stiffeners for stability (see further discussion below).
- Deck Transverse Span – 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 feet present transportation difficulties; widths above 14 feet may require provisions for a longitudinal bottom flange splice so that the tub can be transported to the bridge site in two sections.

Figure 7: Maintain a minimum width of 4 feet for tub girders to allow room for workers to complete fabrication of various internal details.

The ratio of the width of an individual tub girder to the spacing between tub girders should also be considered. Article 10.39.1.1 of the AASHTO *Standard Specifications for Highway Bridges* (7) states: “The average distance center to center of flanges of adjacent boxes shall not be greater than 1.2 times and not less than 0.8 times the distance center to center of the flanges of each box.” However, note that the main purpose of this limit is to keep the width/spacing ratio

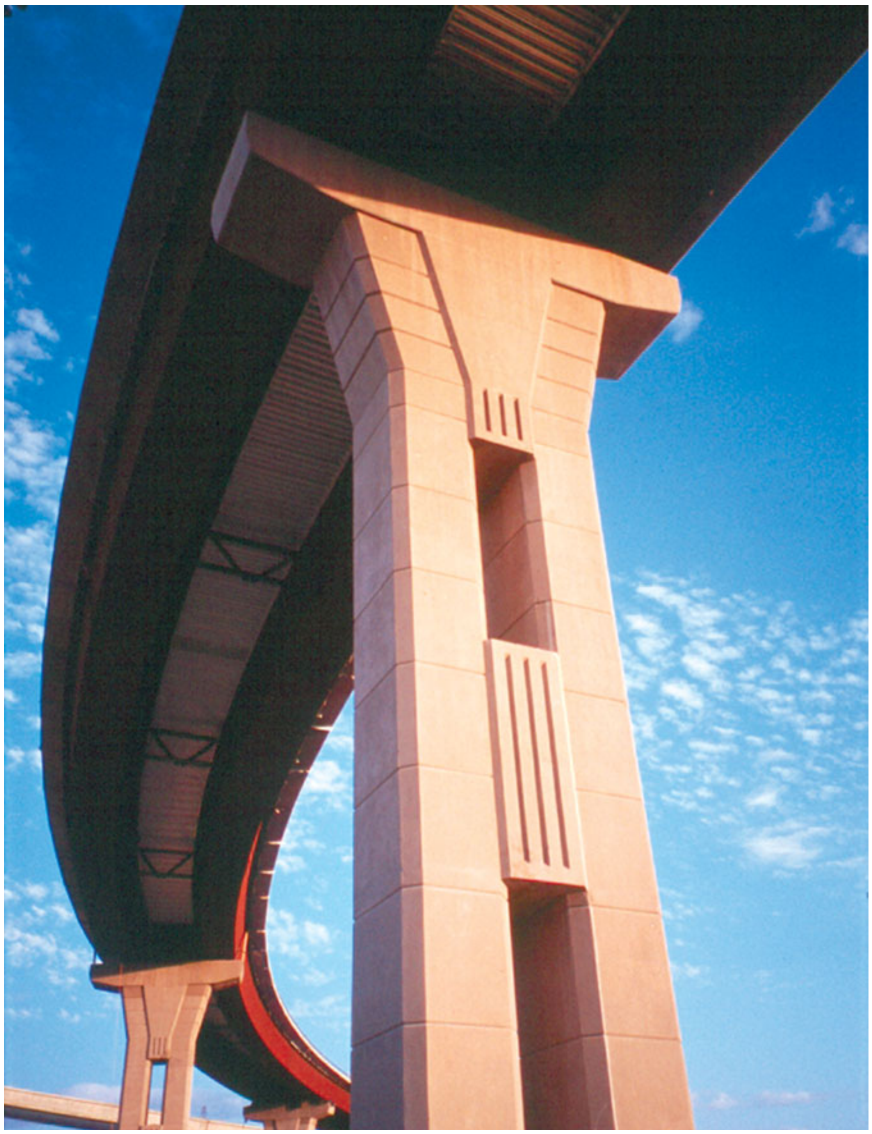


Overall width of a bridge is also a controlling factor in setting individual tub girder widths and spacings...

within the bounds of applicability for the empirical live load distribution factors provided in the AASHTO *Standard Specifications for Highway Bridges* (7). Exceeding these limits may be acceptable, provided a more refined analysis is performed that more rigorously quantifies live load distribution. Designers should take care to consider all the implications if they choose to exceed these limits.

Overall width of a bridge is also 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 feet 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 (see bearing discussions later).

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



While some simple rules of thumb exist to help guide the designer through the preliminary design phase, each steel tub girder bridge is unique...

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 in the end. Before beginning to run numbers, designers are advised to consider numerous framing plan issues discussed throughout this book.

While some simple rules of thumb exist to help guide the designer through the preliminary design phase, each steel tub girder bridge is unique. Additionally, because so many design, detailing and construction issues overlap, no hard and fast laws always govern. The most reliable rule is to take time during the preliminary layout and design phase to identify and evaluate these issues.

In this section, the tools and techniques for overall tub girder superstructure design are discussed. However, it must be noted that virtually all components of tub girder bridges require design. Some more sophisticated computer analysis tools (discussed below) can calculate many or all loads needed, but no program will perform all necessary design checks and many require supplemental hand analysis to quantify loads in secondary members.

Because tub girders are often used in curved bridges, much of the discussion of tub girder design is presented in a curved girder context. Furthermore, even tangent tub girders are subject to torsional loading and thus deserve many of the same considerations as for curved tub girders. However, discussion of tangent tub girder analysis is worthwhile both for direct applications to tangent tub girder bridges and for use of tangent girder design tools as part of approximate preliminary design.

TANGENT TUB GIRDER DESIGN TECHNIQUES AND TOOLS

Straight bridges containing multiple tangent tub girders can be designed using a single girder model. Dead loads can be analyzed assuming tributary load distribution, and live loads analyzed with appropriate live load distribution factors. Tub girder live load distribution factors were developed by Mattock in the early 1970s (8) and are still included in the current AASHTO *Standard Specifications for Highway Bridges* (7) and AASHTO *LRFD Bridge Design Specifications* (9). Sennah and Kennedy (10) discuss recently completed research into live load distribution factors for tub girders, including recommended distribution formulas, but these have not been incorporated into the AASHTO design specifications. Designers are advised to always verify the applicability of any code or research recommendations by checking the scope and assumptions of the background research.

Design can be facilitated using any of several commonly available tangent girder programs, some of which are specifically developed for tub girder design. Others are normally only applicable to plate girder or rolled beam design, but can provide sufficient design information to shortcut some tedious analysis steps. See Appendices A.1 and A.2 for further information.

For shear design, designers must remember that all tub girders carry torsion, which increases effective design shear in one web. In most straight bridges, web shears due to torsion are significantly smaller than those due to bending. Torsional analysis may be warranted for fascia girders and for girders at construction phase lines. 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.

Designers must remember that tangent girder programs do not account for load shifting and torsion effects caused by horizontal curvature...

APPROXIMATE/PRELIMINARY CURVED GIRDER DESIGN

According to the AASHTO *Guide Specifications for Horizontally Curved Steel Girder Highway Bridges* (5), the effects of curvature on primary bending loads can be neglected for slightly curved bridges (see the code for the exact definitions and limits). That is, the "load shifting" phenomenon in curved girder bridges (described below) can be neglected when curvature is slight. However, in all cases torsional effects should be examined. In many cases, these torsional effects only influence the design of the diaphragms and bracing.

In cases where the effects of curvature on primary bending loads can be safely neglected, the straight girder methods previously mentioned in this book should produce a design very close to the final design where torsion is considered, and the resulting cross-section sizes should be good candidates for the trial member sizes in the final design model. When analyzing a curved bridge using tangent girder methods, developed span lengths of the tub girders should be used. Live load distribution factors can be found in the commentary to the ASD provisions of the previous edition of the AASHTO *Guide Specifications for Horizontally Curved Highway Bridges* (11). Impact factors for steel tub girders are found in the current AASHTO *Guide Specifications for Horizontally Curved Steel Girder Highway Bridges* (5).

In cases where the effects of curvature on primary bending loads cannot be neglected, approximate analysis can still be performed using straight girder methods as the basis, but with the straight girder results amplified with factors to account for curvature as described below.

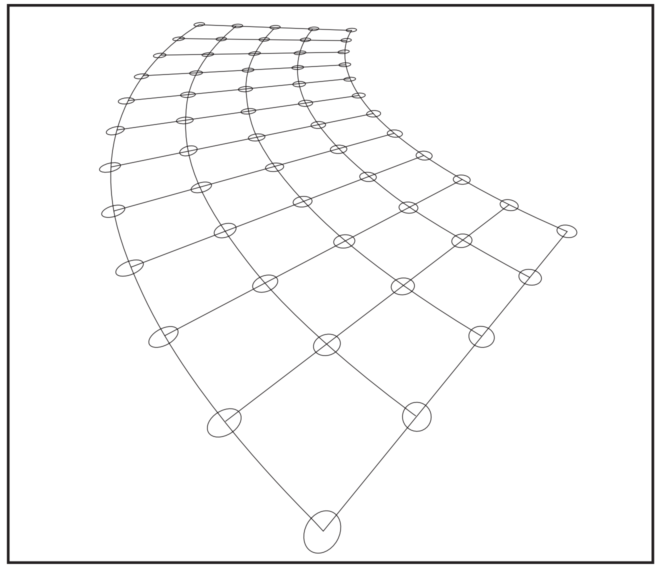
EFFECTS OF CURVATURE, M/R METHOD

Designers must remember that tangent girder programs do not account for load shifting and torsion 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, in the case of self load of an individual girder or the case of deck placement if 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 (3, 12, 13).

When girders are connected either by diaphragms or by a hardened deck, this effect results in a tendency for load shifting to the outside girders of the curve, due to the global overturning moment present in curved spans; that is, girders on the outside of the curve carry more load than girders on the inside of the curve. For preliminary design of curved tub girders, this effect can be approximated using guidance in the Federal Highway Administration (FHWA) "Curved Girder Workshop Lecture Notes" (12), which presents graphs for quantifying increases in design vertical moments due to curvature.

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 can also 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



In a grid analysis, a two-dimensional grid is used to define the overall geometry, with line elements used to model the girders and diaphragms...

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. Appendices A.3 and A.4 provide information on two of these programs. 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.

Because girders and diaphragms are modeled by line elements, the elements should be able to account for all related deformations, including bending, shear, torsion, axial deformation and cross-sectional distortion. Numerous elements have been developed, but commercial programs currently available in the market do not cover all types of deformation. Engineers should ensure that the program employed is at least able to model the bending, shear and pure torsion correctly, since warping stresses are usually small in properly braced box girders. Caution should be exercised in modeling trusses (such as external K-frames) as equivalent line elements. Additional calculations are required after the analysis to account for such items as brace forces and lateral bending stresses.

One major difficulty in grid analysis is modeling the concrete slab under live loads. Some programs and research investigations use line elements – aligned transversely like diaphragms – to model slab action. Be aware that validation of this simplification is not available from research.

For preliminary design, using these tools may be a more involved undertaking than performing a tangent girder analysis, because much more input is required to fully define a framing plan. However, there is a definite benefit to avoiding performing manual approximations to address curvature and/or perform design checks specific to tub girders. Note that at the preliminary design phase, it is both difficult and not valuable to perform detailed sizing of diaphragms and lateral bracing members beyond perhaps cursory evaluations to verify the soundness of a proposed framing plan.

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 either

Comparisons between field measurements and analysis – including 3-D FEA – continue to show disparity, because many aspects of tub girders are difficult to accurately model...

using plate/shell elements or solid brick elements. Bracing members are also 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 methods. However, implementation of 3-D FEA into common engineering practice still has many challenges. Comparisons between field measurements and analysis – including 3-D FEA – continue to show disparity, because many aspects of tub girders are difficult to accurately model. Because 3-D FEA is still a simplification of actual structure and load history, the necessity of detailed 3-D FEA is disputable if only marginal improvement is achieved, while the investment of computer and human resources is increased significantly. Also, some engineers still prefer to perceive tub girder bridges as beams and grids rather than a special assembly of plates and blocks. Finally, 3-D FEA generally requires additional efforts to convert stress results back to member forces (such as moments and shears) as are used in design.

This method, however, remains an effective tool in research and in studying bridge behavior under isolated load cases. If interpreted properly, 3-D FEA results can contribute to a better understanding of curved bridge behavior. Three-dimensional analysis programs or services exist that include code-checking procedures for main girders (see Appendix A.5 for information). 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* (5).

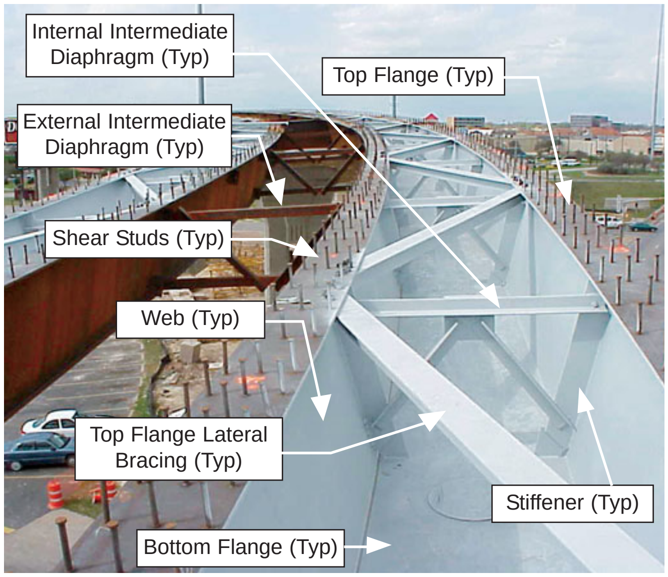


Figure 8 (near right): The various parts of a typical tub girder.

The girder design process for tub girders should begin with development of a framing plan. 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 (1, 3, 4, 5, 7, 14, 15).

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* (7) 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 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 a costly addition 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 the *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

Figure 9 (near right): Webs for tub girders carry both vertical shear and shear due to torsion, and are often horizontally curved, sloped and cambered.

than 1-inch thick to determine whether practical stiffness needs are met. In no case should bottom tension flanges be less than 1/2-inch 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 the *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 are also discussed in the Commentary to Section 10.4.2.4 of the AASHTO *Guide Specification for Horizontally Curved Steel Girder Highway Bridges* (5). 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 formulation $q = T/2A$, 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 early on 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

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...

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 are often 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 to Article 9.7.2.4 of the AASHTO *LRFD Bridge Design Specifications* (9) 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 behavior. 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.

Occasionally, project requirements dictate the need to use dapped girder ends. These complicate design, detailing and fabrication of the girders and their use should not be undertaken lightly. Reference 27 has current guidance on the design of dapped girder ends for both steel plate and steel 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 are also subjected to significant lateral bending stresses. These lateral bending stresses can be generated by horizontal girder curvature (4), sloping webs (18), and temporary supporting brackets for slab overhangs (5). 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 (5) 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 is also 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 carefully followed.

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 are typically provided with shear studs along the entire girder length to achieve the desired torsional rigidity from a closed cross-section.

In addition, top flanges are also 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.

Internal intermediate diaphragms are provided in tub girders to control cross-sectional distortion, which introduces additional stresses in box girders and should be minimized...

GENERAL GUIDELINES

Section 10.2 (and the associated Commentary) in the AASHTO *Guide Specifications for Horizontally Curved Steel Girder Highway Bridges* (5) offers discussion of framing plan issues such as diaphragm and top flange lateral bracing configurations, spacing and proportioning. Other good discussions can be found in the Bethlehem Steel *Designer's Guide to Box Girder Bridges* (19) and the USS *Steel / Concrete Composite Box-Girder Bridges – A Construction Manual* (20). More current sources for guidance include articles by Fan and Helwig (18, 21); although the articles focus more on the analytical aspects of determining diaphragm and bracing forces, they also provide insights valuable for establishing effective, efficient framing plans. "Section 21, Curved Steel Box Girder Bridges," of the *Structural Engineering Handbook* (3) also touches on some of these issues.

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 along the circumference 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.

Figure 10: Connection of K-frame diagonals to top chord. Tub girders have many parts and pieces; try to keep the details simple with clean load paths

Figure 11: Stiffeners to which bracing members are attached should be connected to girder flanges to reduce local distortion. Shown is a typical detail for connection to the bottom flange.

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 these 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

Internal cross frames also act as braces to prevent girder top flanges from buckling...

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* (5) has 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 (21). Strength verification should be conducted on diaphragms with controlling (largest) brace forces, usually located in the largest bending moment regions. The above-mentioned guides (3, 5, 19, 20, 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 will likely continue to be the primary approach in the 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 the 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 are typically 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 (25) will provide more insight into this behavior; publication is expected by summer 2005. 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

Figure 12 (near right): External intermediate diaphragms control differential displacement and rotation of tub girders during slab placement.

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 on curved girders without external intermediate diaphragms have occurred.

Many previous and current research investigations report that external intermediate diaphragm forces are small, and concluded that they are not needed. Yet, girder rotations in curved bridges are frequently observed. 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 is also 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

Pier and end diaphragms are generally full-depth plate girder sections...

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

One related question is whether (and how) to include external intermediate diaphragms in analysis models. External intermediate diaphragms are typically installed after girders are erected, so they do not carry load due to girder self-weight. Once the deck is hardened, it serves as a more effective load path between girders for distributing live loads; consequently, external intermediate diaphragms carry little or no load due to live load. Care should be taken to properly address these issues in tub girder analysis models.

INTERNAL DIAPHRAGMS AT SUPPORTS

Pier and end diaphragms are generally 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 along the circumference of the diaphragms, due to the torsional moment reactions on box girders.

Figure 13: Internal diaphragms at supports generally use full-depth solid webs, with access manholes between the bearing stiffeners.

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 can also 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. 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 is often conservatively neglected. Bearing design is discussed in more detail later.

Large torsional reactions may be needed at the girder support points, which results in the use of solid plate girders for the diaphragms in many curved bridges...

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 should be theoretically stress free. If single-point support is used, however, torsional moments must be resisted by external diaphragms that bend vertically.

Figure 14: Full-depth plate girder sections are typically used for external end diaphragms. Access holes are sometimes required to facilitate field connection, but they were not used for this particular bridge.

Large torsional reactions may be needed at the end of the girder support points, which results in the use of solid plate girders for the diaphragms in many curved bridges. With only one point support under each girder, the internal end diaphragms in adjacent girders combine with the external diaphragms to form a beam to resist the torsional moments from individual girders (Fig. 15). Because the total torque may be resisted by differential reactions at the bearings of different girders, the diaphragms are subjected to both bending and shear, as shown in Fig. 15. If the torsional reaction on each of the twin girders is T, the moment and shear on the diaphragm can be derived from statics as M = Tb/L and V = 2T/L, where L is the girder spacing and b is the average length of the diaphragm. More complex systems, including multiple girders and unequal torsional reactions, can be solved similarly.

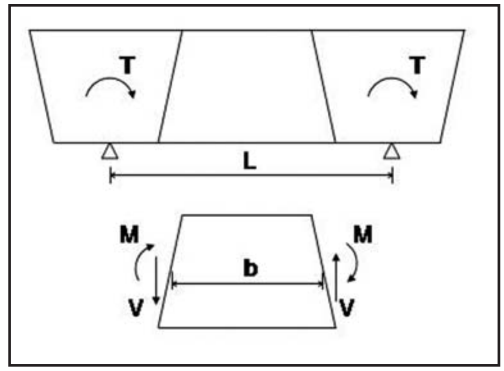


Figure 15: Freebody diagrams for calculating diaphragm forces.

The above procedure would result in a bending moment at the end of diaphragms (M) smaller than the torque on the girder (T). However, the resultant reaction may not be at the center of the girder width, due to rotation of the bearings. A larger moment may result at one end of the diaphragm than at the other. A conservative approach is to simply assume the moments at the diaphragm ends to be equal to the torsional reaction of the box girder (M = T), while shears are still calculated using the above method. These moment and shear values are important to the design of the plate girder diaphragms, and to the design of the connections between the diaphragms and the girders.

Large moments at diaphragms often require the use of a moment connection to the girder (diaphragm flanges are also

Figure 16: At supports, full-depth plate girder sections are often used for external diaphragms.

Top flange lateral bracing is required in tub girders to form the “fourth side” of the box until the slab is in place...

connected to the girders); the connection may be designed in a similar manner to the splice design for the girders. Torsional reactions at girder supports should be available from analysis results. The largest torsion is very likely to occur during the construction stage. Torsional moments in straight girders are significantly smaller, and a plate girder may not be warranted for external diaphragms.

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 potentially be detailed as pairs of right (nonskewed) diaphragms to work around the problems of skewed diaphragms.

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 (3, 5, 18, 19, 20) offer good discussions on sizing and spacing issues. This bracing is typically 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 is also 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, α, is an important design parameter. Ideally, an α around 45 degrees is desired. A small α leads to larger cross frame spacing and fewer bracing panels and connections. However, larger brace forces and girder stresses result from a small α. 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 (guidelines for the spacing of the internal cross frames were discussed earlier). For narrow girders, these guidelines may suggest a cross frame spacing much larger than the girder width, which would result in a small α. Therefore, the location of the internal diaphragms should be planned with consideration of lateral bracing.

It is suggested to keep the plane of the top flange lateral bracing reasonably close to the plane of the top flanges...

An alterative 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 labor requirements. 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 the girder crosssections, 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 the 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 have to 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

The effects of transverse loading on the weld because of web slope and lateral bracing loads should be included...

FLANGE-TO-WEB WELDS AND SHEAR STUDS

Flange-to-web welds are designed in a similar manner as for plate girders, but there are additional loading effects exclusive to tub girders that must be addressed. The shear contributing to longitudinal load in the weld (calculated as VQ/I) should include both vertical shear (resolved to account for web slope) and shear due to torsion. In addition, the effects of transverse loading on the weld because of web slope and lateral bracing loads should be included. Shear studs are designed in a similar manner as for plate girders, but there are additional loading effects exclusive to tub girders that must be addressed. The shear contributing longitudinal load in the studs (calculated as VQ/I) should include both vertical shear (resolved to account for web slope) and shear due to torsion. Shear studs must be provided along the total girder length to ensure that the slab is acting as the fourth side of a box girder.

BEARINGS

An important decision to make about bearings is how many to use: one or two per support. As with most decisions, the choices offered each have pros and cons. The number of bearings per support can change from support to support, allowing designers to optimize the design.

Using two bearings allows girder torsion be directly removed through the force couple provided by the bearings, and reduces reaction demand on the bearings. Two-bearing systems work well with radial supports, but are impractical with supports skewed more than a few degrees, where tub girder and/or diaphragm stiffness work against uniform bearing contact during various stages of girder erection and slab construction. One way to try to ensure proper contact in two-bearing systems is to build the structure on temporary supports and then grout the bearings in place prior to shoring removal.

Using one bearing per support optimizes contact between girder and bearing. One-bearing systems also are more forgiving of construction tolerances and, for skewed supports, one-bearing systems are demonstrably better than two-bearing systems. The drawback to one-bearing systems is that stiff cross frames or diaphragms between girders are required to resolve girder torsion into the bearings. Fabricating and erecting these stiff elements is demanding, and bearing installation can be difficult with very wide bottom flanges.

What type of bearing to employ is as important as deciding how many bearings per support will be used. Steel-reinforced neoprene and pot bearings are the most commonly encountered bearings with tub girders, although disc bearings are occasionally used. Steel-reinforced neoprene bearings generally cost less than pot bearings in smaller sizes, are forgiving of construction tolerances, and are easily

The decision to use an empirical deck design with tub girders must not be taken lightly...

inspected. However, steel-reinforced neoprene bearings usually have less reaction capacity than pot or disc bearings. The upper reaction limit with neoprene bearings is about 1,000 kips (22); with reactions much greater than this, pot or disc bearings become a more practical choice. Note that pot bearings also have a lower limit on reactions (22) and may not be an appropriate choice when low reactions are expected.

Figure 17: Steel-reinforced neoprene bearings are a simple, low-maintenance bearing option.

Girder translation is readily accommodated with steel-reinforced neoprene bearings. In cases where the amount of translation creates tall, unstable pads, a stainless steel/polytetrafluoroethylene (PTFE) sliding surface can be introduced. Pot bearings always need a stainless steel/PTFE sliding surface to accommodate translations.

Regardless what type is selected, designers should ensure bearings can be replaced with limited jacking.

Both the current AASHTO *Standard Specifications for Highway Bridges* (7) and AASHTO *LRFD Bridge Design Specifications* (9) include general formulas for designing reinforced neoprene bearings. For specific guidance on designing steel-reinforced neoprene bearings for tub girders, see the article “Elastomeric Bearings for Steel Trapezoidal Box Girder Bridges” (23). For an overview of most bearing types used with steel bridges, see the National Steel Bridge Alliance’s (NSBA’s) “Steel Bridge Bearing Selection and Design Guide” (22).

BRIDGE DECKS

Concrete deck slabs on most tub girders are cast in place with extensive use of stay-in-place metal deck forms. If designers elect to use the empirical design method in the AASHTO *LRFD Bridge Design Specifications* (9), Section 9.7.2 for the bridge deck, intermediate diaphragms between tub girders are required at 25-foot maximum spacing. Such diaphragms partially negate the reasons to use tub girders in the first place. In lieu of this requirement, the LRFD Specifications, Section 9.7.2.4 (9), permits designers to investigate if additional slab reinforcement over the girder webs is necessary for transverse slab bending between girders. A bridge deck could be modeled with both tub girders and I-shaped girders to ascertain differences in transverse deck bending moments. The moment difference between the two girder systems could be accommodated with supplemental reinforcement.

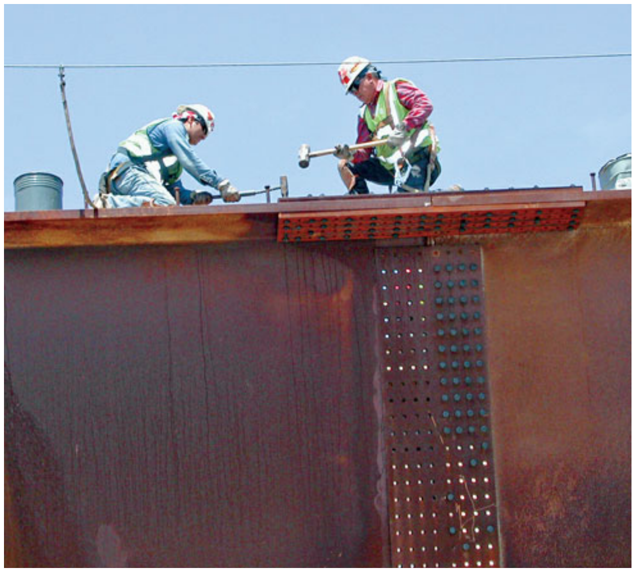
The decision to use an empirical deck design with tub girders must not be taken lightly. Designers should carefully weigh their perceived advantages against their many limitations.

FIELD SPLICES

Just as with plate girders, tub girders usually need field splices to facilitate girder transportation and erection. Maximum allowable shipping lengths of about 120 feet are common throughout much of the U.S., but some states are more restrictive. Designers must be cognizant of oversize/overweight permit requirements imposed by the state the bridge is located in. Sometimes the fabricator is not in the same state as the bridge and designers do not know which fabricator will be building and shipping the girders, adding a level of uncertainty in locating field splices for shipping purposes.

Overall girder width of tub girders (including sweep for curved girders) should be considered for shipping purposes. Though not likely, shipping costs of very wide girders

Figure 18 (near right): Bolted field splices are generally quicker to complete than welded splices, but may prove more prone to corrosion.



(in excess of 14 feet) can outweigh costs of adding additional field splices. In rare cases, tub girders are so wide they must have longitudinal field splices in bottom flanges.

Weight can be another consideration in locating field splices. Tub girders are heavier than plate girders and weight can become excessive for economical shipping and erection. When determining field splice location, consultation with several regional fabricators familiar with tub girders is recommended. For field splices provided only for transportation issues, designers should be clear in their plans that these splices can be completed prior to final girder erection.

As with plate girders, field splices can be bolted or welded. Welded field splices with tub girders are difficult – even for highly trained welders experienced with field welding bridge girder splices – and time consuming. The longer time required to complete welded splices can result in high costs from traffic control if the splice is over traffic. Fitup tolerances prior to welding are in some ways more restrictive than with bolted connections. Bolted field splices predominate throughout much of the United States for tub girders. Although not as aesthetically appealing as welded splices, they are quicker to complete with less skilled labor. The current AASHTO *Guide Specifications for Horizontally Curved Steel Girder Highway Bridges* (5) has a comprehensive example to guide designers through the bolted splice design.

However, some erectors maintain that welded splices are more forgiving with regard to fit-up tolerances, given skilled welders. Some owners suggest welded splices may be more durable, due to observed corrosion in some bolted splices. The decision to use welded field splices should be undertaken with care and full consideration of all issues.

While painted faying surfaces may be more durable, allowing painted faying surfaces can create confusion, since two types of paint are usually encountered with tub girders. Exterior paint system primers are usually different from interior paint. Designers must be clear on which, if any, paint is allowed in bolted faying surfaces and what slip coefficient the design is based on.

DETAILING

For detailing guidelines, the reader is directed to *www.steelbridge.org*, the AASHTO/NSBA Steel Bridge Collaboration's Web site. One of the Collaboration's documents is helpful in outlining detailing requirements specific to tub girders: G1.4, "Design Details Sample Drawings," which is currently (May 2003) in draft form; a final version should be available in the near future.

Also, Appendix B of this book contains guidance on detailing steel tub girders, including a set of suggested details. These details have been reviewed by a wide range of engineers, detailers, fabricators and erectors, and represent the best consensus on good detailing practice for steel tub girders. Of course, each tub girder is unique

Placing wire mesh over any copes or clips in end plates and bottom flange drain holes are two more suggested details peculiar to tub girders...

Figure 19: Cost versus benefit: Full shop fit-up was required on this tub girder project to help ensure erection would go smoothly. Requiring full bridge assembly in the shop may avoid some fit-up problems in the field, but can be quite costly. Designers are advised to explore all consequences of their decisions.

and may have design or detailing requirements which differ from these suggested details, but these details represent a broadly applicable set of “recommended practices.”

Figure 20: Steel tub girders, top flanges and stiffeners are typically assembled as a sub-unit upside down, then flipped over and welded to the bottom flange.

This Web site also offers two more good references in detailing tub girders: the Texas Steel Quality Council’s “Preferred Practices for Steel Bridge Design, Fabrication, and Erection” (16) and Mid-Atlantic States Structural Committee for Economic Fabrication (SCEF) Standards (24). Some important detailing issues bear repeating:

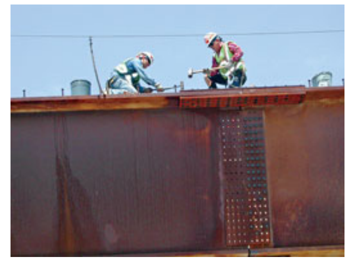
- Both webs should be equal in height and the girders should be rotated about the point defined by the intersection of the girder’s centerline and the top of the webs.
- Bottom flanges must extend at least 1½ inches beyond web centerlines for welding purposes.
- Provide details consistent with future inspection needs, such as adequately proportioned access holes, light access doors, electrical service along the girders, and painting interiors a light color.
- Connection details for external cross frame or diaphragm details must be prepared in consideration of the tremendous lateral and torsional stiffness of tub girders. Tub girders cannot be manipulated as easily as plate girders during erection.

Placing wire mesh over any copes or clips in end plates and bottom flange drain holes are two more suggested details peculiar to tub girders. Wire mesh, which prevents girder interiors from becoming wildlife habitat, should be 10 gage to withstand welding and blasting and have a tight weave, about ½ inch by ½ inch. Suggested bottom flange drain holes are 1 ½ inches in diameter and spaced along the bottom flange’s low side every 50 feet. Holes should be placed 4 inches away from the web plate. Place a ½ inch bead of adhesive caulk to direct water to the drain hole. These holes will mitigate rainwater buildup in tub girders prior to deck placement and provide helpful ventilation for the life of the girder. See Fig. 22 for suggested drain hole details.

MATERIAL SELECTION

Tub girders offer excellent opportunities to use High Performance Steel (HPS) A709 Grade HPS- 70W. Like its use in plate girders, HPS will be most economical in hybrid applications. Negative and positive moment tension flanges

Figure 21 (near right): Tub girder interior surfaces should always be coated with a white or light colored paint.



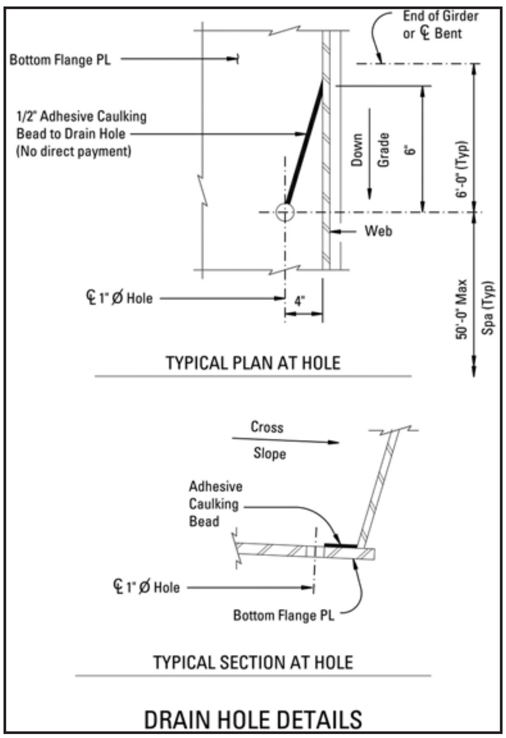


Figure 22: Suggested details for drain holes in tub girder bottom flanges.

show the greatest potential in utilizing HPS. Recent studies such as found in Reference 28 discuss this in more detail. It should be noted that the current AASHTO *Guide Specifications for Horizontally Curved Steel Girder Highway Bridges* (5) does not provide for hybrid girders, due to lack of related research. However, upcoming revisions to the AASHTO "LRFD Bridge Design Specifications" (see Reference 26) do include provisions for hybrid girders.

In addition to HPS, A709 Grades 50 or 50W are the next steel of choice for tub girders. It is hard to justify using Grade 36 steel for economics; Grade 100 steels will likely see economical applications in bridges with unusually long span lengths.

As with all steel bridges, weathering steel is the material of choice unless site conditions or aesthetic requirements preclude its use. To optimize weathering steel aesthetics, fabricators should be required to perform an SSPC-SP 10 level blast cleaning of fascia web surfaces (and bottom flanges and other webs if they will be visible to the public) prior to shipping girders. It is easier–and therefore less costly–to do this at the fabricator's shop than in the field.

PAINTING

Considerations for painting the exteriors of tub girders are the same as for plate girders. Tub girder interiors should always be coated. Without owner direction towards a specific coating and preparation, girder interiors should receive a light brush blast and be painted with a white or light colored paint capable of telegraphing cracks (which aids bridge inspection). Specified interior paint should be tolerant of minimal surface preparation. In most cases, interior paint is provided not for corrosion protection but for girder inspection. As such, localized paint failure can be tolerated. Specifying stringent requirements for tub girder interior paints and surface preparation must not be taken lightly, because they will add significant costs to projects. Note that the painted interior surfaces do not necessarily need to include the top flange lateral bracing members.

An interesting finding from these projects was that lateral brace forces arising from slab placement were sensitive to where concrete pours were initiated and their subsequent direction...

IH 35/US 290 INTERCHANGE, AUSTIN, TEXAS

Tub girders were selected for all the direct connection structures in the IH 35/US 290 interchange in Austin. Approach spans used precast, prestressed concrete Texas U-beams; the longer span and/or curved portions used steel tub girders.

Figure 23: The simplicity of steel-reinforced neoprene bearings illustrated in an expansion bearing at the end of an 880-foot continuous tub girder unit

TxDOT funded two research projects related to steel tub girders prior to the construction of this interchange. One project, under the direction of Karl Frank, Ph.D., covered the measurement of forces in tub girders at different stages of construction. The other project dealt with simplifying costly details associated with tub girders and was directed by Joseph Yura, Ph.D. An interesting finding from these projects was that lateral brace forces arising from slab placement were sensitive to where concrete pours were initiated and their subsequent direction. To illustrate: if an end span pour began close to midspan and progressed toward the span end, maximum lateral brace forces were close to calculated values. If end span pours began at the span's end and progressed into the span, maximum lateral brace forces were significantly less than predicted. This phenomenon was attributed to the stiffness gain rate of freshly placed concrete.

Steel-reinforced neoprene bearings were employed throughout the interchange. One bearing per support was used in all cases. The screening instrument in reference (22) indicated that pot bearings were the best choice, since maximum reactions were over 1,000 kips; however, the designers elected to use neoprene bearings based on cost, ease of manufacturing and long-term maintenance. See Figs. 17 and 23 for the simplicity of an expansion bearing at the end of an 880-foot continuous girder unit. Fixed bearings were detailed with anchor bolts penetrating through the bearings and girder bottom flange. This turned out to be a poor detail, due to the reality of construction tolerances, and accommodations had to be made in several instances where anchor bolts did not align with their respective holes.

VARIOUS TXDOT HOUSTON DISTRICT PROJECTS, HOUSTON, TEXAS

Within the last 10 years, TxDOT's Houston District completed or has under construction dozens of steel tub girder bridges, totaling more than 110 million pounds of steel. About half of these projects were designed directly by TxDOT Houston District engineers. There is simply no substitute for steel structures when roadway geometry is complex, spans are unbalanced, and speed of construction is important.

In the Houston area, tub girders are selected primarily for aesthetic reasons, despite the cost premium. Steel I-girder bridges range in cost from $70 to $80 per square foot of deck area, while steel tub girders range in cost from $90 to $105 per square foot of deck area. The reasons for this approximately 25 percent cost premium for tub girders are complex and many, but several have been identified as the primary causes. Fabrication and erection costs are higher because the individual pieces are larger than for comparable I-girder bridges. Unbalanced spans are common in steel units these

Nearly all the field problems encountered to date can be traced to fundamental engineering errors not unique to tub girder design...

Figure 24: Some of the 110 million pounds of tub girder steel designed in the TxDOT Houston District.

days, but web depth is maintained throughout, so efficiency is sacrificed for aesthetic reasons. Another source of inefficiency is that tub girder spacing requirements force the designer to use one or more webs than is required for a comparable I-girder superstructure. Essentially, tub girder superstructures can tend to weigh more than equivalent I-girder superstructures due to additional conservatism on the part of engineers, since design and analysis procedures are much less well defined for tub girders than I-girders.

Nearly all the field problems encountered to date can be traced to fundamental engineering errors not unique to tub girder design. Over the last 10 years, the full range of analysis tools has been used to design tub girder bridges in the Houston area. Many problems have occurred during construction, especially when general-purpose analysis programs were used or when the superstructure was designed as individual girders and interaction with adjacent girders was not properly accounted for. Incorrect modeling of boundary conditions has also resulted in field problems especially with bearings and uplift at the support. Recent bridges designed with commercially available software, based on grid analysis, have performed well. Problems with secondary members have occurred but the frequency has diminished as more emphasis has been placed on accounting for sequence of loading and connection eccentricity. For example, intermediate external diaphragms are installed after the girders have deflected under self weight. This increases forces in the top lateral brace system and has resulted in bowing of braces during construction. While not perfect, grid analysis programs have proven to be a reliable and efficient design tool for tub girder systems.

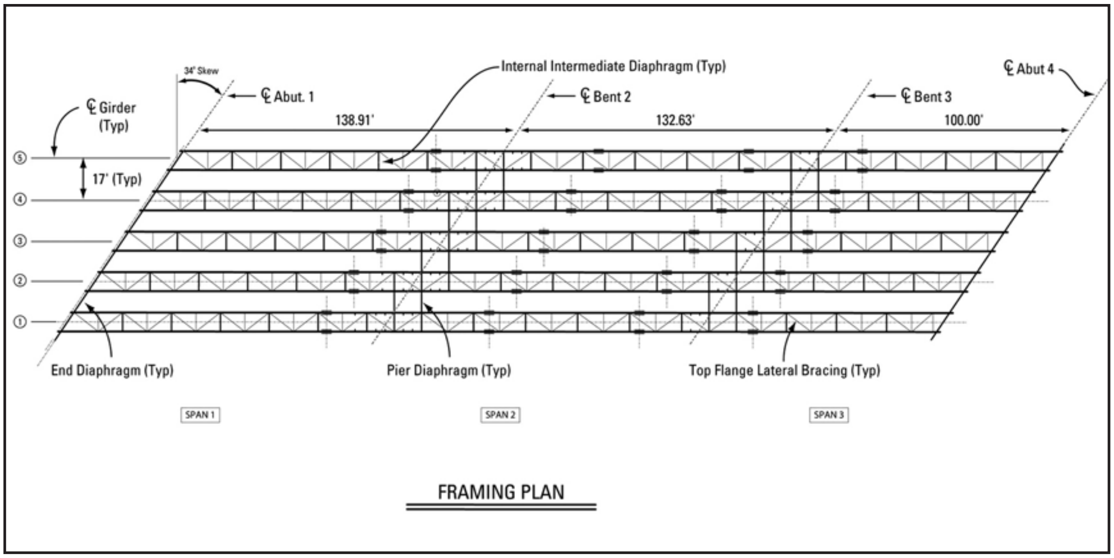
US 75 UNDERPASS AT CHURCHILL WAY, DALLAS, TEXAS

The US 75 Underpass at Churchill Way is a three-span continuous tangent steel tub girder bridge (138.91 feet – 132.63 feet – 100 feet), built on a 34 degree skew. This bridge might be described as a good example of several things to try to avoid in steel tub girder bridges.

First, the span arrangement is far from ideal, both in terms of its lack of good span balance and in the relative shortness of the spans. The span arrangement was dictated by existing site conditions, including the locations of abutment provisions in an existing retaining wall and existing lane configurations of the eight-lane expressway, frontage roads and ramps below the bridge. To blend the bridge's appearance with the strong existing aesthetic theme of the US 75 expressway, it was desired to provide a smooth soffit superstructure, thus eliminating steel I-girders as an option. Lane closure limitations and the given span lengths eliminated precast and cast-in-place concrete options, leaving steel tub girders as the most viable superstructure type.

Next, the 34-degree skew noticeably complicated the design and detailing of this bridge, as might be expected. A 3-D FEA analysis was performed (using the proprietary BSDI3-D System) to ensure skew effects were adequately quantified. The designers chose to use paired,

Figure 25 (near right): Framing plan for US 75 Underpass at Churchill Way; right diaphragms used at interior piers to simplify detailing for this bridge, which is built on a 34-degree skew.



staggered, right (non-skewed) pier diaphragms to simplify detailing and erection. At the end diaphragms, there was not much choice but to detail the diaphragms on the 34-degree skew; the designers consulted a steel detailing shop and a local fabricator during the design process to solicit input on preferred details for the skewed end diaphragms.

To simplify and streamline erection (a key project criterion was minimizing the duration of lane closures on the expressway below), the designers chose to omit any external intermediate diaphragms. As would be expected, this resulted in higher top flange lateral bracing loads, but the benefit of simpler, quicker erection was deemed of greater value. The designers also denoted two of the four field splices as “optional,” so that a properly equipped contractor could erect fewer, larger field sections and avoid some lane closures (and the associated “lane rental” costs stipulated in the construction contract).

Designers should also realize that in many cases, there are no hard and fast rules associated with these issues...

Steel tub girders are often 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. In addition, steel tub girders have distinct aesthetic advantages, due to their clean, simple appearance.

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 on one bridge might not necessarily work on 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.


NORTH
610

1. Hedefine, A., Swindlehurst, J., and Sen, M., “Section 12, Beam and Girder Bridges”, *Structural Steel Designer’s Handbook*, (Brockenbrough, R.L, and Merritt, F.S., Editors), 3rd Edition, 1999, McGraw Hill, Inc. Current edition can be ordered from McGraw-Hill publishers, *www.mcgraw-hill.com*, or call 1-877-833-5524.

2. Hall, D. H., “Why Steel Box Girders?” Modern Steel Construction, April 1997. Reprint may be downloaded from the website of the National Steel Bridge Alliance, *www.aisc.org*, click on the National Steel Bridge Alliance button, then click on “Bridge Crossing Articles.”

3. Poellot, W.N., “Section 21, Curved Steel Box Girder Bridges,” *Structural Engineering Handbook*, (Gaylord, E.H, Gaylord, C.N., and Stallmeyer, J.E., Editors), 4th Edition, 1997, McGraw-Hill, Inc. Current edition can be ordered from McGraw-Hill publishers, *www.mcgraw-hill.com*, or call 1-877-833- 5524.

4. AISC Marketing, Inc., *Highway Structures Design Handbook*, ADUSS 88-8535, 1981. This book has long been out of print. The NSBA offers a modern version (see references 15 and 21), but its examples for steel tub girders lack some of the introductory text of the older version; this older version can sometimes be found in office libraries or in the personal collections of experienced engineers.

5. American Association of State Highway Transportation Officials (AASHTO), *Guide Specifications for Horizontally Curved Steel Girder Highway Bridges*, GHC-4, 2003. Current edition can be ordered from AASHTO, *www.aashto.org* (click on “Bookstore”), or call 1-800-231-3475.

6. Kase, R.A., “Twelve Commandments for Economic Steel Box Girders,” Modern Steel Construction, August 1997. Reprint may be downloaded from the Web site of the National Steel Bridge Alliance, *www.aisc.org*, click on the National Steel Bridge Alliance button, then click on “Bridge Crossing Articles.”

7. American Association of State Highway Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges,* 17th Edition, HB-17, 2002. Current edition can be ordered from AASHTO, www.aashto.org (click on “Bookstore”), or call 1-800-231-3475.

8. Mattock, A.H., “*Development of Design Criteria for Composite Box Girder Bridges*,” Development in Bridge Design and Construction, Crosby Lockwood & Sons, London, England, 1971.

9. American Association of State Highway Transportation Officials (AASHTO), *LRFD Bridge Design Specifications*, 3rd Edition, LRFD-3, 2004. Current edition can be ordered from AASHTO, www.aashto.org (click on “Bookstore”), or call 1-800-231-3475.

10. Samaan, M., Sennah, K., and Kennedy, J.B., “Distribution of Wheel Loads on Continuous Steel Spread- Box Girder Bridges,” *ASCE Journal of Bridge Engineering*, Vol. 7, No. 3, May-June 2002, pp. 175-183. Copies may be obtained by contacting ASCE (www.asce.org, or call 1-800-548-2723) or can usually be found in local university libraries.

11. American Association of State Highway Transportation Officials (AASHTO), *Guide Specifications for Horizontally Curved Highway Bridges*, GHC-3, 1993 with Interim Revisions through 1995. This is the previous version of this specification, but it may still be available from AASHTO, check at AASHTO, *www.aashto.org* (click on “Bookstore”), or call 1-800-231-3475.

12. Richardson, Gordon, and Associates (presently the Pittsburgh, Pennsylvania, office of HDR), “Curved Girder Workshop Lecture Notes,” prepared under Contract No. DOT-FH-11-8815, Federal Highway Administration, 1976. This document has long been out of print and may be difficult to locate.

13. Tung, D.H.H, and Fountain, R.S., “Approximate Torsional Analysis of Curved Box Girders by the M/R Method,” *AISC Engineering Journal*, Vol. 7, No. 3, July 1970. Reprint may be downloaded from AISC’s Web site at *www.aisc.org*. Click on the Engineering Journal button, then click on “Downloadable PDFs.”

14. American Iron and Steel Institute, “Four LRFD Design Examples of Steel Highway Bridges,” Vol. II, Chapters 1A and 1B, *Highway Structures Design Handbook*, 1996. Available from the National Steel Bridge Alliance, *www.aisc.org*, click on the “Bookstore” button.

15. American Iron and Steel Institute and National Steel Bridge Alliance, “Four LFD Design Examples of Steel Highway Bridges,” Vol. II, Chapter 2B, *Highway Structures Design Handbook*, 1999. Available from the National Steel Bridge Alliance, *www.aisc.org*, click on the “Bookstore” button.

16. Texas Steel Quality Council, *Preferred Practices for Steel Bridge Design, Fabrication, and Erection*, November 2000. Current edition can be downloaded from http://www.steelbridge.org, click on “Regional Groups” and find the document link in the Texas Quality Council section.

17. Wolchuk, R., and Mayrbaurl, R.M., “Proposed Design Specifications for Steel Box Girder Bridges,” Report No. FHWA-TS-80-205, Federal Highway Administration, January 1980. Copies can be purchased by contacting the National Technical Information Service at 1-800-553-6847.

18. Fan, Z., and Helwig, T.A., “Behavior of Steel Box Girders with Top Flange Bracing,” *ASCE Journal of Structural Engineering*, Vol. 125, No. 8, August, 1999, pp 829-837. Copies may be obtained by contacting ASCE (*www.asce.org*, or call 1-800-548-2723) or can usually be found in local university libraries.

19. Heins, C.P., and Hall, D.H., *Designer’s Guide to Steel Box Girder Bridges*, Booklet No. 3500, February, 1981, Bethlehem Steel. This guide has long been out of print. Copies may be obtained by directly contacting the National Steel Bridge Alliance technical staff (not the NBSA publications department).

20. United States Steel, *Steel / Composite Box-Girder Bridges–A Construction Manual*, ADUSS 88-7493- 01, December 1978. This guide has long been out of print. Copies may be obtained by directly contacting the National Steel Bridge Alliance technical staff (not the NBSA publications department).

21. Fan, Z., and Helwig, T.A., “Distortional Loads and Brace Forces in Steel Box Girders,” *ASCE Journal of Structural Engineering*, Vol. 128, No. 6, June, 2002, pp 710-718. Copies may be obtained by contacting ASCE (*www.asce.org*, or call 1-800-548-2723) or can usually be found in local university libraries.

22. American Iron and Steel Institute and National Steel Bridge Alliance, “Steel Bridge Bearing Selection and Design Guide,” Vol. II, Chapter 4, *Highway Structures Design Handbook*, 1996. Available from the National Steel Bridge Alliance, *www.aisc.org*, click on the “Bookstore” button.

23. Bradberry, T.E., Cotham, J.C., and Medlock, R.D., “Elastomeric Bearings for Steel Trapezoidal Box Girder Bridges,” *Proceedings of the ASCE Texas Section Meeting*, Waco, TX, October 2-5, 2002. Copies can be downloaded from the TxDOT Bridge Division website, *www.dot.state.tx.us/brg*, click on Division Publications.

24. Mid-Atlantic States Structural Committee for Economic Fabrication (SCEF) Standards, available on line at *www.steelbridge.org*, click on the “more” where it says “Mid-Atlantic States Structural Committee for Economic Fabrication (SCEF) Standards - more” on the home page.

25. Yura, J. A., Helwig, T. A., Herman, R. H., and Williamson, E.; "Trapezoidal Box Girders - State of the Art", Research Report 0-4307-1, Report for Texas Department of Transportation, anticipated publication date: May 2005. Contact the University of Texas at Austin, Department of Civil, Architectural, and Environmental Engineering (http://www.ce.utexas.edu/) or the University of Houston, Department of Civil and Environmental Engineering (http://www.egr.uh.edu/CIVE/).

26. American Association of State Highway Transportation Officials (AASHTO), LRFD Bridge Design Specifications, 2005 Interim Revisions to the 3rd Edition, anticipated publication date: May 2005. Contact AASHTO, www.aashto.org (click on "Bookstore"), or call 1-800-231-3475.

27. Fry, G. T., Bailey, B. M., Farr, J. L., Elliot, J. E., and Keating, P. B.; "Behavior and Design of Dapped Steel Plate Girders", Research Report 0-2102-1, Report for Texas Department of Transportation, anticipated publication date: May 2005. Contact Texas A&M University, Department of Civil Engineering (http://www.civil.tamu.edu).

28. Horton, R., Power, E., Azizinamini, A. and Krupicka, G., “High Performance Steel Cost Comparison Study - Box Girder,” Conference Proceedings, 2004 FHWA Steel Bridge Conference, San Antonio, Texas, Dec. 2004.

A.1 AISC SIMON

This DOS-based line girder program with tub girder design features is available from AISC (*www.aisc.org*, type “SIMON” into the search box, or call 312-670-2400).

A.2 VARIOUS TANGENT I-GIRDER PROGRAMS

STLBRIDGE is a Windows-based program available from Bridgesoft, Ltd. (*www.bridgesoftinc.com* or call 402-449-1581). It is one of many tangent I-girder design programs (both new and old, commercially sold and “in-house” developed programs) commonly available to bridge designers. Designers can use these programs to develop preliminary shear and moment diagrams and/or flange stresses. However, these programs are limited to I-shaped members (plate girders and rolled beams) and thus do not provide the ability to directly design all pieces of a tub girder. Generally, one half of the tub girder is input as the cross-section, and web slope is ignored.

A.3 DESCUS II

DESCUS II is a commercially sold program (*www.opti-mate.com*). DESCUS II is a DOS-based program (now with a Windows interface) that performs a grid analysis for curved tub girder design in accordance with AASHTO ASD or LFD (currently only per the previous edition of the AASHTO Guide Specifications for Curved Steel Girder Highway Bridges) specifications.

A.4 MDX

MDX is a commercially sold program (*www.mdxsoftware.com* or call 573-446-3221). MDX is a Windowsbased program that performs a grid analysis and curved tub girder design in accordance with the AASHTO ASD, LFD (currently only per the previous edition of the AASHTO Guide Specifications for Curved Steel Girder Highway Bridges), and LRFD specifications.

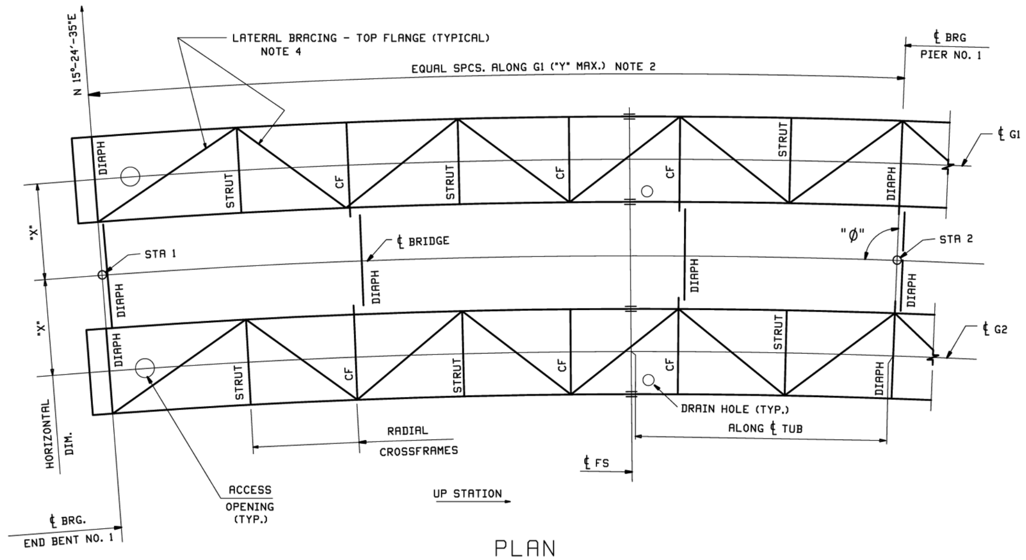
A.5 BSDI3-D SYSTEM

The BSDI3D System is a design service (BSDI Ltd., call 610-282-2888) that provides 3-D FEA of plate or tub girders, including code checking in accordance with the AASHTO ASD and LFD specifications. Forces are calculated for all members modeled, but code checks are only provided for webs and flanges.

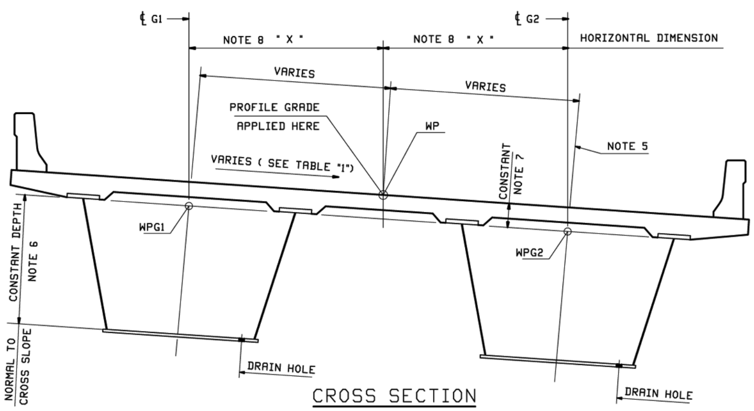
APPENDIX B - EXAMPLE DESIGN DETAILS

Appendix B consists of example steel tub girder detail drawings. These details represent a starting point that shows preferred detailing practice for cost-effective fabrication. These details do not reflect any specific actual design conditions. All components of any tub girder bridge require careful analysis that may require deviation from preferred detailing practices for economical fabrication.

These details are based on similar details published by the AASHTO/NSBA Steel Bridge Collaboration, Task Group 1.4, in their document “Steel Bridge Design Detail Guidelines,” which is available at http://www.steelbridge.org.



PLAN

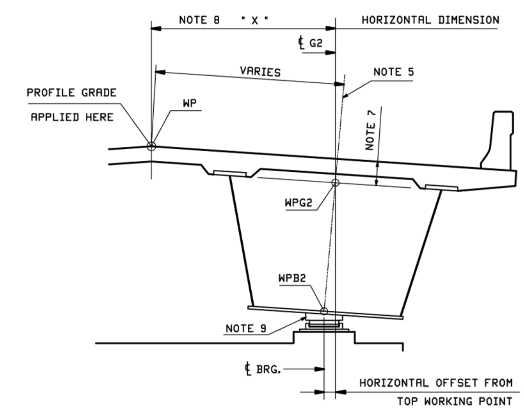


CROSS SECTION

LOOKING UP STATION

TABLE "I"

STATION	CROSS SLOPE OR SUPERELEVATION RATE



CROSS SECTION SHOWING

BEARING OFFSET

GENERAL GEOMETRY FOR STEEL TUB GIRDERS

KEEP THE HORIZONTAL DIMENSION BETWEEN TUB GIRDERS TOP WP'S CONSTANT AND ROTATE TUBS FOR THE REQUIRED BRIDGE CROSS SLOPE. ALSO SEE NOTE 6. THIS MAY RESULT IN THE THE LENGTH OF THE EXTERNAL DIAPHRAGMS BEING DIFFERENT DUE TO CHANGES IN THE SUPERELEVATION BUT WILL KEEP THE TUB GIRDER CENTERLINES RELATIVE TO THE HORIZONTAL CONTROL LINE.

FRAMING PLAN GEOMETRY

1... BASIC INFORMATION REQUIRED:
- STATIONS AT PC, SC, PT, ℄ BRG, ETC.
- AZIMUTHS , BEARINGS OR SKEW ANGLE OF ℄ PIER / ℄ BRG
- STATION AT CROSS SLOPE OR LANE WIDTH CHANGES.

2... SPACE CROSSFRAMES ALONG ℄ OF LONGEST GIRDER FOR MAX. SPACING.
3... LOCATE FIELD SPLICES (FS) RADIALLY TO EACH TUB. LOCATE ℄ FS FROM ℄ PIER.
4... CROSSFRAME LOCATION BASED ON DESIGN, TRY TO KEEP LATERAL BRACING AT AN ANGLE BETWEEN 30 DEGREES TO 45 DEGREES TO GIRDER WEB.

CROSS SECTION GEOMETRY

5... ROTATE TUB GIRDER WITH CROSS SLOPE.
6... MAINTAIN CONSTANT TRAPEZOIDAL SHAPE FOR ALL GIDERS ON A STRUCTURE. VARY DISTANCE BETWEEN TUBS IF DECK WIDTH FLARES.
7... MAINTAIN A CONSTANT DIMENSION FROM THE TOP OF DECK TO THE TOP OF THE WEB PLATE. SHOW THIS DIMENSION ON DESIGN DRAWINGS.
8... ALL TRANSVERSE GIRDER LOCATIONS SHALL BE LOCATED HORIZONTALLY. DO NOT CONTROL THE GIRDER LOCATIONS ALONG THE DECK.
9... USE A SINGLE BEARING ON CURVED AND/OR SKEWED STRUCTURES, TO ALLOW FOR TRANSVERSE ROTATION OF TUB. NOTE: BEARING DESIGN MUST ACCOMODATE THIS ROTATION.

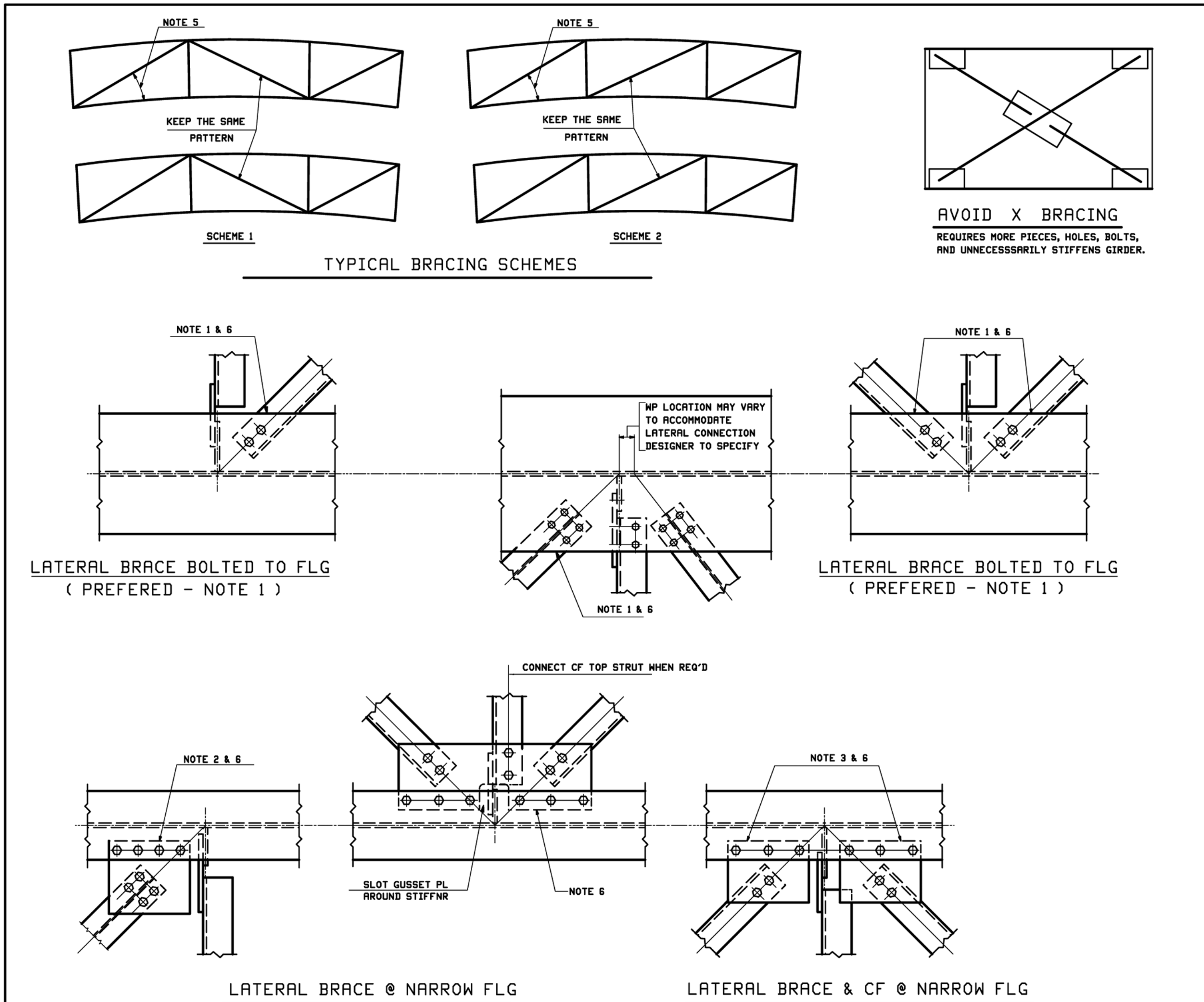
DISCLAIMER NOTE

INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

BASIC GEOMETRY - STEEL TUB GIRDERS

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS



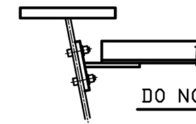
NOTES TO DESIGNER

1...WHEN POSSIBLE, BOLT LATERAL BRACING DIRECTLY TO FLANGES.

2...KEEP LATERAL GUSSET PLATES RECTANGULAR.

3...USE INDIVIDUAL GUSSET PLATES FOR EACH LATERAL BRACE.

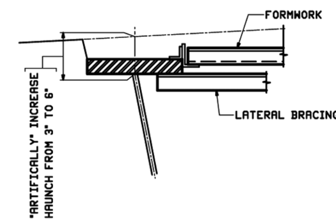
4...BRACING CONNECTIONS TO THE FLANGES ARE MORE ECONOMICAL, THAN CONNECTIONS TO THE WEBS, SINCE THEY INVOLVE FEWER COMPONENTS, AND BETTER FROM A DESIGN VIEWPOINT , SINCE IT PROVIDES A MORE DIRECT LOAD PATH.



DO NOT USE THIS DETAIL

5... ANGLE BETWEEN BRACING AND GIRDER SHOULD NOT BE LESS THAN 30° DEGREES, 45 DEGREES IS AN IDEAL ANGLE.

6... FILLS CAN BE USED TO DROP BRACING PLANE BELOW THE FORMWORK SUPPORTS. HOWEVER IT MAY BE MORE ECONOMICAL TO CUT FORMWORK AROUND THE BRACING MEMBERS. ANOTHER OPTION WOULD BE TO INCREASE THE HAUNCH DIMENSION AND RAISE THE FORMWORK SUPPORT ANGLES, SEE SKETCH:



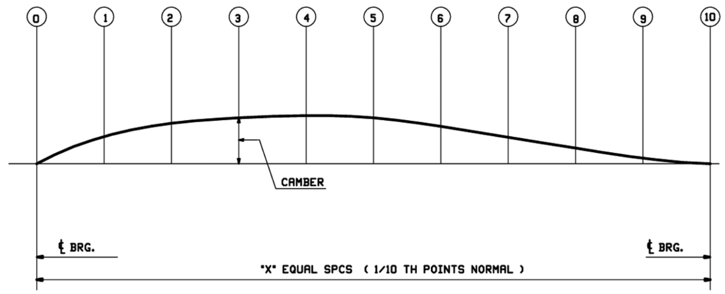
OPTIONAL HAUNCH DETAIL

THIS DETAIL APPLIES TO STAY IN PLACE FORMS.
DESIGNER TO INVESTIGATE IF S.I.P FORMS CAN BE USED

DISCLAIMER NOTE
INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

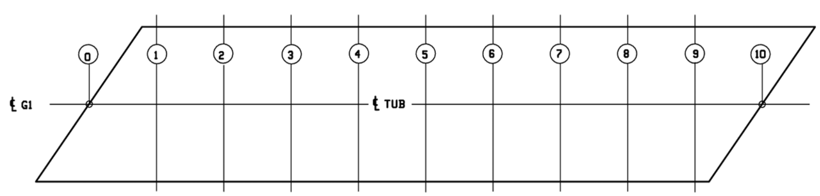
TUB GIRDER LATERAL BRACING

AASHTO/NSBA STEEL BRIDGE COLLABORATION
TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS



LINE	CAMBER VALUES DUE TO DL	0	1	2	3	4	5	6	7	8	9	10
		VALUES SHOWN ARE IN INCHES (NOTE 1)										
G1	STEEL DL	0										0
	DECK DL	0										0
	SUPERIMPOSED DL	0										0
	TOTAL DL	0										0

DO NOT PROVIDE GEOMETRIC CAMBER FOR TUB GIRDERS SINCE THIS INFORMATION IS NORMALLY GIVEN IN A VERTICAL PLANE AND A TUB IS NORMAL TO THE CROSS SLOPE. THE CAMBER DIAGRAM FOR EACH WEB IS BASED ON A MODIFIED CONICAL SHAPE IN THE PLANE OF THE WEB PLATE, WHICH IS NOT IN A VERTICAL PLANE.



TUB GIRDER CAMBER DIAGRAM

Show Camber Data Along ℄ Tub

DISCLAIMER NOTE

INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

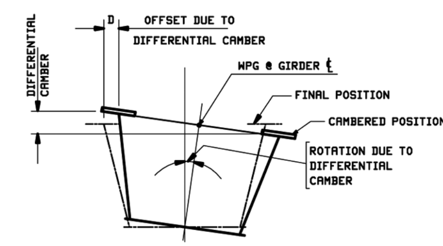
NOTES FOR DESIGN DRAWINGS:

1...CAMBERS CAN BE GIVEN IN FRACTIONS, DECIMAL OF A FOOT OR DECIMAL INCHES. STATE CLEARLY IF DIMENSIONS ARE IN FEET OR INCHES.

2...CAMBER INFORMATION IS SHOWN FOR A TUB GIRDER.

NOTES TO DESIGNER

1... AVOID HAVING DIFFERENT CAMBERS FOR EACH WEB BECAUSE OF PROBLEMS DUE TO DIFFERENTIAL CAMBERS.



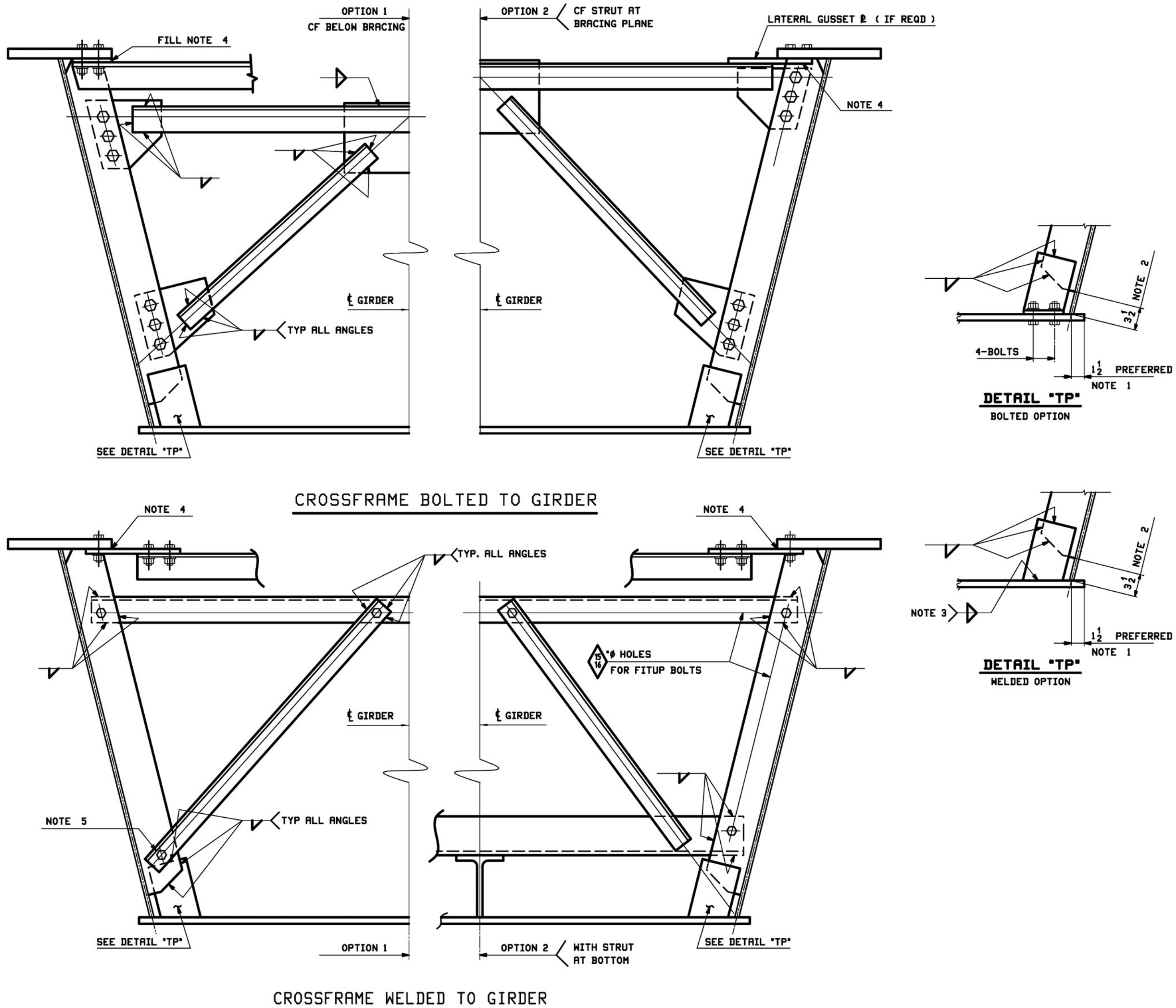
DIFFERENTIAL CAMBER

PROBLEMS WHEN NEAR WEB AND FAR WEB HAVE DIFFERENT CAMBERS. THIS CAN BE AVOIDED IF THE TUB IS CAMBERED ALONG ITS ℄

TUB GIRDER CAMBER DIAGRAM

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS



TYPICAL CROSSFRAME DETAILS FOR TUB GIRDER BRIDGES

DESIGNER MAY SHOW EITHER WELDED OR BOLTED CROSSFRAMES, BUT CONSIDER A FABRICATOR'S REQUEST FOR ALTERNATIVES.

1...THE BOTTOM FLANGE EXTENSION IS REQUIRED FOR FLUX SUPPORT AND THE WELDING MACHINE TRACKING. 1" MIN. MOST FABRICATOR'S PREFER 1½"MIN.

2...A GAP OF 3½" IS PREFERRED, DESIGNER TO CHECK 4 TO 6 tw REQUIREMENT. THE GAP AT THE BOTTOM ALLOWS THE WEB TO FLANGE WELDING TO BE MADE WITHOUT INTERRUPTIONS. HOWEVER THIS DETAIL MAY VARY DEPENDING ON THE FABRICATOR'S EQUIPMENT AND PROCEDURES. EXTENDING THE STIFFENER TO THE FLANGE WITHOUT THE USE OF TAB PLATES IS ACCEPTABLE.

3...WELD TO FLANGE WHEREVER FATIGUE STRESS RANGE PERMITS. IF REQUIRED FOR FATIGUE , USE A BOLTED TAB PLATE. SEE DETAIL "TP"

4...FILLS CAN BE USED TO DROP BRACING PLANE BELOW THE FORMWORK SUPPORTS. HOWEVER IT MAY BE MORE ECONOMICAL TO CUT FORMWORK AROUND THE BRACING MEMBERS.

5...ASSEMBLY BOLTS MUST NOT INTERFERE WITH WELDING.

USUAL ASSEMBLY SEQUENCE

A...WEB, TOP FLANGE AND STIFFENERS ARE USUALLY FABRICATED AS A SUB-ASSEMBLY PRIOR TO FITTING TO THE BOTTOM FLANGE.

B...THE CROSSFRAME IS BUILT IN A JIG AS A SUB-ASSEMBLY, FIT-UP AND WELDED. NOTE THAT ALL WELDING IS MADE FROM NEAR SIDE. BOLTED CROSSFRAMES ARE PREFERRED BY MOST FABRICATOR'S WHICH MINIMIZES ROLLING TUBS TO GET PROPER POSITION FOR WELDING.

C...THE CROSSFRAME SUB-ASSEMBLY IS THEN BOLTED TO THE WEB/TOP FLANGE SUB-ASSEMBLY WHICH WILL SHAPE THE FINAL GIRDER ASSEMBLY.

D...THE WEB/TOP FLANGE SUB-ASSEMBLY WITH THE CROSSFRAMES BOLTED IN PLACE IS THEN FITTED TO THE BOTTOM FLANGE PLATE WHICH HAS BEEN BLOCKED TO ITS CAMBERED SHAPE. THE WEB TO BOTTOM FLANGE PLATE WELDS ARE THEN MADE.

DISCLAIMER NOTE

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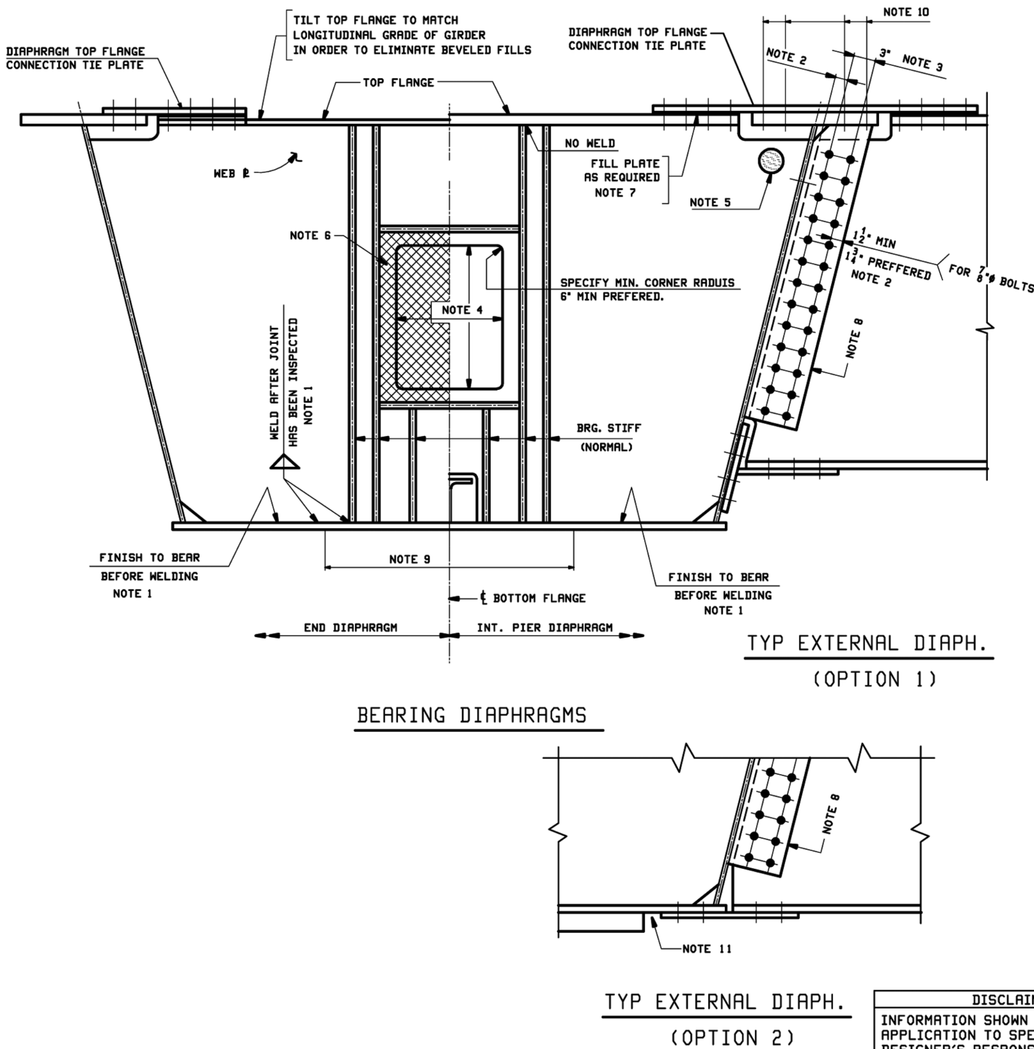
TYPICAL CROSSFRAME DETAILS
FOR TUB GIRDERS

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS



DISCLAIMER NOTE

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NOTES TO DESIGNERS

1...DETAIL DIAPHRAGM ASSEMBLY/BOTTOM FLANGE CONNECTION WITH FILLET WELDS AND FINISH TO BEAR SURFACES. AVOID FULL PENETRATION WELDS.

2...SINCE MOST FABRICATOR'S PREFER MORE THAN MINIMUM EDGE TO ALLOW FOR FABRICATION AND DRILLING TOLERANCES, PROVIDE ANOTHER $\frac{1}{8}$" TO $\frac{1}{4}$" MORE MATERIAL THAN THE MINIMUM REQUIRED.

3...ALLOW SUFFICIENT DISTANCE BETWEEN END DIAPHRAGMS AND BACK WALLS TO FACILITATE FIELD BOLTING (PREFERRED) OR PROVIDE ACCESS HOLES.

4...SIZE DIAPHRAGM ACCESS OPENINGS IN ACCORDANCE WITH STATE DESIGN GUIDELINES. 18" x 36" PREFERRED MINIMUM.

5...PROVIDE OPENINGS IN DIAPHRAGM TO FACILITATE RACEWAYS FOR MAINTENANCE LIGHTING CONDUIT. USE EXISTING CLIPS AT WEB TO FLANGE WELD IF POSSIBLE.

6...DIAPHRAGM ACCESS OPENINGS AT END OF UNITS SHALL BE COVERED BY SCREENED DOOR TO ALLOW FOR VENTILATION AND INSPECTION ACCESS AT PIERS WHILE PROHIBITING ANIMAL ACCESS. AT END DIAPHRAGMS, ADJACENT TO ABUTMENTS, AN ACCESS HATCH IN THE BOTTOM FLANGE SHOULD BE PROVIDED. SEE PAGE NO. 118 FOR DETAILS.

7...FILLS MAY BE REQUIRED FOR FIT UP ($\frac{1}{8}$" MIN) THICKER FILLS MAY REQUIRE DEVELOPING THE FILL BY EXTENDING PAST THE END OF THE TOP FLANGE CONNECTION TIE PLATE, DESIGNER TO CHECK AASHTO REQUIREMENTS.

8...CONNECTION CAN BE MADE WITH EITHER A CONNECTION ℄ OR A ANGLE. AVOID USING END PLATES WELDED TO THE DIAPHRAGM WEB.

9...FINISH BEARING CONTACT AREA TO BEAR. (PER 3.5.1.9 OF D1.5 BRIDGE WELDING CODE)

10...WHEN POSSIBLE, DO NOT CONNECT TIRE PLATE TO TOP FLANGE. IF BOLTING TIE PLATE TO GIRDER IS REQUIRED, DESIGNER SHOULD INVESTIGATE NET SECTION OF TOP FLANGE DUE TO THIS HIGHLY STRESSED AREA.

11...DESIGNER TO CHECK CLEARANCE TO SOLE PLATE.

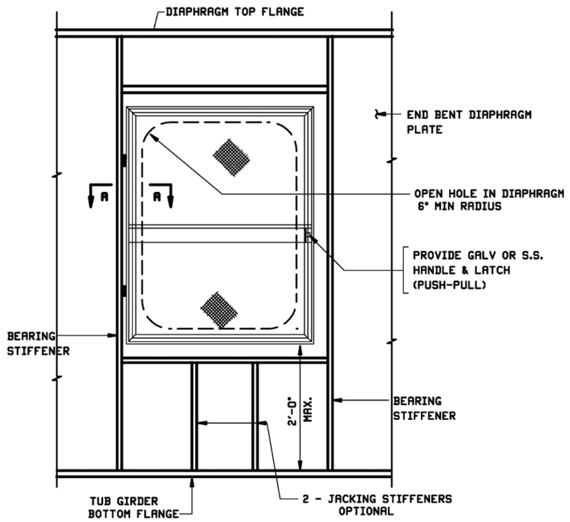
GENERAL DETAILING FABRICATION METHODS:

A...DETAIL DIAPHRAGM SO IT CAN BE SUB-ASSEMBLED, THEN FITTED TO BOTTOM FLANGE AND WEB ASSEMBLIES IN THE SHOP. KEEP STIFFENERS NORMAL TO BOTTOM OF FLANGE. IF BEARINGS ARE BOLTED TO FLANGE, THEN CHECK BOLT CLEARANCE TO STIFFENERS AND WELDS.

BEARING DIAPRAGMS - TUB GIRDER BRIDGES

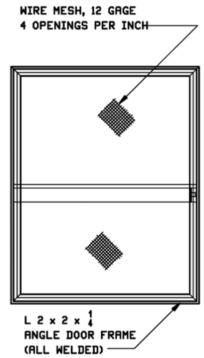
AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

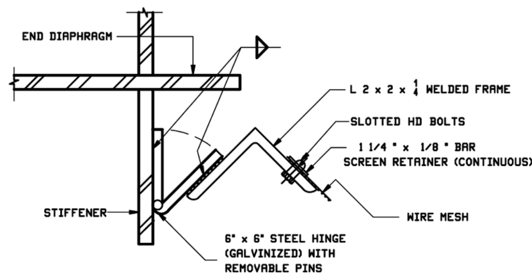


ELEVATION OF ACCESS DOOR

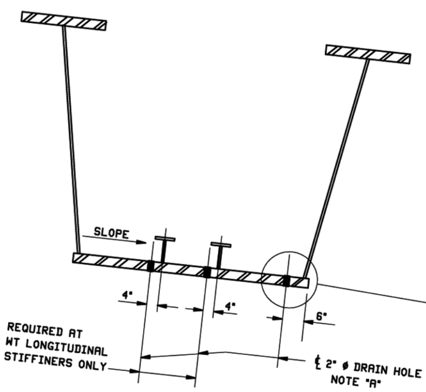
AT END BENT DIAPHRAGM OPENING



DOOR FRAME

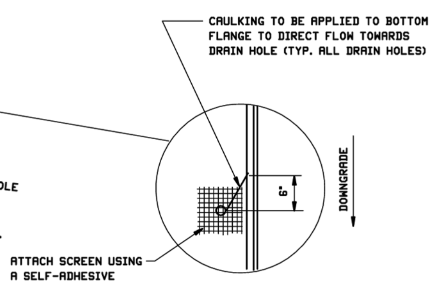


SECTION A-A



SECTION - GIRDER DRAIN HOLE DETAIL

PLACE DRAIN HOLES AT 50'-0" MAXIMUM SPACING,
AND 5'-0 MIN. FROM ℄ PIER OR BACKWALL.



PLAN OF DRAIN HOLES

DISCLAIMER NOTE

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DESIGNER'S RESPONSIBILITY.

NOTES FOR DESIGN DRAWINGS:

1...DOOR MUST OPEN TOWARDS THE INSIDE OF THE STEEL TUB GIRDER.

2...COST OF SCREENED CLOSURE DOOR IS INCIDENTAL TO THE COST OF STRUCTURAL STEEL.

3...STRUCTURAL STEEL FABRICATOR SHALL SUBMIT SHOP DRAWINGS FOR APPROVAL.

4...ALL WORK SHOWN ON THIS SHEET SHALL BE SHOP FABRICATED AND MOUNTED PRIOR TO SHIPPING TO THE JOB SITE.

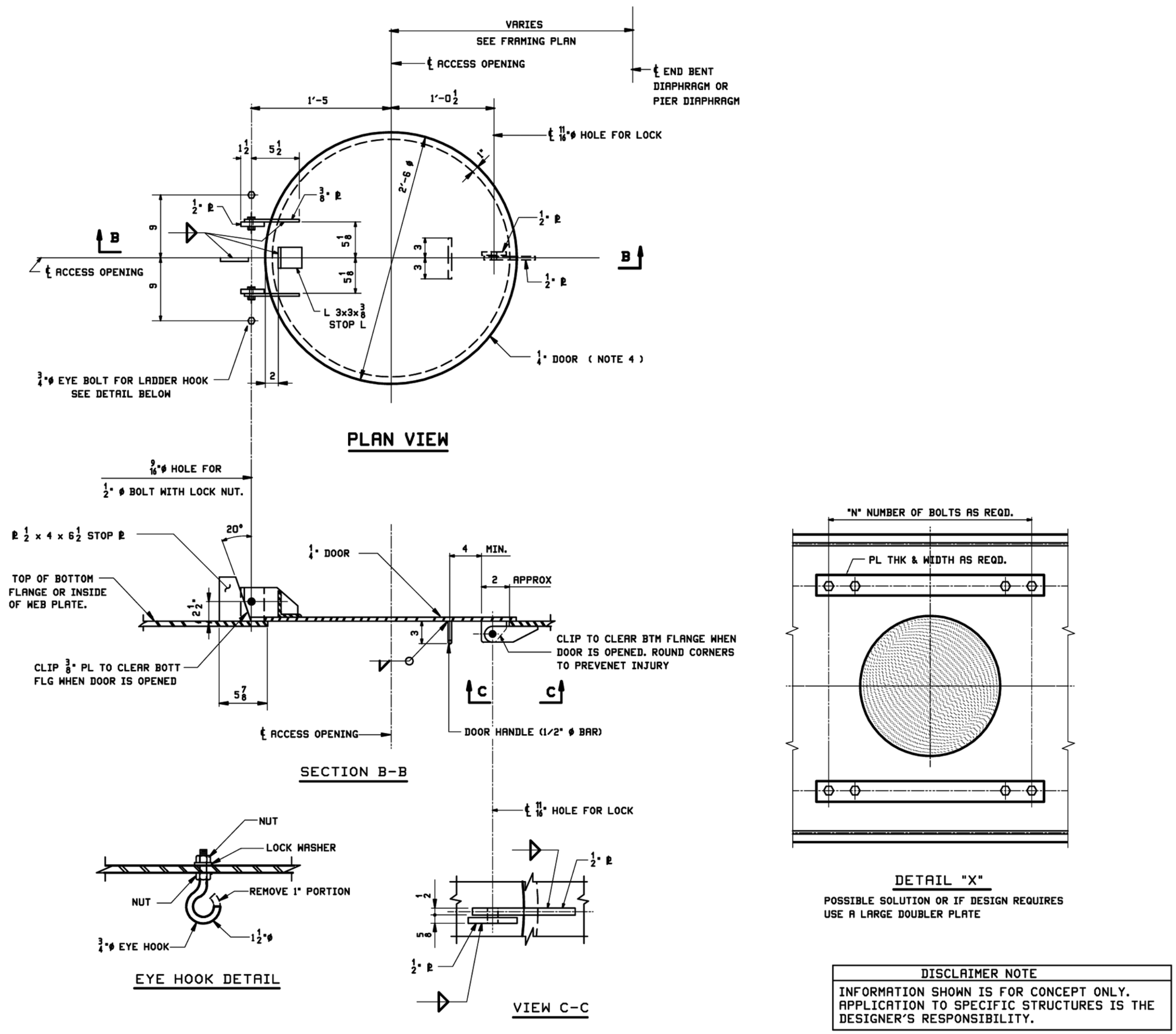
5... THESE DETAILS & MATERIAL SIZES ARE SUGGESTED AND ARE FOR A GUIDE ONLY, ENGINEER SHOULD CHECK WITH OWNER FOR POSSIBLE PREFERRED STANDARDS.

NOTE "A":
COVER VENT HOLES AND DRAIN HOLES WITH 20 GAGE GALVANIZED WELDED METAL SCREENING (1/4 " OPENINGS). ATTACH TO GIRDER WEBS/FLANGES, WITH AN OWNER APPROVED METHOD.

STEEL TUB SCREENING DETAILS

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS



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NOTE !!

THESE DETAILS & MATERIAL SIZES ARE SUGGESTED AND ARE FOR A GUIDE ONLY, ENGINEER SHOULD CHECK WITH OWNER FOR POSSIBLE PREFERRED STANDARDS.

THIS DETAIL COULD BE USED FOR A WEB OR FLANGE ACCESS DOOR.

NOTES FOR DESIGN DRAWINGS:

1...FOR ACCESS OPENING LOCATION SEE FRAMING PLAN.

2...ALL STRUCTURAL STEEL IN ACCESS HATCH SHALL BE ASTM A709 GRADE 36 AND SHALL BE GALVANIZED AFTER FABRICATION IN ACCORDANCE WITH ASTM A-123.

3...ALL EXPOSED EDGES OF PLATES AND OPENINGS SHALL BE GROUND SMOOTH.

NOTES FOR DESIGNERS

CHECK OUT AREA OF BOTTOM FLANGE, A REINFORCING PLATE MAY BE REQUIRED. SEE DETAIL "X"

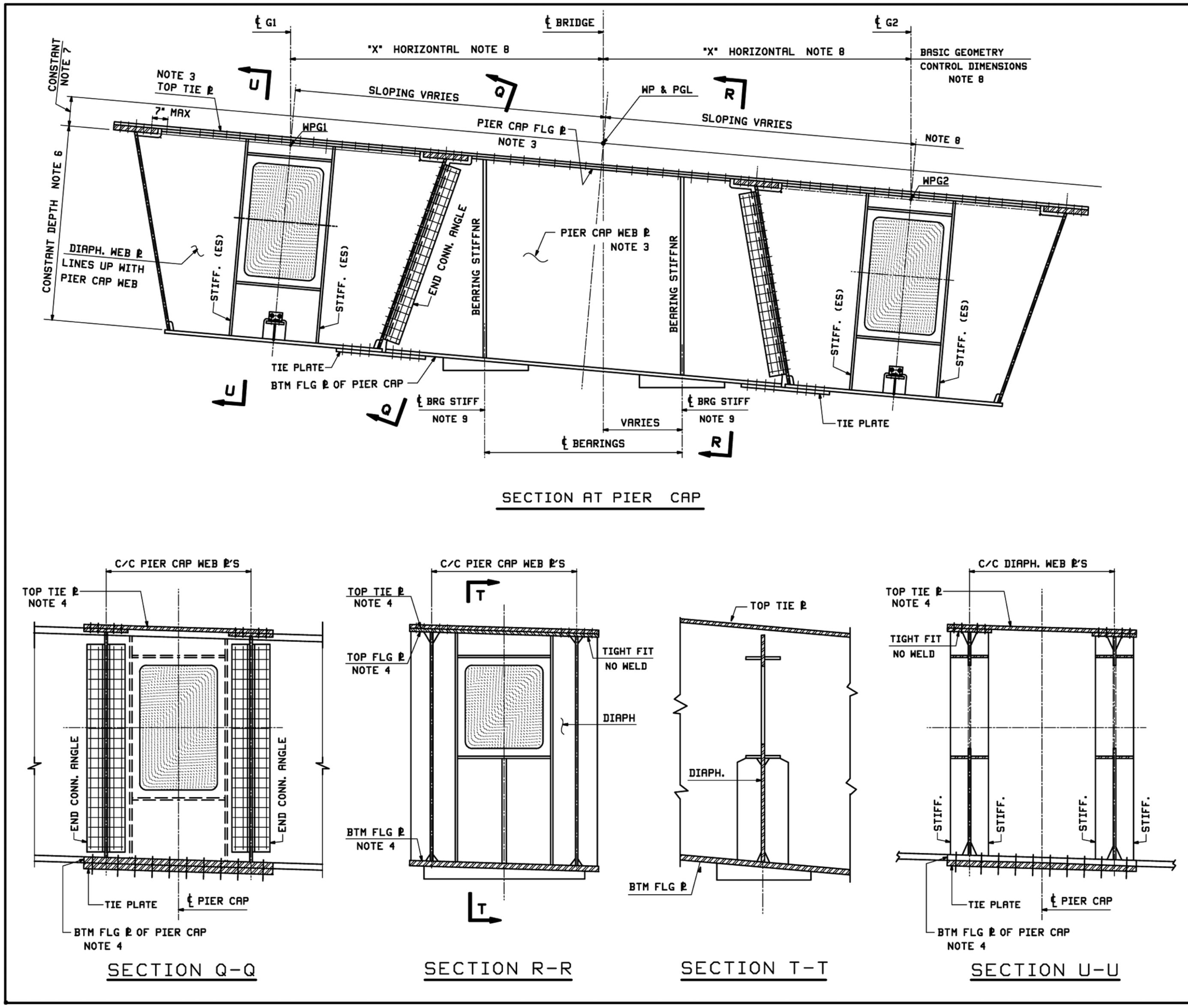
4...INVESTIGATE USE OF A PERFORATED PLATE, GRATING, OR OTHER OWNER PREFERRED MATERIAL.

5...DOOR MUST OPEN TOWARDS INSIDE OF THE STEEL TUB GIRDER

ACCESS OPENING DETAILS

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS



GENERAL NOTES

1...ALL STRUCTURAL STEEL SHALL BE ASTM A709 UNLESS OTHERWISE NOTED.

2...ALL BOLTED CONNECTIONS SHALL BE SLIP-CRITICAL.

3...TOP FLANGES AND WEBS OF DIAPHRAGM ARE FRACTURE CRITICAL AND SHALL MEET THE REQUIREMENTS OF CHAPTER 12 OF ANSI/AASHTO/AWS D1.5 BRIDGE WELDING CODE, FRACTURE CONTROL PLAN (FCP) FOR NONREDUNDANT MEMBERS.

4 ...SLOPE TOP AND BOTTOM FLANGES OF PIER CAP TO BE PARALLEL TO GRADE OF LONGITUDINAL GIRDER. THIS WILL ELIMINATE USING BEVELED FILL PLATES.

CROSS SECTION GEOMETRY

5... ROTATE TUB GIRDER WITH CROSS SLOPE.

6... MAINTAIN CONSTANT TRAPEZOIDAL SHAPE FOR ALL GIDERS ON A STRUCTURE. VARY DISTANCE BETWEEN TUBS IF DECK WIDTH FLARES.

7... MAINTAIN A CONSTANT DIMENSION FROM THE TOP OF DECK TO THE TOP OF THE WEB PLATE. SHOW THIS DIMENSION ON DESIGN DRAWINGS.

8... ALL TRANSVERSE GIRDER LOCATIONS SHALL BE LOCATED HORIZONTALLY. DO NOT CONTROL THE GIRDER LOCATIONS ALONG THE DECK.

9... BEARING STIFFERNERS MAYBE VERTICAL OR NORMAL TO FLANGES.

NOTES FO RDESIGNERS

A... AVOID FULL PENETRATION WELDS OF PLATES OR STIFFENERS TO FLANGES. USE FINISH TO BEAR WITH FILLET WELDS

B... SEE PAGE NO. 117 FOR ADDITIONAL APPLICABLE NOTES FOR ACCESS OPENINGS, BOLT SPACING, EDGE DISTANCES, WELDING , ETC.

C... SEE PAGE NO. 118 FOR DRAIN HOLES NOTES.

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EXAMPLE OF INTEGRAL PIER CAP STEEL BOX

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS