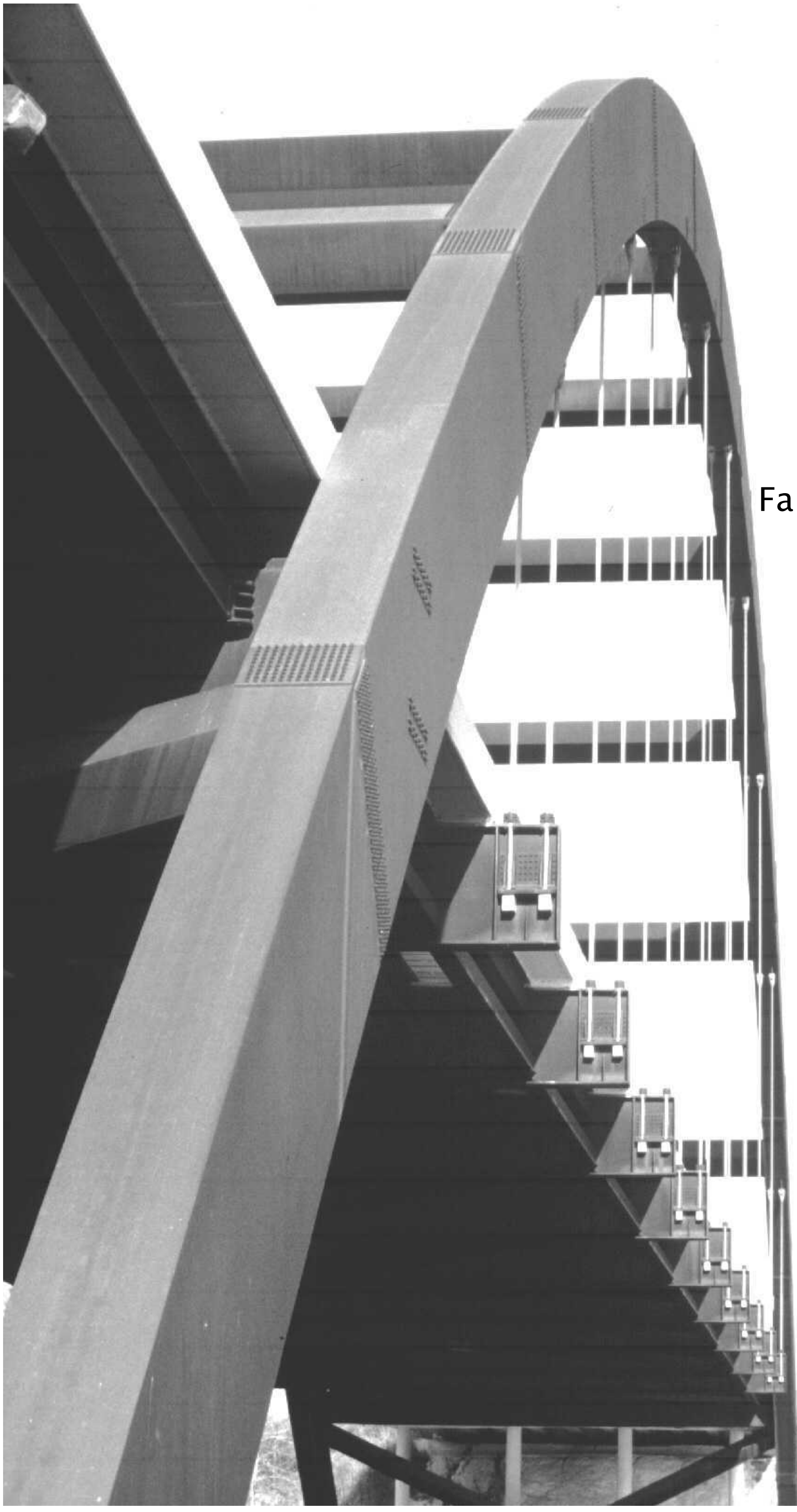

Preferred Practices

for

Steel Bridge

Design,

Fabrication, and Erection

2007

Texas Steel

Quality Council

Texas Department
of Transportation
(TxDOT)

Preferred Practices for Steel Bridge
Design, Fabrication, and Erection

April 2007

Table of Contents

1. Introduction

1.1. Overview

This document provides guidance to help steel bridge designers working on Texas Department of Transportation (TxDOT) projects to achieve optimal quality and value in steel bridges.

It is maintained by the Texas Steel Quality Council, which is a joint owner-industry forum comprised of the following:

- TxDOT design, fabrication, and erection engineers; TxDOT inspectors
- FHWA bridge engineers
- Academics
- Steel bridge fabricators, detailers, and trade association representatives
- Steel mill representatives; and design consultants.

The Council meets regularly in an open forum to discuss best practices for achieving the best steel bridges, and this document reflects the Council's agreements.

Open and informed participation by representatives from all aspects of steel bridge construction is instrumental to the Council's success, and the Council welcomes and encourages all comments. Submit comments and suggestions to the TxDOT Steel Quality Council, c/o the Texas Department of Transportation, Bridge Division, 125 E. 11th St., Austin, Texas, 78701-2483, or send comments by email to jholt@dot.state.tx.us.

Version	Publication Date	Summary of Changes
1	November 2000	New Document
2	October 2005	Updated to accommodate AASHTO Load and Resistance Factor Design (LRFD) specifications.
3	January 2006	Added reference to National Steel Bridge Alliance publication and updated reference to information on distribution of approved shop drawings.

2. Design

2.1. Preliminary Design Considerations

The success of a steel bridge design depends on the preparation of the designer:

- Have a well founded knowledge of design specifications.
- Be familiar with the construction and fabrication specifications and standard drawings that apply to steel structures.
- Be aware of construction and fabrication specifications' influence on design and any modifications or special provisions they may require.
- Take time during initial decision making to consult with fabricators, steel erectors, and contractors for ideas on achieving economical, easily built designs.
- While designing, think about how everything fits together—for example, how rotation, deflection (especially differential deflection), twist, stiffness (vertical bending, lateral bending, and torsion), and skew affect interaction between different elements.
- Always provide clear and distinct load paths that mitigate or, preferably, eliminate out-of-plane bending.
- Always provide enough access for bolting, welding, and painting. Keep designs simple by maximizing the use of common details and minimizing the number of plate sizes and rolled shapes fabricators are required to purchase. Complicated details are always hard to fabricate and build.
- Never use details that permit water and debris to collect on girders.

Designs that merely satisfy design specifications are rarely good. Good designs reflect consideration of the requirements of fabrication, construction, and maintenance.

2.1.1. Steel Grade Selection

If it is acceptable for the location, use weathering steel (A 709 Grades 50W and HPS 70W) left unpainted. FHWA Technical Advisory T 5140.22 and Section 2.1.2.3 of this document provide guidelines on acceptable locations. Some TxDOT districts object to the use of weathering steel, so obtain district approval before using weathering steel in a design.

Although weathering steel is slightly more expensive initially than non-weathering steel, it is ultimately more economical because it does not require initial or maintenance painting.

Weathering steel performs as well but not better than non-weathering steel in painted applications, so avoid requiring weathering steel in painted applications. Fabricators may prefer to use weathering steel for painted bridges, and TxDOT allows substitution of weathering steel for non-weathering steel if all other material requirements are satisfied.

Avoid using hybrid girders consisting of A 709 Grade 36 webs with A 709 Grade 50 flanges for painted bridges. Although these hybrids may seem economical, the cost difference of the two grades of steel is actually minimal.

In contrast, A 709 Grade HPS 70W may be economical only in hybrid girders. With Grade 50W webs, use a hybrid configuration with HPS 70W tension flanges and HPS 70W compression flanges in negative moment regions. The use of HPS 70W steel may be restricted by:

- The fact that its stiffness is not increased over lesser grades.
- Its availability. Do not specify HPS 70W steel in a design unless availability is verified with local fabricators—for example, rolled sections are currently not available in this steel.

A 709 Grade HPS 50W is also a high performance steel, but TxDOT does not recommend designs requiring its use.

A 709 Grade 50S (bridge steel equivalent to A 992) is acceptable for painted rolled beam bridges and anywhere structural shapes are used in painted applications.

Avoid using A 709 Grade 100 or 100W steel; Grade 100 steel has no real application for Texas because typical bridges are short- and medium-span bridges. Use A 709 Grade HPS 100W if 100-ksi steel is needed.

For cost analyses in the design phase, use the actual price of steel per pound instead of a typical bid price per pound. Fabricators, steel suppliers, or AISC can be consulted during the design phase of a job to provide the latest information about steel costs.

2.1.2. Proper Use of Weathering Steel (A 709 Grades 50W and HPS 70W)

You can easily ensure good performance from weathering steel and reduce or eliminate unsightly concrete staining. For detailed recommendations, see TxDOT's report on Project 0-1818, “Performance of Weathering Steel in TxDOT Bridges,” Chapter 4.

2.1.2.1. Design Strategies

Provide drip plates (also called drip tabs) to divert runoff water and protect abutments and columns from staining. Provide them on every girder because staining may occur before slab placement. Consider what the diverted water will stain—for example, do not place drip plates so close to substructure elements that wind blows diverted water onto the substructure. Drip plates over concrete riprap at bridge embankments cause stain spots on the riprap, but the alternative is widespread staining of both the abutment and riprap.

If necessary, provide stainless steel drip pans to protect abutments and columns. Doing so is impractical with large tub girders and may not be necessary for plate girders with good drip plate details.

Provide details that take advantage of natural drainage.

Provide adequate drainage beneath overpass structures to prevent ponding and continual traffic spray from below. Communicate the importance of adequate drainage to roadway designers.

Provide stiffener clips for proper ventilation and drainage. The stiffener clips shown on TxDOT standard drawings SGMD, “Steel Girder Miscellaneous Details,” and SBMD, “Steel Beam Miscellaneous Details,” are adequate for this purpose.

Eliminate details that retain water, dirt, and other debris.

Do not detail deck drains that can discharge water onto the steel, especially in regions that use de-icing chemicals.

Either completely seal box or tube members or provide adequate drainage and venting to allow condensation in unsealed tubular and box sections to dissipate. See Section 2.5 for more on this subject.

Use sealed expansion joints. See Section 2.1.5 for more guidance on expansion joints. Avoid any type of open joint that allows runoff to reach the steel.

Specify application of an adequate protective coating to surfaces that may be subject to standing water. However, avoid details that create this situation.

Specify application of an adequate protective coating to weathering steel that will be embedded in soil or gravel pockets. The coating should be one of the types used on carbon steel in the same environments, and it should extend above the interface of the embedment for several inches.

2.1.2.2. Fabrication and Construction

The 2004 TxDOT Standard Specifications require the following:

- Fabricators must blast clean (SSPC-SP6) fascia surfaces of weathering steel elements before shipping them.
- The Contractor must spot clean fascia steel surfaces in the field after the slab concrete has been placed.

Eliminate identification and other markings on the outside face of any fascia beam, including any markings placed during erection.

2.1.2.3. When Not to Use Weathering Steel

Do not use weathering steel in the following conditions:

- If the atmosphere contains concentrated corrosive industrial or chemical fumes.
- If the steel is subject to heavy salt-water spray or salt-laden fog.
- If the steel is in direct contact with timber decking; timber retains moisture and may have been treated with corrosive preservatives.
- If the steel is used for a low urban-area bridge or overpass that creates a tunnel-like configuration over a road on which deicing salt is used. In this situation, road spray from traffic under the bridge causes salt to accumulate on the steel.
- If the location has high rainfall and humidity or is constantly wet. This situation is rare in Texas.
- If the structure provides low clearance (less than 8 to 10 feet) over stagnant or slow-moving water.

2.1.3. Paint

TxDOT district offices usually select paint systems. If the plans do not specify a paint system, System II is the default paint system specified in the 2004 TxDOT Standard Specifications.

The 2004 TxDOT Standard Specifications have three paint systems for new steel bridge construction:

- System II—two-coat system with epoxy zinc primer.
- System III—three-coat system with inorganic zinc (IOZ) primer.
- System IV—two-coat system with inorganic zinc primer

Item 446, “Cleaning and Painting Steel,” provides a more detailed description of these systems.

TxDOT Standard Specifications require the inside of all tub and box girders to be painted with a light-colored paint (white polyamide cured epoxy) to facilitate future inspection. This paint is not intended to provide corrosion protection.

2.1.4. Span Configuration

Span configuration plays an important role in the efficient use of steel. Two-span continuous girders are not efficient because of high negative moments. However, they can be economical if maximizing prestressed concrete beam approaches leaves only enough room for a two-span continuous unit to fill in the remainder of the structure. Three- and four-span units are preferable but not always possible. Units more than four spans are not recommended. For three- and four-span units, make interior spans about 20 to 30 percent longer than end spans. If end spans are short in relation to interior spans, uplift can be a problem at the girder ends. If end spans are too long in relation to interior spans, a disproportionate amount of steel will be required for the end spans.

Always consider the presence of uplift at ends of continuous girders, particularly with light, rolled beam units or short end spans. Commentary to *AASHTO LRFD Bridge Design Specifications*, Article C3.4.1, indicates uplift to be checked as a strength load combination and provides guidance in the appropriate use of minimum and maximum load factors. Uplift restraint, when needed, should satisfy the Strength limit state and the Fatigue and Fracture limit state.

Using a continuous slab with simple span girders, typically done in Texas with prestressed concrete beams, can be an economical framing method. Advantages over continuous girder designs include elimination of costly air splices and heavy lifts during girder erection. Construction is also faster with simple span girders than with continuous girders; however, a potential drawback is loss of internal redundancy. In addition, more section depth is required. Investigate this framing method on a case-by-case basis to determine if it is economical.

Another economical framing method may be use of simple spans for dead load (beam and slab) and continuous for live load (and dead loads applied to the composite sections). TxDOT has done this with prestressed concrete I beams, but it did not prove economical. However, steel beams may be different if continuity details are simple enough. No detailed recommendations or suggestions are offered with this framing system.

2.1.5. Expansion Joints

Try to limit expanding lengths to allow use of standard strip seal expansion joints. Modular joints and finger joints are expensive and difficult to construct and maintain, and they have shown poor long-term performance; use them only as a last resort.

See TxDOT standard drawings SEJ-A and SEJ-P, “Sealed Expansion Joint Details (Without Overlay),” for strip seal expansion joint details. TxDOT prefers the joint on the SEJ-A standard for most bridges. The SEJ-P standard drawing has a larger rail section, and TxDOT recommends it only for structures carrying a large amount of heavy truck traffic such as those on NAFTA routes and interstate highways. The SEJ-P joint also requires more slab depth at the joint location than the SEJ-A joint because of the larger rail. Ensure slab depth is adequate for whichever joint is used.

Consider using an inverted-T bent rather than a modular or finger joint if you need to accommodate more thermal expansion than a 5-inch strip seal expansion joint’s capacity. You can extend the stem of the inverted-T bent through the slab to become the finished riding surface. You then place an expansion joint at each face. This type of bent is often designed with an assumed 6- or 8-inch riding surface placed on top of the stress-carrying cap after the bridge slabs in the adjacent units are placed. Use adequate reinforcing steel in the riding surface to accommodate live load strains. You can use this technique to eliminate finger or modular expansion joints. Aesthetics is a consideration, but you can usually design a corbel with a shape that complements the rest of the bridge. If the SEJ-P standard is used with this system, the riding surface should be 10 inches above the top of the cap so that the large joint rail will fit.

With a finger joint, use a trough consisting of a steel channel (C shape) and a deflector plate underneath the joint to direct water away from the girders. Neoprene troughs have been used in the past under finger joints but have performed poorly. The channel/deflector system should stop at slab edges and allow water to be discharged directly out low ends. Drain pipes are typically not necessary with this system, and if provided, they may quickly become clogged. Use a channel/deflector system on at least a 4-percent grade to ensure good drainage.

2.1.6. Girder Spacing (Plate Girders, Tub Girders, and Rolled Beams)

Many studies show that the weight of structural steel per square foot of deck area decreases as girder spacing increases. However, you should limit girder spacing (or web spacing with tub girders) to 10 feet for the following reasons:

- TxDOT standard drawings do not support girder spacings of more than 10 feet.
- Slabs (or floor systems) cannot adequately support certain overloads.
- Re-decking while maintaining traffic is more challenging.

When setting the girder spacing, also consider using prestressed concrete panels to form the deck. (See TxDOT standard drawing PCP, “Prestressed Concrete Panels.”) This deck-forming option is the most economical slab-forming method. TxDOT allows panels only on straight girders because TxDOT wants the stiffness full-depth cast-in-place decks provide for curved girders. However, TxDOT accepts use of prestressed panels for most straight girder applications with girder spacing of 10 feet or less.

TxDOT prefers a minimum of four I-shaped beams/girders for a vehicular bridge span. TxDOT prefers a minimum of three tub girders in order to eliminate fracture-critical designation. Three tub girders are not possible on one-lane connectors, forcing the use of a two-tub-girder cross section.

2.1.7. Available Length of Material

Design consideration for splice location depends in part on the length of plate that is available. No single maximum length is supplied for all plate sizes. The maximum plate length depends on a couple of factors:

- Weight of the material—The thicker the material, the shorter the maximum available plate lengths.
- Material type—Fracture-critical materials may have special length limitations depending on the procedures and equipment used by the mill. For example, steel that must be normalized must be short enough to fit inside a furnace.

Consult a fabricator or steel mill on typical length limits. These limits vary from mill to mill and with material type and thickness.

2.2. Plate Girders

TxDOT standard drawing SGMD, "Steel Girder Miscellaneous Details," provides common details for use with typical plate girder spans.

When you consider material transitions, weigh labor and welding costs against potential material savings. When these costs are high, minimize the number of splices. Allow fabricators the flexibility to adjust the number and location of splices with designer approval. Designers should include a note in design details stating that adjustment to the number and location of transitions may be allowed with designer approval.

2.2.1. Flange Width

For curved girders, flange width should be approximately one-third the web depth and no less than 30 percent of the web depth. For straight girders, a flange width of about one-fourth of the web depth should be sufficient. Do not use flanges less than 15 inches wide. The extra width for curved girders enhances handling stability and helps keep lateral bending stresses within reason.

Maintain a constant flange width for each girder field section. Girders adjacent to each other should have the same flange width dimension to simplify slab formwork and to prevent variation in diaphragm or cross-frame geometry at interior bearings.

Flange width transitions are permissible only at field splices. All girders should have the same width transition at the same field splice location.

Width increments should be in whole inches.

If you allow panel forming, which is recommended for straight girders, the designer is responsible for ensuring that the required studs can be placed on the girder without interfering with the panels.

Flange width affects girder stability during handling, erection, and deck placement. Keep the girder length (field section length) to flange width ratio below 85.

In most cases, top and bottom flanges should be the same width. Girders in positive bending that are composite with a slab can have a top flange narrower than the bottom flange, but assess weight savings against reduced lateral stability before hardening of the deck.

2.2.2. Flange Thickness

TxDOT prefers a minimum flange thickness of 1 inch. For straight girders, you may reduce flange thickness to ¾ inch. Thinner plate will "cup" excessively when welded to the web.

TxDOT prefers a maximum flange thickness of 3 inches. Grade 50 and HPS 70W steels are not available in thicknesses greater than 4 inches. Weld time increases disproportionately when splicing plates thicker than 3 inches.

Flange thickness must satisfy AASHTO b/t requirements. AASHTO requirements for compression flange b/t ratios are more stringent than for tension flanges. Remember that flanges in tension under dead load can experience compression with certain live load positions or with construction sequencing.

Use a 10-foot minimum length for any given flange thickness on a girder. This is more a practical than a design consideration.

Use only a few flange sizes, and do not use a small quantity of one flange size. An economical girder will have as many as four or six flange sizes on a continuous girder and as many as two or three sizes for a simple span. On jobs with multiple structures, designers should communicate with each other and establish a preliminary list of no more than eight flange plate thicknesses to use. Refine the list as the designs progress if beneficial.

You can probably economically introduce a flange splice if you can save about 800 to 1000 pounds. These numbers are approximate and are a function of the current cost of steel plate.

TxDOT prefers flange thickness increments of 1/4 inch from 1 to 3 inches, and 1/2 inch from 3 to 4 inches. Fabricators prefer no plate thickness changes that are relatively small—for example, from 1 to 1.125 inches. Generally, the thicker plate should provide at least 25 percent more area than the thinner plate. In addition, the thinner flange should be no less than half the thickness of the thicker flange.

If nominal fatigue resistance is not satisfied at Category C and C ' details, increase flange thickness to eliminate the problem. The extra weight of a slightly thicker flange is less expensive than bolted stiffener tabs. Category C (stud connectors) and C ' (toe of stiffeners) are typically the controlling fatigue details for plate girders. Avoid details more critical than these Categories.

Do not use lateral bracing. Increase flange thickness if necessary. Lateral bracing creates fatigue-sensitive details and is costly to fabricate and difficult to install.

For curved girders, it is more economical for some fabricators to heat-curve the girders than to cut the flanges from a wide plate, producing waste. For other fabricators, the cost of the waste can be less than the time and labor to heat-curve. Designs should allow heat-curving (see Steel Bridge Fabrication Guide Specification, AASHTO/NSBA Steel Bridge Collaboration S2.1-2002). Let fabricators decide the most economical way to produce curved flanges.

"Slabbing and stripping" refers to a process in which wide plates of different thickness are welded to each other and individual flanges are then cut from this assembly. Because plates typically come in 42-inch or 48-inch minimum widths, this process reduces the number of individual flange splices required. It also reduces flange plate handling costs in the shop. The fabricator may not elect to slab the flanges, and it is not always feasible with curved girders. However, straight girder designs should allow the possibility of slabbing and stripping.

2.2.3. Flange Splice Locations

Locate splices at least 6 inches away from a web splice or transverse stiffener in order to facilitate non-destructive testing of welds.

Splices should be at least 10 feet apart.

Change flange width only at field splices, which are also good locations to change flange thickness.

2.2.4. Web Depth

The recommendations in *AASHTO LRFD Bridge Design Specifications*, Article 2.5.2.6.3, provide a good estimate of a minimum web depth for straight girders. Consider this depth a starting point. If vertical clearance is not a problem, adding depth can result in lighter girders. For curved girder web depth, use either the AASHTO recommended minimum depth for straight girders, increased by 10 to 20 percent, or use LRFD Equation 2.5.2.6.3-1 as a starting point. Aesthetics also has a role in girder depth. A rule of thumb for a well-proportioned superstructure is to have total section depth (slab plus girder) in the range of 0.033L to 0.04L (L = c–c brg length).

Use web depths in whole-inch increments.

At girder ends, dap webs such that total superstructure depth, including bearings, closely matches that of an adjacent span if the difference in depth is much more than 6 inches. This is done primarily for aesthetic reasons and to minimize tall reinforced concrete pedestals for bearings on substructure elements. Dapping girder ends can also be done at abutments to keep backwall heights within reason. Show the web dap to the nearest whole inch. No more than 40 percent of the web depth should be dapped.

Dap girder ends by cutting the web at a slope (1:1 minimum, 4:1 maximum) and allowing for cold bending of the flange to fit the web. Although AWS D1.5, Article 12.12, prohibits cold bending of fracture critical members, neglect this AWS provision because the stress range is very low near the support (dap location). Make a provision for an allowable shop splice in the flange immediately beyond the bends so that the fabricator does not have to work with a long piece of flange during bending operations. The slope should begin a sufficient distance away from the face of the substructure (6 inches minimum, taking substructure skew into account) so that the girder will not hit the substructure when it expands and so that erectors will have some latitude in moving the girder longitudinally.

Do not use haunched webs where the web is deeper over interior supports for spans less than 400 feet. Haunched webs have more potential for fabrication error and material waste can be excessive, and with the advent of A 709 Grade HPS70W, there is little reason to use them with spans less than 400 feet. The only arguments for their use are a possible aesthetic advantage over normal girders and maximizing freeboard for river crossings.

2.2.5. Web Thickness

Minimum web thickness is 1/2 inch. Thinner plate is subject to excessive distortion from welding.

For web depths up to 96 inches, provide sufficient thickness to preclude the need for longitudinal stiffeners. Longitudinal stiffeners do not provide economical designs and present fabrication and fatigue problems that make their use in a design unwise. They may be justifiable in deep girders, but you can easily design most structures built for TxDOT without them.

Web thickness should also be sufficient to eliminate the need for transverse stiffeners either entirely or partially. In high shear regions if transverse stiffeners spaced at about 8 to 10 feet prevent the need for a thicker web, you can justify using the stiffened web. Consider diaphragm or cross-frame connection plates as transverse stiffeners if they are needed in order to obtain a higher shear capacity, provided their spacing does not exceed AASHTO requirements. TxDOT discourages the use of fully stiffened web designs.

Optimum designs have few sizes, similar to flanges. A reasonable target would be three or fewer sizes for a continuous girder and one or two for a simple span.

Web thickness increments should be 1/16 inch up to a plate thickness of 1 inch. Use 1/8-inch increments beyond this.

2.2.6. Web Splice Locations

Splices should be at least 10 feet apart.

Locate web shop splices at least 6 inches away from a flange splice or transverse stiffener in order to facilitate non-destructive testing of welds.

2.2.7. Web-to-Flange Welding

Design web-to-flange welds and show them in the span detail drawings on the girder elevation. In most cases, the American Welding Society (AWS) minimum size weld (5/16 inch; see AASHTO/AWS D1.5) is sufficient. Welds more than 3/8 inch require multiple passes and drive fabrication costs up significantly.

2.2.8. Stiffeners

Plates provided only as a means to connect diaphragms or cross frames to girders are included as stiffeners even though they are not typically considered stiffeners.

TxDOT standard drawing SGMD shows stiffener lengths and how the stiffeners are welded to the girders. If a design requires welds larger than AWS minimums, indicate this on the span detail drawings. The SGMD standard drawing also provides direction to the fabricator on the orientation of the stiffeners—for example, plumb or perpendicular to the girder. Corner clips are also detailed on the SGMD standard drawing.

Fabricators strongly discourage full-penetration welding of bearing stiffeners to flanges. Full-penetration welds distort the bearing area of the bottom flange. Use finish to bear at the bottom flange and tight fit at the top. If a diaphragm or cross frame is attached to the bearing stiffener, use fillet welds to connect the stiffener to both flanges as shown on the SGMD standard drawing.

2.2.8.1. Width

Bars are more economical than plates for stiffeners. Bar widths come in 1/4-inch increments for widths under 5 inches and in 1/2-inch increments for widths of 5 to 8 inches. To take advantage of bar use, specify stiffener widths in 1/2-inch increments. Specify thickness in 1/8-inch increments using 3/8-inch as an absolute minimum.

The stiffener's width should be sufficient to provide clearance for field welding of diaphragm and cross-frame members to the stiffener, particularly when field welding is required near the face of the web. Welders recommend 4 inches or more of clearance between the web face and vertical welds on a gusset plate/diaphragm member. Welders recommend three inches or more of clearance between a gusset plate/diaphragm member and a flange. The SGMD standard drawing specifies minimum stiffener widths, which are based on these recommendations. (See Figure 1.)

Bearing stiffeners should extend to about 1/2- to 3/4- inch from the flange edge. They should be wide enough to facilitate field welding of diaphragm members as shown in Figure 1.

Stiffeners can extend beyond the flanges if they do not interfere with slab forms and bearing anchor bolts and lateral guides.

If a diaphragm or cross frame is not connected to the stiffener, use the minimum size stiffener allowed by AASHTO.

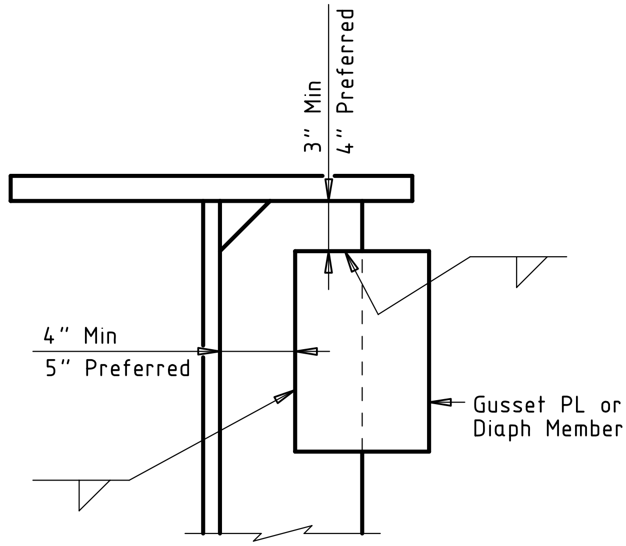

Figure 2-1. Recommended Clearances for Field Welding

2.2.8.2. Thickness

Use few stiffener sizes for a girder. For example, if the design requires minimum bearing stiffener thicknesses of 1 inch, 1.25 inches, and 1.5 inches, use 1.5-inch stiffeners at all locations. Use a bearing stiffener thickness that matches a flange thickness.

Size intermediate bearing stiffener thickness according to AASHTO requirements, and round up to the nearest 1/4 in.

Bearing stiffeners should be thick enough to preclude the need for multiple bearing stiffeners at any given bearing. Multiple stiffeners present fabrication difficulties and usually are not needed.

2.2.9. Bearings

For most plate girder spans, select bearings from TxDOT standard drawing SGEB, or use a modified version of a bearing on this drawing. Avoid costly proprietary High Load Multi-Rotational (HLMR) bearings (disc, pot, and spherical bearings). Always provide enough cap width, length, and depth to accommodate bearings and their anchor bolts.

On skewed structures, ensure that sole plates do not conflict with abutment backwalls, inverted-T stems, or beams in an adjacent span.

2.2.10. Field Splices

Show field splices in the design detail drawings as welded splices. Offer bolted splices as an option, and show them in the design details unless they are not desired for aesthetic reasons or if splice plates interfere with diaphragm or cross-frame locations. In the design details, note that structural steel pay weight is based on welded field splices regardless of splice type chosen by the contractor. No design for a welded field splice is typically required other than locating the splice and ensuring that nominal fatigue resistance is satisfactory. Item 448, “Structural Field Welding,” in the 2004 TxDOT Standard Specifications provides fabricators and erectors the necessary details of welded field splices.

See Section 2.7, “Bolted Connections,” for more information related to bolted field splices.

Locate field splices at points of dead load contraflexure. They do not have to be at the exact contraflexure point but should be reasonably close. Field splices do not have to be present at every contraflexure point. If the spans are short enough, you may be able to skip a contraflexure point without violating length limitations.

Make girder field lengths about 130 feet maximum, keeping in mind site access and the stability criteria in Section 2.2.1. Provide optional field splices in the design if girder field length is longer than 130 feet. Keep the stresses in the girder at an optional splice as low as possible. A flange splice is not a good location for an optional splice because the stresses in the thinner flange, by design, are usually close to the maximum permitted.

Splices must be far enough away from diaphragms or transverse stiffeners to allow room for splice plates.

For curved girders, do not let the girder sweep plus the flange width exceed 6 feet for ease of shipping. The current legal vehicle width is 8 feet 6 inches without a permit. Limiting the overall shipping width of curved girders to 6 feet permits fabricators to offset the girder on the trailer, as is frequently done, while not exceeding an overall width of 8 feet 6 inches. Add optional field splices if required, as noted above.

For straight girders, pay close attention to the interaction between the panel bedding strips and top flange splice plates and bolts where prestressed concrete panels may be allowed as a slab-forming option. Additional slab haunch may be required to accommodate bolt head height.

The legal vehicle height limit is 14 feet. Most trailers are approximately 4 feet high. Assume approximately 6 inches for dunnage. Overall girder depth, including stud connectors, should not exceed 9 feet for ease of shipping.

Optional bolted field splices often require fill plates in flange splices. The steel grade specified for the girders is frequently not available in thicknesses of 3/8 inch or less. If you need flange fill plates less than 3/8 inch thick, allow optional fill plate material (A 606, A 570, etc.) that meets design requirements.

Do not bring HPS 70W steel into a field splice unless stress demands require it. At low stress regions, where splices should be located, AASHTO’s minimum splice strength requirement forces a greater number of bolts and larger splice plates than would otherwise be required.

2.3. Rolled Beam Sections

TxDOT standard drawing SGMD, "Steel Girder Miscellaneous Details," provides common details for use with rolled (wide-flange) beam spans.

Rolled beams can be more economical than plate girders for their applicable span lengths because of decreased fabrication costs.

2.3.1. Sections

Select beams that have a top flange that is sufficiently wide to provide adequate spacing for three stud connectors per row. If prestressed concrete panels are allowed as a forming option, the flanges should be at least 12 inches wide. The designer is responsible for ensuring that the required stud connector spacing does not create a conflict with prestressed concrete panels if panels are allowed as an option.

Satisfy flange proportion limits in the *AASHTO LRFD Bridge Design Specifications*, and ensure that flange width is sufficient for handling and erection stability (see Section 2.2.1).

For continuous spans, if changing weights at splices, beams must be from the same rolling family as given in American Institute of Steel Construction (AISC).

The beams should be large enough that the elastic neutral axis of the composite section is within the steel beam, not within the slab or haunch.

Do not use cover plates. Their fatigue category is too low.

Do not use sections smaller than W21, which would require modifications to the SGMD standard drawing.

2.3.2. Stiffeners

Rolled beams usually do not need bearing stiffeners. Verify this using the provisions in AASHTO LRFD Article D6.5.

2.3.3. Bearings

Select bearings from TxDOT standard drawing SGEB, or use a modified version of these bearings. For simple spans with rolled beams, bearing designs depicted on TxDOT standard drawing SBEB are more economical designs. These bearings were designed for TxDOT's standard steel beam spans only, but they may work for custom-designed bridges if analyzed for adequacy.

If calculated uplift is present at ends of continuous units, an uplift restraint satisfying the strength and fatigue limit states is required. The possibility of uplift during slab placement must be investigated and accommodated if present.

Do not use a continuous beam that has calculated uplift at end bearings with the Service I load combination under any circumstances.

2.3.4. Field Splices

The information in Section 2.2.10 applies.

2.3.5. Camber

Camber rolled beams for all continuous beams and for simple spans over about 50 feet. Camber continuous beams for total DL deflection and roadway vertical curves. Camber simple spans for total DL deflection only. For rolled beams, specify the welded plate girder camber tolerances in AWS D1.5 in the design detail drawings. Consult fabricators to determine whether the required camber can be achieved with the proposed beam section.

For detailing required camber, show only the mid-ordinate of simple-span beams in the design details. For simple span beams less than about 50 feet, note in the design details to erect beams with natural camber up.

2.4. Tub Girder Sections

There are no standard TxDOT details for tub girders at present. Most of the guidelines outlined are for curved, continuous tub girder units. Application of these guidelines to tangent and simple span construction is at the designer's discretion. Additional suggestions are available in the National Steel Bridge Alliance publication, "Practical Steel Tub Girder Design" (see http://www.aisc.org/Content/ContentGroups/Documents/NSBA5/TubGirderBookLink.pdf).

Tub girders should have a constant trapezoidal or rectangular shape and should be rotated with the cross slope. Keep the top-of-slab to top-of-web dimension constant. See Figure 1 for preferred horizontal geometry. Departure from the shown geometry can result in extreme difficulties generating shop drawings. The profile grade line and horizontal control line locations in Figure 2-2 are for example purposes only.

Take the centerline-of-bearing offset into account when substructure elements are detailed.

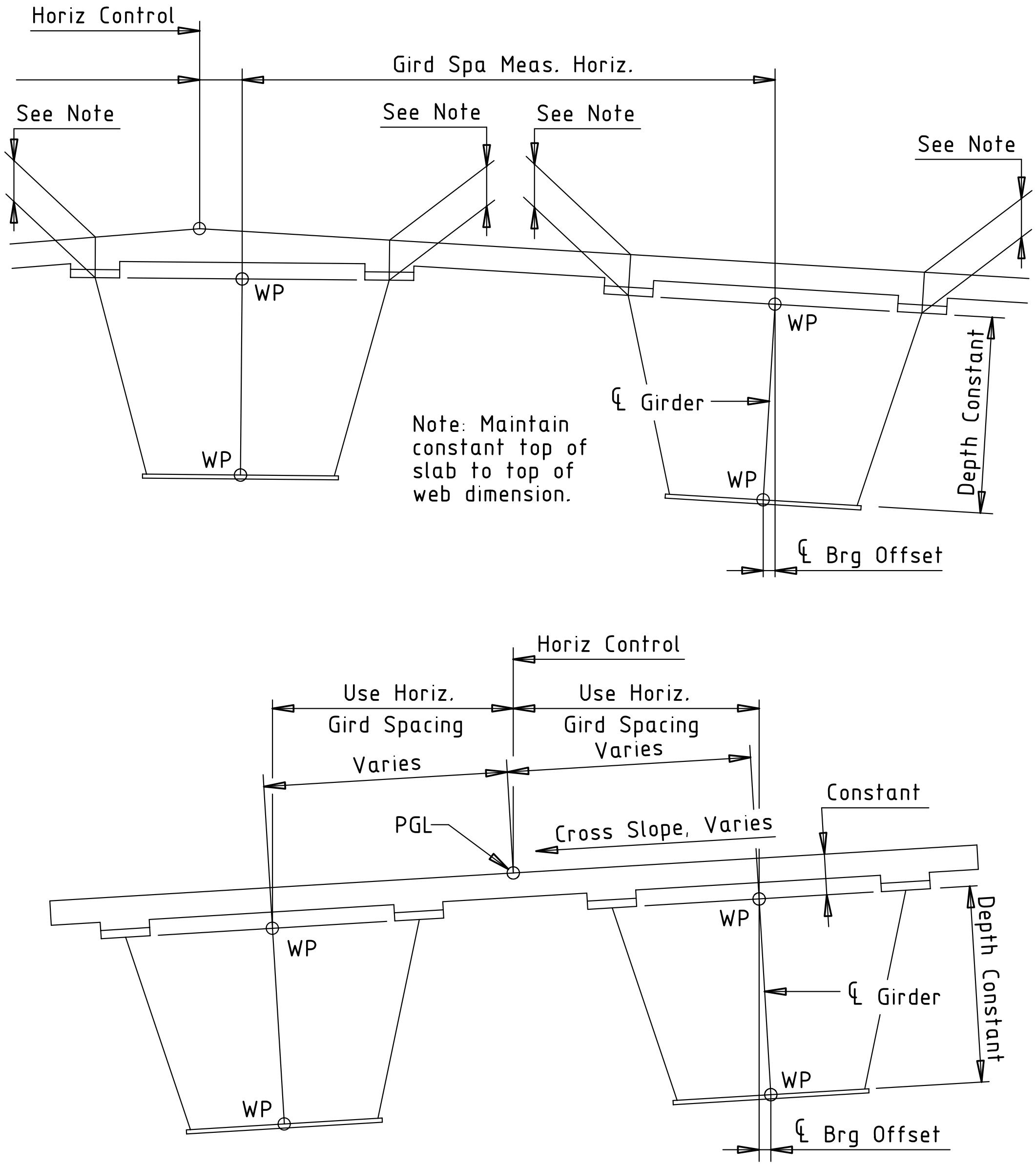

Figure 2-2. Recommended Tub Girder Horizontal Control

2.4.1. Flanges

In addition to AASHTO requirements, top flanges for tub girders should follow the suggestions for plate girder flanges in Section 2.2.

For bottom flanges, plate distortion during fabrication and erection can be a problem. Check with fabricators when using bottom tension flange plates less than 1 inch thick to determine whether practical stiffness needs are met. Bottom tension flanges should never be less than 3/4 inch thick. In addition, the bottom tension flanges should have a w/t ratio of 80 or less.

At present, no information is available regarding the possible economic benefits of using a thinner, longitudinally stiffened compression flange over a thicker, unstiffened flange. Until such information is available, discuss options with fabricators who are experienced with tub girders.

If using longitudinal stiffeners, try to maintain a clear distance between longitudinal stiffeners of no less than 24 inches (more is better) to accommodate automated welding equipment. Therefore, the minimum flange width, between webs, is 48 inches when using one stiffener and 72 inches for two stiffeners. Do not use more than two stiffeners per flange.

For straight girders, plates or bars are recommended over WT shapes for longitudinal stiffeners as long as they meet AASHTO criteria. Plate and bar sections are less expensive and easier to splice than WT sections. For curved girders, WT sections are recommended. If WT sections are used, the suggested ratio of the depth to one-half the WT flange width should be greater than 1.5 to provide good welding access. Try to allow termination of longitudinal stiffeners at a bolted field splice such that fatigue is not a concern at the stiffener's end.

Bottom tension flanges of two-girder spans are classified as fracture-critical. Top tension flanges are not fracture-critical unless only one girder per span is used. The tension portion of each web plate attached to a fracture-critical flange plate is fracture-critical itself. Avoid details more critical than Category C with fracture critical members (and for non-fracture-critical members as well).

Bottom flange edges should extend at least 2 inches beyond the web centerline to facilitate automated welding.

2.4.2. Webs

The suggestions in Sections 2.2.4 and 2.2.5 apply.

2.4.3. Stiffeners

For stiffeners and connection plates for internal cross frames, a good option to provide fabricators is shown in Figure 2-3. Cutting the stiffeners short of the bottom flange facilitates automated welding of the web to the bottom flange. After this welding is complete, the stiffener can then be attached to the bottom flange with an additional plate.

Refer to Section 2.11, Bearing Replacement, for information on stiffeners to be used for future bearing replacement.

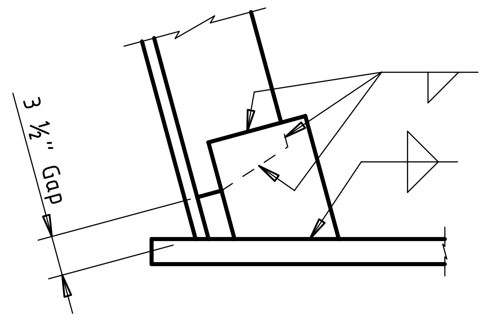

Figure 2-3. Recommended Tub Girder Stiffener Option

Do not specify complete penetration groove welds to connect bearing stiffeners to bottom flanges. Weld-induced flange distortion is even more of a problem with tub girder flanges than with plate girders.

2.4.4. Top Flange Lateral Bracing

Use lateral bracing in straight tub girders and in curved tub girders.

Bolt lateral bracing directly to the top flange. Provide enough slab haunch that formwork does not interfere with the bracing. Do not use shims or fills between the lateral bracing and top girder flange that increase eccentricity of the connection.

TxDOT prefers single-laced lateral bracing over double-laced bracing. The angle between the girder flange and bracing should be at least 35 degrees. An angle closer to 45 degrees is ideal.

Consider erection loads and sequential concrete placement when determining the worst-case loading for lateral bracing.

2.4.5. External Diaphragms and Cross Frames (between Piers)

External diaphragms or cross frames are normally used to control relative displacement and twist of girders during slab placement. Once the slab has matured sufficiently, they may be removed, which is done primarily for aesthetic reasons. If they are to remain in place, they should complement the overall structural aesthetics and should contain fatigue-resistant details.

For curved tub girders, external diaphragms or cross frames at span quarter or third points are usually sufficient; adding more is unnecessary. With straight tub girders, one external cross frame or diaphragm at mid-span should be sufficient. External diaphragms or cross frames must be backed up with an internal diaphragm or cross frame.

2.4.6. Internal Diaphragms and Cross Frames (between Piers)

Internal diaphragms and cross frames are used to control cross-section distortion. For curved tub girders, locate an internal cross frame or diaphragm at every other lateral bracing point, which should result in a spacing of 14 to 18 feet. Place horizontal struts, usually angle sections, at the lateral brace point between internal cross frames to control horizontal bending of the flange during concrete placement. Like lateral bracing, they should be attached directly to the flanges.

For straight tub girders, internal cross-frame or diaphragms can be spaced every third or fourth lateral bracing point.

2.4.7. Pier Diaphragms and Cross Frames

Assuming one bearing per girder, diaphragms at bents should be plate girder sections that are approximately the same depth as the girders themselves, and they should connect to the tub girder’s flanges and webs if their span-to-depth ratio is 3 or more. With two bearings per girder, a cross frame may be a better choice at piers. Verify that bearing assemblies do not interfere at the bottom flange connection. Carefully consider constructibility of diaphragms at the girder ends as the presence of abutment backwalls, stems of inverted-T bents, or other girders can complicate bolting these diaphragms in place. Provide an inspector access hole through the diaphragm web plate at intermediate supports. See Section 2.5.3 for recommended opening sizes. See Figure 2-4 for a sketch of a typical pier diaphragm between girders (with one bearing per girder).

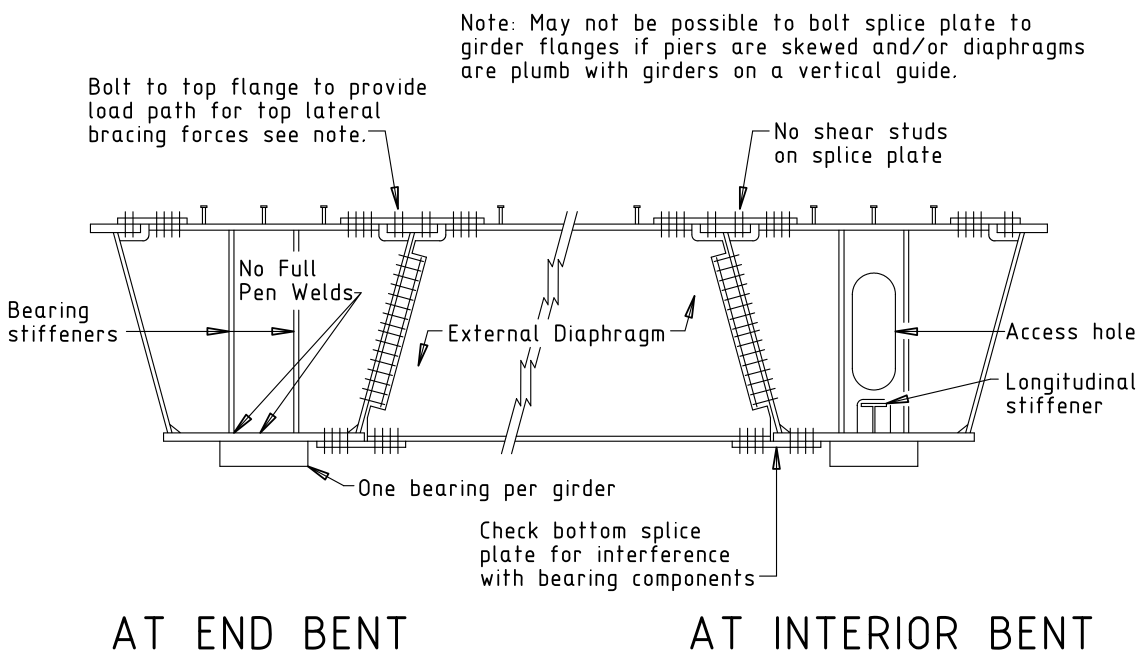

Figure 2-4. Moment-Connected Pier Diaphragm

2.4.8. Field Splices

Bolt field splices. The suggestions in Section 2.2.10 apply. In addition, overall girder width, including sweep, should be no more than 14 feet for ease of shipping.

2.4.9. Bearings

TxDOT prefers one bearing per girder at each support. A girder may not bear evenly on both bearings or, at worst, on only one of the bearings with two bearings per support. This is especially true with skewed piers.

High Load Multi-Rotational (HLMR) bearings may be necessary. TxDOT prefers neoprene bearings over HLMR bearings. HLMR bearings are good only with very large reactions, more than 1,200 kips.

TxDOT prefers bearing designs without anchor bolts (except anchor bolts through masonry plates only). Do not require anchor bolts to pass through the girder flange. A better method of detailing for restraint is to design an external alignment device that is flexible in terms of placement after girder erection.

Bearing designs should accommodate bearing replacement with a minimal amount of lifting.

2.4.10. Electrical Service and Inspection Access

Design details should provide for electrical service on the inside of the girders, with outlets spaced at no more than 100 feet to facilitate maintenance and inspection during the life of the bridge. The long girder length between access holes or doors necessitates this provision.

Provide an access hole with lockable door or cover in the bottom flange near each end support for inspector access. The door or cover must be light enough to be easily managed by an inspector (suggested weight is 25 pounds or less).

2.5. Box Girder Sections (Closed Boxes for Straddle Bents)

Straddle bents are sometimes employed when there is limited vertical clearance between a bent and lower roadway. Steel box girders are a good solution in these situations.

Longitudinal beams/girders should be supported on top of the straddle bent, but they may be supported on the sides of the bent if necessary to satisfy vertical clearance requirements. TxDOT standard drawing MEBR(S), “Minimum Erection and Bracing Requirements (Steel I-Beams and Plate Girders),” does not apply to straddle bents, and the designer should address beam/girder bracing requirements through special details or notes.

Provide two inspector access doors or hatches in all box girders. Typically, a hatch-type, lockable door at each end of the box is sufficient. It is important to provide access holes at each end of the box.

Boxes can be designed to be completely sealed (preferred) or to be well drained. Coordinate all aspects of box design toward one or the other of these designs. For example, a design that calls for a sealed door should not have drain holes or corner clips on outside plates. Remember that small openings into the box occur at bolted field splices if a sealed box design is being considered. Keep any openings as small as possible or install screens to keep birds and bats out of the boxes as they can plug the drains.

Recommendations for plate girders generally apply to straddle bent box girders.

2.5.1. Flanges

Tension flanges for straddle bents are fracture-critical.

Extend flanges past the outside edge of each web a minimum of 2 inches to allow for automated welding equipment.

Flange width depends somewhat on the need for enough room inside the box girder to allow passage of inspection personnel. If fabrication is required within the box, provide at least 48 inches between webs.

See Section 2.2.1 for more details.

2.5.2. Webs

When estimating web depth for a straddle bent, use the span length divided by 12 (L/12) to obtain a reasonable starting point for design.

The tension half of each web plate is fracture-critical, so take care to avoid weld details on the web that are more critical than Category C '. Also, if a detail welded to a fracture-critical member is long enough, it becomes fracture-critical itself (see AWS D1.5, Section 12.2).

2.5.3. Stiffeners

Stiffeners, both intermediate and bearing, should consist of plates sized to match the box's interior dimension with an opening that is sufficiently large to permit fabrication and inspection functions within the box. The absolute minimum opening size is 18 inches wide by 30 inches deep. If possible, provide openings that are at least 32 inches wide by 36 inches deep. Place the holes at mid-depth and concentric with the box. The opening corners should have a radius equal to one half of the opening width.

If multiple bearing stiffeners are required at a bearing, space them far enough apart to provide adequate welding access. Although not always possible, stiffeners should be spaced 36 inches apart to facilitate welder access.

Stiffeners should be welded to all four inside box surfaces. Avoid a tight fit condition without a weld because it can create an unstiffened web gap from which fatigue cracks can propagate. Avoid complete penetration groove welding because of weld-induced distortion that invariably occurs in the flanges.

See Figure 2-5 for a sketch of a typical box girder stiffener

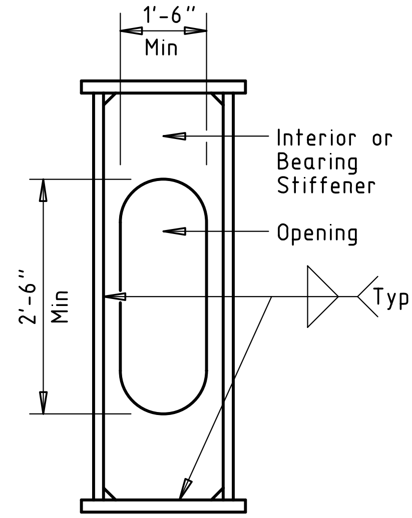

Figure 2-5. Recommended Box Girder Stiffener Detail

2.5.4. Bearings

A preformed fabric pad is the type of bearing most often employed with straddle bents. Base pad thickness primarily on rotational capacity requirements set forth in the *AASHTO LRFD Bridge Design Specifications*. Reinforced neoprene bearings are a good alternative if rotation is difficult to satisfy with preformed fabric pads.

If the straddle bent is not level, use a sole plate, beveled to match the slope of the cap.

If feasible, use a bearing system that is forgiving of anchor bolt misplacement.

2.5.5. Field Splices

Field splices, if needed, should be bolted. See Sections 2.2.10 and 2.7for more information related to bolted field splices.

2.5.6. Flange-to-Web Welding

For flange-to-web welds on box girders, designers should provide options. These may include the following:

- Double fillet welds
- Double unbalanced fillet welds
- A full penetration weld
- A partial penetration weld with fillet backing

These options are shown in detail in Figure 2-. Allowing these options provides flexibility to the fabricator, which helps to ensure the most economical product. The choice of joint detail for a box girder corner has a great effect on quality and economy. Access inside the boxes is generally limited, especially in smaller boxes, and it is usually best to minimize the amount of welding that must be accomplished from inside the box. Fillet welds are generally more economical than groove welds, so fillet welds are encouraged. Additionally, large welds can cause distortion problems during fabrication.

One flange can be welded to the webs with relative ease. The need for options becomes more pronounced for the remaining flange, sometimes referred to as the “lid.” Detail the tension flange to web welds as double fillet welds, and provide options for the compression flange. This assumes that the tension flange changes at a field splice or that the box is simply supported. If the tension flange changes sides without a field splice, the weld detail should remain constant per side.

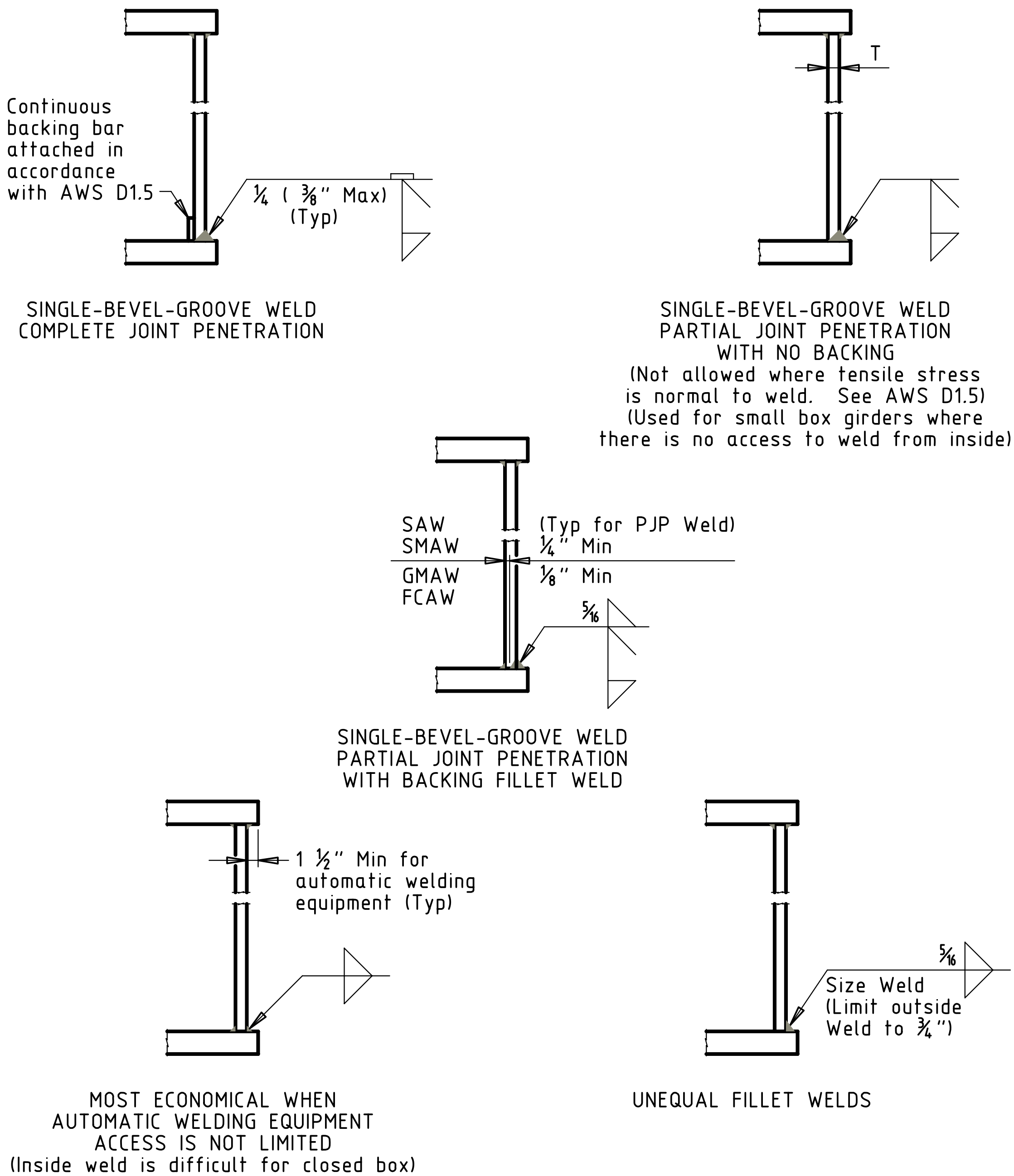

Figure 2-6. Recommended Box Girder T-Joint Weld Details

2.6. Diaphragms and Cross Frames

TxDOT has traditionally used field welding as the preferred method of connecting diaphragms and cross frames to girders because it is more forgiving with respect to erection tolerances than bolted connections. For bolted connections, use standard size holes to control girder geometry.

The diaphragms and cross frames shown on TxDOT standard drawing SGMD should be acceptable for the beam spacing and depth limits noted on those standards. However, because of the variability in steel bridges, always confirm the adequacy of these standard diaphragms and cross frames, including their connections, before using them with your design details.

For straight beams and girders, observe the 25-foot spacing limit from the AASHTO *Standard Specifications* as a starting point. A larger spacing is acceptable if a larger spacing can be achieved without temporary bracing and if all other limit states are satisfied. A tighter-than-normal diaphragm/cross-frame spacing near interior supports may prove beneficial in terms of increasing negative moment bending capacity. This should be investigated for each straight continuous bridge design. For curved girders, TxDOT prefers that diaphragms or cross frames be placed at 15 to 20 feet maximum to help limit lateral flange bending stresses and cross-frame/diaphragm member forces.

You may need to consider bracing beyond what permanent diaphragms and cross frames provide for erection and slab placement. In some cases fascia beams have twisted during slab placement, a problem that permanent diaphragms have not prevented.

2.6.1. Member Selection

Equal leg angles are often more cost-effective than unequal leg angles. Fabricators discourage back-to-back angles used as cross-frame members. Some common angle sizes for diaphragms are L3.5 x 3.5 x 3/8, L4 x 4 x 3/8, and L5 x 5 x 1/2.

Fabricators discourage the use of WT shapes, especially in small quantities. If channel sections are used, C shapes are preferable to MC shapes.

If you use a channel, provide an option for the fabricator to bend a plate into an equivalent channel shape instead. A bent-plate diaphragm, in the shape of a channel, is a possible option to provide to the fabricator for diaphragms on shallow plate girders (4-foot-deep web or less) or rolled beams.

It may be beneficial to use a larger shape than is required if the larger shape is being used in significant quantities elsewhere in the project.

Design and detail cross frames such that they can be erected as a single unit. Fabricators and erectors discourage diaphragms that require erection in separate pieces.

Detail cross frames such that all welding during fabrication can be done from one side to minimize handling costs.

2.6.2. Stage Construction and Skews

If the bridge is to be built in stages or if the skew is large, differential deflection between girders due to slab placement can be significant. Special diaphragm/cross-frame details may be required for these cases. An example is slotted holes for erection bolts and the requirement of field welding after slab placement if the designer has ensured that erection bolts alone can accommodate the loads.

TxDOT standard drawing SGMD covers skew angles up to 45 degrees. Anything beyond this requires special design details showing diaphragm/cross-frame attachment to the girders.

2.6.3. Diaphragm and Cross-Frame Orientation

Standard drawing SGMD indicates that diaphragm/cross-frame lines at end bearings are parallel to the skew up to a 20-degree skew. Between 20- and 45-degree skews, diaphragm/cross-frame lines at end bearings are not quite parallel to the centerline of bearing.

AASHTO permits interior diaphragm/cross-frame lines parallel to the skew up to 20-degree skews. When diaphragms/cross frames are placed along the skew, the designer should be aware that Dart welders are commonly used industry-wide to attach the stiffeners to the girder webs. Dart welders can weld a stiffener plate that is skewed to the web up to 20 degrees, which works well with the AASHTO limitation. Beyond 20 degrees, fabricators will have to use a more costly welding method.

Placing all diaphragms/cross frames along the skew is acceptable for skews up to 20 degrees. All other diaphragms/cross frames should be normal to the girders. Curved girders are an exception and should always have radial diaphragm/cross-frame lines at intermediate locations.

A good, economical design minimizes the number of diaphragms/cross frames with different geometry. Superelevation changes, vertical curves, different connection plate widths, and flaring girders all work against this goal.

2.7. Bolted Connections

One-inch, 7/8-inch, and 3/4-inch diameter bolts should be the only sizes considered for bridges. One-inch bolts often provide the most economical design. However, for small rolled beam flanges, smaller bolts may be better due to net area requirements. Do not mix sizes within a splice or within a unit unless A490 bolts are used with A325 bolts. (See Section 2.7.2.)

Web and flange splice plates should be at least 1/2 inch thick.

Provide more edge distance for bolt holes than the AASHTO minimums. If the drill drifts during the drilling operation, the hole could violate minimum edge distances. Add 1/4 inch to the AASHTO minimums.

2.7.1. Slip Coefficient

The 2004 TxDOT Standard Specifications allow painted faying surfaces if the paint is documented to meet slip and creep requirements, so show the slip coefficient assumed in the design on the design detail drawings.

TxDOT recommends using Class A surface conditions (slip coefficient = 0.33) for design for the following reasons:

- It allows for surface deterioration before the splice is made.
- Slip might not control the design, and this information is not normally conveyed in the plans. If a 0.33 design slip coefficient is adequate, it permits more flexibility to fabricators in the coating types to be allowed on faying surfaces.

For unpainted weathering steel structures, faying surfaces must be blast cleaned and be free of any mill scale. Research attests that mill scale on weathering steel plate is more slippery than mill scale on non-weathering steel plate, and it is detrimental to the slip resistance of connections. The Standard Specifications call for an SSPC-SP 10 (“near-white”) blast-cleaning to ensure that all mill scale is removed, but the near-white finish is not required to remain at the time of erection. See further discussion under Section 4.2.

2.7.2. A325 vs. A490 Bolts

For the following reasons, do not use A490 bolts unless absolutely necessary:

- TxDOT’s bolt installation procedure (in TxDOT Standard Specification Item 447) specifies the use of fit-up bolts, which are used to bring all the plies into full contact. The erector is often able to release the crane from the member using these bolts to support the joint as part of the erection procedure. Before the joint is complete, these fit-up bolts must be loosened. They can be retightened if A325 bolts are used but must be replaced if A490 bolts are used. Contractors strongly prefer loosening and retightening to replacement, and TxDOT inspection procedures cannot ensure that the A490 bolts will be replaced.
- A490 bolts are much more sensitive to tightening procedures. If over-tightened, these bolts can unload significantly below their proof load. A325 bolts have much more ductile behavior, so they can be tightened well beyond their proof load and still maintain the required tension.
- A490 bolts require impact wrenches of ample strength and quality that are sometimes not available at construction sites.

A325 bolts and A490 bolts of the same diameter are easily confused. If A490 bolts are necessary, use them in all similar connections or make them a different diameter than A325 bolts used for the bridge. But use common sense—for example, if A490 bolts are required for some lateral bracing connections, use the same size A490 bolts for all lateral bracing connections. Switching bolt grade/size at a common field splice location for adjacent girders would require re-calibration of the tightening equipment in the middle of the erection process, or require separate wrenches calibrated for each bolt grade/size. A good design is not a source of confusion and delay in the field.

2.8. Anchor Bolts and Rods

The most economical anchor bolt for bridges is a mild steel anchor bolt. Mild steel anchor bolts are usually sufficient. Alloy steel anchor bolts may not provide the best value in the long term because their superior engineering properties are not realized on typical bridges.

Anchor bolts should be hot-dip galvanized as specified in the TxDOT Standard Specifications. Do not be concerned about contact between galvanized bolts and weathering steel. The zinc coating resulting from the hot-dip process is thick and sacrifices itself at such a low rate that the service life of the bolt is not compromised.

Use fatigue detail Category E when evaluating anchor bolts of any material for fatigue. Use the bolt tensile area, not the nominal area, when evaluating anchor bolt fatigue.

Despite the best efforts of contractors, anchor bolts are occasionally placed in the wrong location. Any bearing detail incorporating anchor bolts or rods should be able to accommodate their misplacement.

2.9. Stud Connectors

TxDOT has traditionally used stud connectors in only the dead load positive moment regions of plate girders and rolled beams. Studs applied full length of the girders may provide better distribution of deck cracks resulting from shrinkage. If you are placing studs along the full length of girder, do not consider slab reinforcement as part of the negative bending section. Tub girders require stud connectors the full length of the girder to ensure that the box section is “closed” along its entire length.

TxDOT standard drawing SGMD provides details for stud connectors. The designer must show their spacing on girder/beam elevations in the design details. AASHTO requires a minimum center-to-center stud spacing of four stud diameters transversely and six stud diameters longitudinally. Recent research sponsored by TxDOT indicates the longitudinal spacing could be lower as long as concrete consolidation can be achieved. Therefore, a longitudinal stud spacing of 4 inches is acceptable.

You must modify stud connector spacing when allowing the use of prestressed concrete panels as a slab-forming option. To verify that panels will fit with the studs, assume a minimum clearance between panel edges and stud connectors of 5/8-inch, the minimum clearance used with prestressed concrete beam horizontal shear reinforcement.

Designers may present a stud connector spacing modification in the design details other than what is given on the SGMD standard drawing.

Shear connector studs should not be required on top of flange splice plates.

2.10. Design Details

Both web camber diagrams and total dead load deflections should present camber requirements for plate girders. Most fabricators use total dead load deflections in determining web camber. However, some fabricators may opt to use a web camber diagram.

Do not provide web camber diagrams for tub girders because they supply no useful information to the shop drawing detailer. Steel detailers require only total dead load deflections to determine tub girder camber.

Avoid the term “web-cutting diagram” in design details because webs are not actually cut to these diagrams. Rather, fabricators make the adjustments they predict are needed such that the final product will meet specification requirements. Instead use phrases such as “camber diagrams” or “camber in the unstressed condition.”

See the current TxDOT *Bridge Detailing Manual* for more information on requirements for detailing of steel structures.

2.11. Bearing Replacement

The potential need to raise a bridge at some point in its life always exists because of unforeseen circumstances, such as bearing deterioration or failure.

Bearing designs should accommodate bearing removal with minimal lifting (1/4-inch lift requirement is suggested).

Raising a bridge with jacks is typical during bearing replacement. Jacking points on girders should be underneath a stiffened web. On tub girders, where it would be extremely difficult to add stiffeners to the girder webs once the slab is in place, include stiffeners for jacking in the initial girder fabrication. If there is enough room on the bent cap and under the girder to accommodate the predicted jack size, place the stiffeners accordingly. If the bent cap is too narrow to accommodate the jacks or the clearance under the girder is too small, place the stiffeners approximately 1 foot beyond each face of the cap. In this situation, shoring towers will be required in conjunction with jacks to raise the bridge. Place stiffeners symmetrically about the girder and bearing centerline so that no torsion will be induced during lifting operations.

On conventional I girders where it would be relatively easy to add stiffeners at jack locations, do not include these stiffeners in the initial girder fabrication. If the bearings need to be replaced, the absence of stiffeners gives the contractor more latitude in determining the best jack locations.

2.12. Bent Locations for Replacement or Widening

If existing bents are used for replacement or widening, use field-verified bent locations to prepare the plans. Do not rely on existing plans because the actual bent location/skew may differ enough from those plans to create field fit-up problems.

3. Fabrication

3.1. Shop Drawings

3.1.1. Shop Drawing Review

You need not check the welding procedures when reviewing shop drawings; however, you must check the welding symbol.

When stamping drawings, a reviewer need not initial the drawings. The approval engineer is responsible for reviewing shop detail drawings for conformation with the design details and specifications only, and the contractor and fabricator are responsible for all dimensions and fit of the structure.

See AASHTO/NSBA Steel Bridge Collaboration G1.1, "Shop Detail Drawing Review/Approval Guidelines," for dimensions and material requirements that need to be checked and for other guidelines concerning shop drawing review.

3.1.2. Distribution of Approved Shop Drawings

The design office—which is the district design office, the Bridge Division, or a contracted design firm—handles distribution of approved shop drawings. Requirements for shop drawing submittals are outlined in a table titled "2004 Construction Specification Required Shop/Working Drawing Submittals," posted on the internet at http://www.dot.state.tx.us/publications/bridge/items_reviewed.pdf. Additional information is available in the *Guide to Electronic Shop Drawing Submittal* and other information posted on the internet at http://www.dot.state.tx.us/brg/FO/Fabrication.htm.

3.1.3. Shop Camber Checking

TxDOT inspectors check the actual camber condition during laydown. In accordance with the AWS D1.5 Bridge Welding Code, they check camber over the entire span, not just on the individual member.

3.2. Non-Destructive Testing

3.2.1. Use of Edge Blocks for Radiographs

When radiography testing is conducted at width transition splices, the location of the transition is moved 3 inches back from the splice to allow proper fit of the edge blocks. See Figure 3-1. This detail is shown on TxDOT standard drawing SGMD.

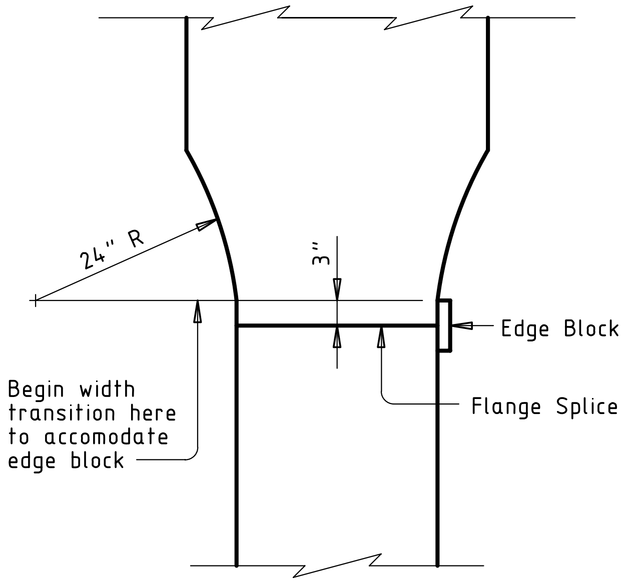

Figure 3-1. Flange Width Transition Detail

3.3. Cleaning and Painting

3.3.1. Painting Box and Tub Girder Interiors

The 2004 TxDOT Standard Specifications Sections 446.2.B and 446.4.F.3.d specify the type of paint (a white polyamide-cured epoxy) to be used on the interior of box girders.

Although paint on the inside of boxes provides some protection from corrosion, its primary purpose is to facilitate in-service inspections. The paint is intended to be “surface tolerant” so that it will adhere well to surfaces that have been cleaned free of grease, oil, and dust but that have not been blast-cleaned to near-white metal.

For the interior paint, the 2004 TxDOT Standard Specifications require a dry film thickness (DFT) of 2–3 mils, the minimum to ensure adequate coverage and the maximum to prevent masking of cracks.

3.3.2. Painting of Faying Surfaces

Faying surfaces may either be blast-cleaned and left bare, or painted with a prime coat after blast cleaning. Painting them might provide some corrosion protection. However, if the splice components are thin enough, bolt tightening will pull the plates together so that there will be no moisture access. Painting the faying surfaces also helps protect them during the time between when they are prepared in the shop and when they are erected in the field. The 2004 Standard Specifications leave the choice of painted or unpainted faying surfaces up to the fabricator, although designers may specify painted surfaces if they find this necessary in a particularly corrosive environment. If you choose to have the faying surfaces painted, ensure the following:

• The coating has been evaluated for actual coefficient of friction values.

• The coating has been evaluated for creep.

• The conditions under which the evaluation took place were representative of conditions under which the coating will actually be applied. If not, the manufacturer of the paint must be consulted regarding appropriate cure time.

The friction test and the creep test are described in the Research Council on Structural Connections (RCSC) *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, which can be found in the AISC Manual of Steel Construction. Paint suppliers provide specific coefficient of friction values for each coating, and they certify that the coating meets AISC requirements.

If a steel structure with bolted connections will be painted but the faying surfaces will not, a small amount of paint in the faying surfaces around the perimeter of the connection is not detrimental. Although it is very important to avoid even small amounts of overspray on the faying surfaces of the connection, paint may be applied on the faying surfaces to within one inch or one bolt diameter, whichever is greater, of the perimeter holes (see Figure 3-2). This allows flexibility in fabrication because the painter may mask inside the edge of the connection instead of trying to mask precisely where the edge of the connection is expected to be. This also provides better protection because carrying the prime coat slightly under the splice plate minimizes the discontinuity in the primer that naturally occurs between splice plates and main members.

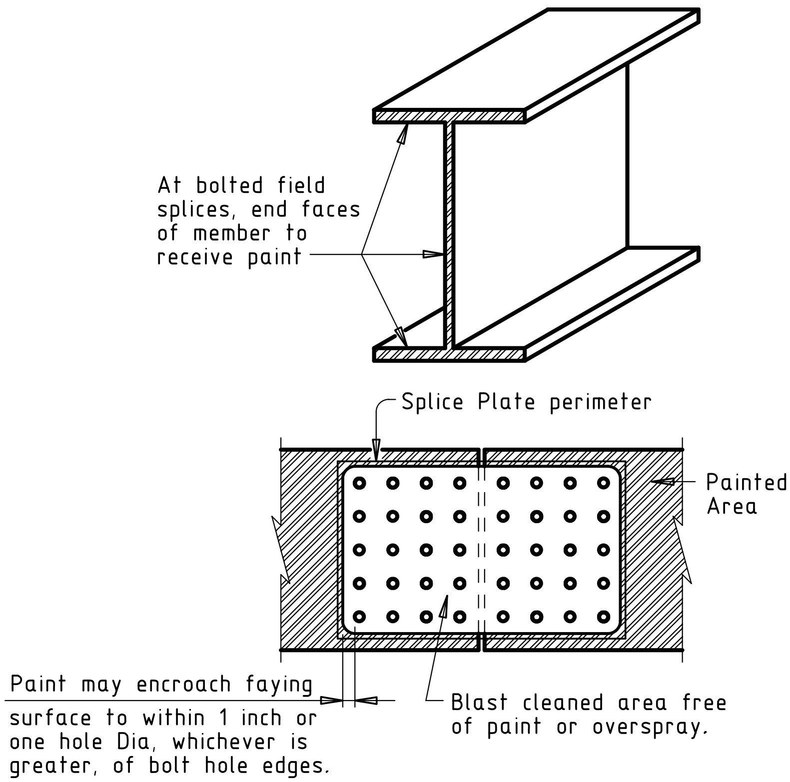

Figure 3-2. Paint Masking Requirements for Bolted Splices

4. Erection/Construction

4.1. Shipment of Bolts

Bolts for steel bridge structures are usually installed in the field although occasionally some are installed in the shop. When bolts will be installed in the field, they should be shipped directly to the field in containers that prevent exposure to the elements and maintain the integrity of the fasteners. All fasteners must be tightened in accordance with specification requirements for tightening. Diaphragm or cross-frame erection bolts, shown on TxDOT standard drawings SGMD and SBMD, are an exception to this rule and need only to be snug tight because they are used only as pins to align the diaphragms/cross frames prior to field welding the final connections.

The actual fasteners to be used on the job should never be shipped installed in the members unless they are completely tightened. This is important for two reasons:

- When fasteners are shipped to the field partially tightened, they are exposed for indefinite lengths of time, compromising the lubricant condition of the threads and washers.
- When fasteners are shipped to the field partially tightened (for example, snug-tight), it is not possible to know just how tight they are. This makes it impossible to tighten the fasteners properly.

4.2. Condition of Weathering Steel Bolted Splice Faying Surfaces

The Standard Specifications call for an SSPC-SP 10 (“near-white”) blast-cleaning to ensure that all mill scale is removed, but the near-white finish is not required to remain at the time of erection. In fact, research has shown that some amount of tightly adhering rust can enhance the slip resistance of a connection, and an SSPC-SP 6 finish is sufficient. Unless the girder has been exposed to particularly corrosive conditions during storage, pressure-washing the faying surfaces at the jobsite shortly before erection will remove any loose material detrimental to the connection.

Appendix A

Frequently Asked Questions about Paint

Frequently Asked Questions about Paint

PAINT

What is paint?

Paint consists of three components (pigment, resin, and solvent) that work together to form a functional film that adheres to the substrate to which it is applied.

The resin, or binder, is the component that forms the paint film and acts as an adhesive to glue the pigments together and the film to the surface. Resins are long stringy molecules that wrap around the pigments and entangle with each other like spaghetti on a plate. Paints are typically named after the type of resin used, like epoxy, acrylic, urethane, etc. Each of these resin chemistries have properties that are suitable to different uses.

The pigment provides strength to the paint film as well as color. Some pigments provide corrosion resistance for steel substrates.

The third component is the solvent or carrier. Solvents exist to make the paint liquid during the manufacturing process and for application. They then evaporate leaving a film consisting of the pigment held together by the resin.

RESINS – Lacquers
Resins set up as a film by several mechanisms. The first type (typically called "lacquers") is where the paint forms a film from the solvent evaporation. The resin molecules entangle as the solvent evaporates, like strings of spaghetti. Since there is no chemical bonding of the resin molecules, these paints can be redissolved at any time in the future. An example of a lacquer is the 742 appearance coating used on bridges. The latex or water-borne paints that TxDOT uses are in this category.

Latexes: Latexes, or emulsified paints, are those where the resin or binder is dispersed (not dissolved) in water. When the water evaporates the resin particles are forced together into a film. This film, once dried, cannot be redissolved by water. Acrylics (water-borne latex type) do not get brittle with age and are resistant to sunlight so they are used as steel appearance coatings and for painting concrete. Vinyl-acrylic and vinyl latexes degrade in high alkalinity like that in concrete, so they do not work well as a coating for concrete.

RESINS – Oxidizing Types
When these paint films form, they dry initially like lacquers through solvent evaporationAfter the solvent evaporates, final cure occurs when the resin reacts

with oxygen from the air to crosslink. Crosslinking is where the resin molecules chemically tie together and can no longer move past each other elastically, making the film hard and maybe brittle. This crosslinking also increases the chemical durability. This is why oil paints on wood start to crack and peel after a few years. As the wood expands and contracts with moisture, the paint can no longer stretch and it splits. On our bridges, after about 15 years the paint that is exposed to the sun gets brittle and peels.

Examples of oxidizing paints include alkyds and those made from oils, like linseed oil.

<u>RESINS – Moisture-cured</u>

<u>Inorganic Zinc Primers (IOZ)</u>: The predominant moisture-cured coating that TxDOT uses is the inorganic zinc primer. This silicate based resin has an organic (molecules made from carbon and hydrogen) tag on the ends of each molecule that allows it to dissolve in the solvent. When the paint forms a film, it absorbs moisture from the air which displaces the organic tag leaving only the inorganic (silicate) portion of the molecule, thus the name. This coating is somewhat porous, so that the out-gassing of the ethyl alcohol group (the organic tag) does not cause any blistering.

<u>Moisture-cured Urethanes</u>: Another moisture cured system is the moisture-cured urethanes (MCU.) Like IOZ's, when these paints form a film after the solvent evaporates, they absorb water to cure. If the film is too thick or the humidity is too high, they can start to dry from the outside and possibly trap solvent inside the film. When the paint sets up and the trapped solvent migrates out, it leaves voids that can be channels for water to penetrate into the film. Also, these coatings give off carbon dioxide during the curing process. If the film caps over too much, the carbon dioxide can cause foaming, resulting in porosity. The zinc-rich MCU's are not as susceptible to these problems, since they are more porous. This porosity allows the residual solvents and the carbon dioxide to escape.

<u>RESINS – Two Components</u>

<u>Two Package Coatings</u>: The last category of resin systems is the two-package (or 2K, as a shorthand notation) that consist of a base component (the pigmented part) and a curing agent. These two components will react with each other and do not need any outside chemistry to cure. It is very important that these paints are mixed in the correct proportions or they will not cure properly and form a fully protective coating.

Once reacted, the paint film cannot be redissolved with solvents. The paint film can also get so hard that the subsequent coats of paint might not adhere well, causing them to peel. The time between when the first coat is dry enough to be painted over and when it has hardened beyond the point where the second coat will not adhere is called the recoat window.

Epoxies are the workhorse of the 2K coatings used on structural steel. They have very good chemical resistance and adhesion to steel, so they are used as primers on steel structures. They degrade when exposed to sunlight. For this is the reason we use appearance coatings to protect the epoxy primer.

Urethanes: Polyurethanes (or more abbreviated, urethanes) are excellent appearance coatings for bridges. They do not have good adhesion directly to steel nor do they make good primers. They adhere well to the epoxy primers and do not degrade in sunlight, hence their use as appearance coatings. To use them as the appearance coats on galvanized steel, the urethanes need an epoxy as an intermediate coat. They will adhere to the epoxy but not the zinc of the galvanizing.

Because urethanes are 2K paints, the crosslinking from curing does not allow them to redissolve if they are washed with solvents. They can function as Departmental Material Specification DMS-8111 Type II Anti-Graffiti paints. These are the permanent types that can withstand the graffiti being washed off with chemical cleaners without damaging the urethane paint. Epoxies are not recommended for use as anti-graffiti paint since they will degrade in sunlight.

PIGMENTS

Pigments are in the paint film to add color and structural integrity or film strength. They also function in most primers to resist corrosion.

In the category of color, there are mineral and organic pigments.

Mineral Pigments: The mineral pigments, those derived from metallic ores, are typically duller in color than the organics but are usually more light-fast or durable. Some of these mineral pigments like red iron oxide (purified rust) do not degrade because they are already in their most stable state. (This is the form of iron that is found in nature and the form to which iron wants to return when it corrodes.) This is the color of barn red.

Organic Pigments: Organic pigments are comprised of organic molecules. These pigments are chemically strained within the molecule; it is this strain that creates the color. The molecule is ready to release the strain, which will return it to a molecule that has no color. That is why organic pigments are susceptible to fading from the UV in sunlight. Organic reds are bright colors, like fire engine red. Within the chemistries of red pigments, there are some that are much worse than others for fading and loosing brightness. Bright reds and greens are the worst organic pigments for fading. In general, the brighter the color, the less light-fast it is and the more likely it will fade and look dingy in a short time.

Anti-Corrosion Pigments

Barriers: Barrier pigments are those that resist corrosion by creating a dense film preventing water from penetrating to the steel. A mix of prime (color) and extender (those that provide physical structure but not color or opacity) pigments are used together to create this barrier effect. These pigments do not have any active anti-corrosion properties. If these primers sustain any damage, there is nothing in the film to prevent corrosion. Micaceous iron oxide is an extender pigment often used in barrier coatings since it is a very flat, planar pigment. This pigment will stack on top of each other to create a dense matrix, thus increasing the travel length moisture would have to follow to reach the steel surface. Aluminum flake is another pigment often used in barrier coatings.

<u>Inhibitive Pigments</u>:I Inhibitive pigments dissolve in water as moisture migrates through the film. When they get to the paint and steel interface, they inhibit the corrosion process. The mechanism for this rust inhibition is not clearly understood. This category of primers is not as effective since the red lead and chromates have been taken out for environmental reasons. The red lead and chromate inhibitive pigments were much more effective than those available now.

<u>Sacrificial Pigments</u>: For iron to corrode it must give up two or three electrons before the iron atom can take on the oxygen atoms of the corrosion process. Based on the Electromotive Force series, zinc functions as a sacrificial pigment because it prefers to corrode more than the iron. The corroding zinc gives electrons to the iron before it takes up oxygen and the iron atom stays neutral. It is important that the steel surface be clean when applying the zinc as a paint or galvanizing so that there are no contaminants insulate the zinc electrons from the iron atoms.

The best method for providing the zinc on the steel is to galvanize it. When steel is dipped into molten zinc (~850ºF,) the zinc and the steel melt together forming an alloyed juncture at the interface. The layer of zinc functions as an excellent barrier because it is impervious to air and moisture. With time the zinc corrodes to protect the steel. Since galvanizing is not always practical, the next best thing is to glue zinc to the steel. This is done with powdered zinc dust in a paint film. These zinc-rich primers can be either organic (like Item 446 System II) or inorganic (Systems III & IV.)

<u>SOLVENTS</u>

Solvents are the portion of the paint that makes it fluid for manufacturing and application. These materials come in a variety of different chemistries. They are usually defined as slow, medium, or fast evaporating types.

If a paint is made with all fast evaporating solvents, it might dry so fast that as it is sprayed the paint droplets are dry by the time they reach the steel surface. This condition is called “dry spray.” The paint then is too dry for the droplets to flow together as a film or melt onto the surface and develop adhesion.

Paint usually has a mix of fast, medium, and slow solvents. Fast solvents make the paint fluid for spraying but leave quickly. Medium solvents help with this fluidity and they keep the paint droplets wet while they are in the air between the spray gun and the substrate. The slow solvents are usually used in small amounts to aid in the last flow out of the droplets and wetting of the surface as the paint sets up.

The other chemistry involved with solvents concerns which materials that they will dissolve. Ketones will dissolve epoxies and urethanes well but mineral spirits will not. Mineral spirits dissolves oil paint resins (linseed, soya, alkyds, etc.) but the ketones do not.

Then there is a class, called diluents, that is somewhat in the middle. These can be used to mix into paints that are already dispersed in a true solvent. They thin the paint but they cannot dissolve the resin by themselves. These are usually cheaper and can help with the dry time. Aromatics, like toluene, xylene, and aromatic 150, are considered diluents for epoxy paints.

It is essential that the correct solvent be chosen from those listed on the technical data sheet for the paint. In one instance, a painter used the aromatic 150 (a very slow evaporating solvent that stopped dry spray problems) in an epoxy zinc primer. (It had been recommended for a different zinc-rich primer [IOZ] that he had been spraying the same day.) It mixed in well and helped with the dry spray problem but a couple hours later it damaged the paint film. The 150, a diluent, was the slowest evaporating solvent used and all of the true solvent evaporated first. Since the 150 was not compatible with the drying epoxy, it caused the paint film to form craters or dimples in the film. The coating was not ruined but the craters or dimples were thin spots that needed to be repaired.

<u>Water</u>: The other carrier of paint components is water in latex paints. Water is not a solvent because the components are not dissolved init. (If they were dissolved in water, the paint film would wash off in the first rain.) The resin and pigments are floating in the water, in a dispersed state, and when the paint dries, these components melt together to form the paint film.

<u>What is a “wet application” of the paint?</u>

This is where the paint film, immediately after the painter has made a spray pass with his spray gun, has a glossy or shiny appearance because it still contains solvent. Even if the paint dries to a flat or non-glossy finish, it should be glossy until the solvent evaporates. This means the paint was fully liquid when it contacted the surface. The droplets will melt together properly and flow into any porosity or roughness to gain adhesion.

If the paint film is flat immediately after the painter has applied it, then it was probably dry sprayed. As mentioned above, when this happens, the paint droplets were too dry when they got to the surface and they cannot flow into the roughness of the surface or melt together to form a durable film. This situation can be remedied by making a slower pass with the spraygun to apply more paint, moving the spraygun closer to the surface to reduce the drying distance, moving the spraygun closer to the surface to reduce the drying distance, using a larger spraygun tip to apply more paint, or by adding a slower evaporating solvent.

STEEL PAINT

What do primers do?

Primers are the primary or first line of defense for protecting steel from rusting. The inhibitive type primers work by having pigments that inhibit the corrosion process. The sacrificial type primers contain zinc metal which sacrifices itself to protect the steel.

Why do we need an appearance coat?

The most important reason for the existence of the appearance coating (sometimes called a topcoat or finish coat) is to protect the primer. Epoxy primers are very chemically resistant but degrade in sunlight. By putting the topcoat or appearance coat over them, the sunlight cannot degrade them. Other primers may not be as susceptible to ultraviolet degradation from sunlight as are the epoxies but the appearance coats can still keep some of the water, dirt, grunge, pigeon deposits, etc. from degrading the primer.

Another major function of the appearance coating is to provide a uniform color and appearance. Zinc-rich primers do not come in a wide variety of colors. They are typically only in these three colors, greenish gray, reddish gray, and gray. If any other color is desired, the appearance coat is needed to provide this color.

How long can an epoxy zinc primer be exposed to sunlight before it starts to degrade?

This is a hard question to give a definite answer. The epoxy begins degrading immediately but there is probably no reason for concern for a year or so. It is important to evaluate the condition of the primer before applying the appearance coating. If the epoxy has a chalky surface (when you wipe it with your hand and a powder is left on the fingers,) this needs to be repaired. Usually a good power washing will remove the chalk but it could need even more, like scrubbing with soap and water. It is important to provide a really good rinsing after the scrubbing with soap, so as to not leave any soap on the surface. The residual soap will draw moisture through the appearance coating causing early failure.

The solvent-borne appearance coatings (742 or urethane) will probably wet through a chalky surface better than the acrylic latex. Since we cannot easily control the recoat windows, it is important to prepare the primed steel properly before applying the appearance coating. For the urethane over an epoxy intermediate coat, this could include a light sweep blast to etch the cured hard epoxy. This also might include a light coat of epoxy for the best adhesion.

Is It “Urethane” or “Polyurethane?”

The terms are interchangeable and mean the same thing. Of the three categories of urethanes, we use the two-component acrylic-cured aliphatic urethane. We do not allow the polyester-cured urethanes because they do not hold up to sunlight as well as the acrylic-cured versions. Nor do we use aromatic urethanes since they also do not hold up to sunlight as well as the aliphatic types.

What is an acrylic latex coating?

The term latex has been stolen from the rubber industry and is used for any emulsified resin. (In the rubber industry, the latex is the rubber in the sap that comes from rubber trees.) Latexes or emulsified paints are those where the resin or binder is dispersed in water, not dissolved. If it was dissolved in water, it would wash off when it rains. In these materials the resin is in the form of tiny balls that float around in the water. When the paint is applied, these resin particles float until the water evaporates, at which time they are forced together. The coalescing agent then melts the particles into a film, then evaporates, leaving a paint film that will no longer dissolve in water.

There are all kinds of latex emulsions that are made into paint and they each have their place in the industry. Vinyls, vinyl-acrylics, styrene butadienes, etc. are some of the others. Vinyls and vinyl-acrylics are used for interior coatings because they get brittle and degrade on alkaline surfaces, such as concrete. Styrene containing resins are hard and cheap but they degrade in sunlight.

For this reason, we specify acrylic latexes. Acrylics do not degrade in sunlight or on alkaline surfaces. They stay flexible and can withstand the expansion and contraction of structures well.

What is System I?

System I, also called “Overcoating,” is used when it is not necessary to blast and repaint an existing structure. It relies on the old primer to continue to protect most of the structure. After powerwashing to remove dirt and other grunge, the rusty areas are hand tool cleaned to remove any loose and flaky rust or old paint.

An epoxy penetrating sealer is applied that will soak into the old paint and give it new strength as well as the ability to withstand the solvents in the subsequent paints. The spots that were rusty have no primer remaining and must be spot primed with an inhibitive primer (DMS-8101 Epoxy Intermediate Coating) to stop further rusting. The steel now has the old anti-corrosion primer on most of it and the new inhibitive primer on the bare spots. Then a new appearance coating is applied. This can be a urethane or an acrylic latex.

What is System II?

System II is the TxDOT epoxy zinc primer (810e) with either 742 gray or an acrylic latex appearance coat. The 810e primer is manufactured by several paint companies to meet the formula in DMS-8100 and regardless who makes it, the paint is exactly the same as that made by other companies.

There is an epoxy zinc primer listing in DMS-8101 but recent evidence leads us to question whether all of the approved paints are equal to each other or equal to the System II 810e primer. Tests are under way to determine which are as effective as the 810e.

Epoxy zinc primers are fairly tolerant of application conditions and painter capabilities. That is why they are used for repaint projects as well as for new steel.

What is System III?

It is a shop applied coating system that consists of an inorganic zinc primer, an epoxy intermediate coating, and a polyurethane appearance coat.

What is System IV?

This shop applied system consists of the same inorganic zinc primer used in System III but it has an acrylic latex appearance coating without an intermediate.

Why are Systems III & IV only used in the shop?

Inorganic zinc primers tend to dry spray and it is important for the painter to have easy access to the surface to reduce the time that the paint spray is in the air and drying out. If the paint droplets travel too far through the air, they dry out so much that when they reach the surface, they are too dry to melt together and to adhere properly. These primers also will mudcrack and flake upon drying if they are applied too heavily. This should be easily visible soon after the primer dries to touch and will look like a mud puddle drying in the sun.

In the shop, the steel is setting on the floor and the painter has easy access to all parts of the surface which allows him to properly spray a good wet coat assuring film integrity and adhesion.

In the field, access is often more difficult depending on the rigging used and the configuration of the work area. Sometimes the painters cannot easily get close enough to the surface for proper spray techniques. At these times, an inorganic zinc primer would be more prone to dry spray than an organic zinc primer.

On the repaint structures, the diaframs and other connectors are in place. For the painter to apply paint to all parts of these pieces, he has to make multiple passes with the spraygun. This will cause excessive thickness leading to mudcracking. Sometimes these areas are not easily visible to the inspector and if mudcracking occurs, it is not easily found.

<u>Why have two systems (III & IV) with the same primer?</u>

We have used System III for many years and have not had any serious failure yet but anticipate that it could happen. The primer and the intermediate coats are typically applied in the shop and the appearance coating is applied later in the field after the steel is erected and the deck placed.

Most of the epoxy intermediate coatings have a recoat window, the time where they can be topcoated with the urethane without a problem. After this window has passed, the epoxy has fully cured and become so hard that the urethane has a difficult time gaining the adhesion it needs. This situation is aggravated by the length of time between when the steel is painted in the shop and when the appearance coat is applied in the field. This time can easily stretch to years after the recoat window has passed. The recoat window can vary from a few days to a year and can be different for each company's products. The epoxy intermediate coating from one company can have a completely different recoat window from the epoxy intermediate coating from another company.

After the recoat window has passed, the painter must lightly brush blast the epoxy to etch it without damaging or removing it. Then another coat of epoxy is applied which will adhere to the cured and etched epoxy before applying the urethane appearance coat.

Another situation that can occur is when the intermediate is applied too early over the inorganic zinc primer. This primer needs moisture to cure and if the epoxy is applied before it is fully cured, no moisture can get through and the zinc primer will split in a short time causing the film to blister.

The System IV was added to the specification, in the 2004 Standard Specification Book, because it does not have these limitations. The acrylic latex develops good adhesion to the primer whether it is fully cured or not. Also, if the latex is applied before the inorganic zinc primer is cured, moisture can migrate through the acrylic to allow the primer to cure.

<u>How do I Select between Systems II, III & IV?</u>

For repaint projects, System II or a Special Protective System is to be used. (System I [Overcoating] is used if the structure does not need to be blasted clean.)

In the shop, any of the three can be used. Many people like to specify System II because it is more forgiving for application. A substandard application that is not caught by the shop inspector is less likely to fail prematurely.

The inorganic zinc primer, used in Systems III & IV, is more sensitive to proper application. If it is applied too thick, it will crack and flake off the steel. The concern here is that it might be just at the point where it is cracking but not bad enough that it is obvious.

System III is thought by many in the industry as the best system, if everything is done exactly correct. Some deviation from optimum application can cause the system to be less durable than the epoxy zinc system (either System II or a Special Protective System.)

There are many people who believe that the inorganic zinc primer (the primer used in Systems III & IV) functions best when it does not have any subsequent coatings on top of it. The System IV is the next best thing to this situation. The acrylic latex appearance coating is permeable to moisture but stops dirt and other contaminants from penetrating. This small amount of moisture that it breathes allows the zinc to corrode similar to an uncoated primer. The zinc can now use its throwing power to protect areas adjacent to damaged areas. With zinc primers that are sealed off, like in System III, the zinc near a breach in the intermediate or top coats cannot help the damaged area. The zinc in this breached area will use itself up allowing corrosion of the steel to start.

<u>What does each of the systems cost?</u>

For repaint projects, System I costs are around $4 per square foot and System II is about $7 per square foot. This can vary somewhat depending on the access and the configuration of the steel. A truss bridge high above water would cost more than a simple beam structure over solid ground.

For new steel, the painting is subsidiary to the production of the steel and prices are not quoted in the bid, so prices per square foot were not available. The price per gallon of the inorganic zinc primer is a little higher than the cost of the epoxy zinc but steel handling costs outweigh the differences.

The inorganic zinc primer sets up so that the beam can be handled much quicker than the epoxy zincs. If a beam is only primed, the inorganic zinc would be cheaper than the epoxy zinc because it can be moved through the shop and out of the paint bay much quicker. The epoxy zinc needs longer to cure to the point where it can tolerate handling with minimal damage. If an intermediate and/or an appearance coat are also applied in the shop, this adds cost because these coatings are easily damaged. Both the 742 and the acrylic latex dry quickly but are soft and are more easily damaged by the handling cranes. The urethane appearance coating takes a little longer to cure but it can tolerate the handling better, with less damage.

With all of the appearance coatings applied in the field (which gives a more uniform appearance,) the System IV would be the cheapest paint system. Next would probably be the System II because it is only primed in the shop. System III would cost more due to the intermediate coat slowing down the movement through the shop. System III with only the primer applied in the shop would be cheaper from the shop but it would require the intermediate and appearance coat to be applied in the field.

One national source provided some average costs but it is not known how close they match with TxDOT shop projects. They did not differentiate between the different categories of paints. This source said that the blast cleaning was about $1 per square foot with $1 per square foot for paint application and $0.50 per square foot for the paint material cost. They also used 7% of the cost of the beam for blast cleaning and priming and 11-12% for blast/prime/intermediate/top coat.

<u>What is a "Special Protection System?"</u>

The Special Protection System is the catch-all category that allows one to specify paints other than those in the other four systems. This can be a different combination of the paints specified in DMS-8101 or any other paint type that is needed for the project. If one wants to specify an epoxy zinc primer with a urethane appearance coating, it is not part of the four systems. This would be a Special Protective System using the epoxy zinc primer and the urethane appearance coating specified in DMS-8101.

In another case, one district painted several bridges with non-drying grease-type paint because they wanted to try something different. (There is still some concern about whether these paints can be abrasive blast cleaned due to their non-drying characteristic. They might just smear around when hit with the blaster rather than come off.)

Since these are not standard systems, the material requirements (paint properties) and types of paints must be identified, the cleaning, preparation and expected dry film thickness of the coating must be specified, and anything else that is needed to get the project done properly must be clearly spelled out.

<u>[For more on Anti-Graffiti coatings, see page 28.]</u>

<u>What System do I specify if I am concerned about Graffiti?</u>

Suppose you wanted to blast and repaint a structure and were concerned with graffiti. If you used System II, which is typical of repaint projects, then the appearance coatings would be 742 or acrylic latex. Neither of these coatings can be cleaned of graffiti without removing the coating itself. (Both can be painted over with the same coating to hide the graffiti but solvent cleaning will remove the appearance coating.)

If we invoke a Special Protection System in Item 446, we can specify an epoxy zinc primer and a urethane appearance coating meeting DMS-8101 and chosen from the preapproved list for anti-graffiti coatings on the Material Producer List. This epoxy zinc is the same type (and sometimes the same coating) as the 810e, System II primer. The urethane appearance coating will function as a permanent (DMS-8111, Type II) anti-graffiti coating where the graffiti can be removed with graffiti removers without harming the urethane.

<u>Are Urethane Appearance Coatings the same as Permanent Anti-Graffiti Coatings?</u>

At this point, the only paints to be approved to meet the DMS-8111 “Permanent Anti-Graffiti” category are the acrylic-cured aliphatic urethanes. These are the same types that are listed in the urethane appearance coating category in DMS-8101. Although some of the anti-graffiti coatings are approved for the steel appearance coat, not all of them have been tested as anti-graffiti coatings.

A structure that is coated with the urethane appearance coat can be cleaned of graffiti by using the chemical cleaners designed for that use without harming the appearance coat.

<u>What coating do you recommend along the coast?</u>

To be safe, System III is still the recommendation for steel that will be placed in service along the coast.

<u>What kind of lifetime can we expect from each of these systems?</u>

On the causeway in Port Isabel, the epoxy zinc primer that the 810e System II primer is modeled after lasted for 15 years before it was repainted. This structure is located in probably one of the harshest environments to maintain a paint job. The first bridge that was painted with the 810e was the bridge at Rio Hondo which is over an arroyo that is fed by the salt water of the Laguna Madre. While this is not a very corrosive site, it has lasted almost 20 years. (It was repainted but it was just an overcoat that went along with a repair project, not because it was corroding.)

It would be reasonable to expect 20 to 50 years on a structure that is in a lower corrosion environment. In west Texas, 50 years might be considered an early failure. There were some structures in Dallas that had an older and lower quality zinc primer on them that were overcoated when they were 30 to 35 years old. They were brittle but they were not rusted and were in good enough condition to allow the overcoating.

The System III should have the same expected lifetime as the epoxy zinc system in the same environment. One incident that occurred with an early failure in System III came about because the fabrication shop rushed the time frame in applying the epoxy intermediate coating. The inorganic zinc primer was not fully cured and when the intermediate covered it, it could not finish curing. Sometime after it was in place, the coating formed large blisters. These occurred because the zinc primer split, leaving only about one half mil of primer on the surface. The painter who did the touch-up and finish painting came in later and repaired this problem.

There is no history yet on System IV. There is a bridge in Amarillo that was changed to the Special Protective System that later became System IV. Since the project was completed in 2000, there have been no reports indicating any failure. The life expectancy is expected to be at least that of System II.

Overcoating (the new System I) has only been used for around 10 years with very few failures. One was a structure that was painted then, after painting, was lifted and moved to the location where it resides. The rust staining on this bridge is probably caused by the joints shifting when it was moved and breaking the seal from the penetrating sealer. Another failure occurred when part of the urethane peeled off the epoxy intermediate coating on a structure. It was later determined that the epoxy intermediate cured for a month before the urethane was applied. The urethane did not adhere to the hard epoxy and peeled within a few months.

At this point, we can say that the overcoating can last at least 10 years and maybe 20 or 30. These projects are more risky for making estimates on their longevity because they rely on the condition of the existing paint and whether or not it was disturbed very much during the painting project.

<u>Can I specify a certain paint and then say “or equal” to have competition?</u>

This is not recommended because the testing lab does not know how equal you want a competitor to be. Does the competitor have to match exactly the density, viscosity, color or whatever? Or does the competition just have to be the same type and generally recommended for the same service as the specified one? Can the competitor be better or does it have to be exactly equal? These are some of the questions that arise when a specifier calls for “Paint XYZ or equal.”

What are “weld through” primers?

These are zinc primers that are applied to new steel that do not have to be removed before welding. When a piece is painted in the shop, the parts that are to be welded are blocked-out so that they do not get any paint where the weld will be. The labor of measuring these areas and applying masking adds cost to the production of the steel piece. By using a “weld through” primer, these block-outs are not used and the labor is saved.

The usefulness of the concept did not overcome the problems that were discovered during investigation of these paints.

There is the safety concern for the welder. Breathing the zinc fumes when he burns through the primer is not good for the worker.

There is the problem of the application of these primers. They need to be kept in the 1 mil range and applying too thick of a coating can be bad for the welding. These coatings are usually inorganic zinc primers with a lower zinc loading than those that meet our specification. This leaves us in the situation of having a lower grade of primer under the main coating that is needed to protect the steel.

Then there is the concern about the residue from primer being trapped within the weld material. For the inorganics, we could get silica and zinc mixed in the weld and if the coating is organic, there could also be carbon and hydrogen.

CONTAINMENT

What is “containment?”

Containment is the requirements, equipment, and actions of containing debris created during the cleaning process in preparation for painting of a substrate. This is typically applies to abrasive blasting but also includes hand- or power-tool cleaning. It requires containing the blast debris of paint chips and blast media and includes containing the dust created during the abrasive blasting.

What is “negative pressure” containment?

Negative pressure containment is a method of containing debris using a dust collector that exhausts more air from the containment structure than is brought in by the blasting process. This dust collection or air removal process must have sufficient volume of air removal that the sides of the containment must be concave inward. It is required by TxDOT for all abrasive blasting.

Is containment required for System I cleaning?

All paint chips must be captured or collected during the power washing portion of the cleaning, as well as, all debris removed during the hand- or power-tool cleaning.

HAZARD WASTE

What is "hazard waste?"

These are materials that are defined by the Resource Conservation and Recovery Act (RCRA) as hazardous to human health if they are allowed to enter into the environment.

What materials are considered hazardous?

The list contains 8 metals and a large list of organic (containing carbon & hydrogen) chemicals. The metals are the ones that concern us in the paint industry. Of the listed metals, (arsenic, barium, chromium, mercury, nickel, lead, silver, & selenium) lead and chromium are the two most likely to be used in paints on TxDOT bridges.

How do we prepare for a painting contract?

One of the first steps is to find out if there is a material on the structure that will be considered as hazardous waste when it is removed. To do this, one would take samples of the paint and have them tested for hazardous content. Some people only test for the lead content but a safer method would be to test for all 8 of the listed metals.

How do I take samples to test?

Paint should be scraped from the structure that is to be painted. The samples must include all coats of the existing paint. Typically, it is the primer that contains the hazardous materials. If only the appearance coat is sampled, the test may not show any hazardous materials and the contractor will be misled.

Find an area where there is little or no rust to remove the samples. The iron in the rust can give a false test result. Some old paints can be softened for scraping by heating with a torch. [Some paints will not soften with heat, so if this is tried and the paint does not soften easily, discontinue heating.] Remove enough paint sample to fill a sandwich type plastic bag about the thickness of a finger.

Some structures have more than one paint system on them. All systems need to be tested. Sometimes, if a bridge was widened, the new steel does not have the same paint as the old part of the structure. [One clue to look for is the overspray on the bottom of the deck adjacent to the steel. If the overspray is different colors on different areas, there is probably more than one paint system on the bridge.]

<u>What is the “total lead” results that we receive from the lab?</u>

This is a test result telling you how much lead was in the paint chips that were tested. These results are used to determine the probability that the contained debris will test as hazardous.

<u>What is “TCLP?”</u>

This test is the “Toxicity Characteristic Leaching Procedure.” It is the test used to determine if the waste that was generated is hazardous.

This test simulates how much hazardous material would leach into ground water if the waste was buried in a standard landfill. The waste is soaked in water then the water is tested for the hazardous materials.

<u>How is the TCLP different from the total lead test?</u>

The total lead test tells us how much lead is in the sample and the TCLP tells us how much would leach out into the water table if the waste is dumped improperly.

<u>Why not just run the TCLP on the paint chips?</u>

Before the paint is removed from the bridge, it is not a waste and this test does not apply. Also, the waste generated during removal is composed of more than the paint waste. It contains some of the blast media and other dirt and contaminants that were on the paint when it was blasted off.

<u>Why do we have to know this before letting a contract?</u>

We need to let the contractor know if the project is a lead removal project before he bids it. A lead removal project invokes recyclable abrasive requirements, OSHA safety rules for the workers, and the disposal of the waste to a proper disposal site.

<u>Why do we specify “recyclable abrasives” for leaded jobs?</u>

By reusing the abrasives, the contractor generates less waste and we do not create problems for the disposal sites.

<u>**[BEFORE LETTING A PAINTING CONTRACT CONTACT THE CONSTRUCTION/MAINTENANCE BRANCH OF THE BRIDGE DIVISION FIELD OPERATIONS TO FIND OUT THE LATEST DETAILS ABOUT CONTAINMENT AND WASTE DISPOSAL & HANDLING.]**</u>

GALVANIZED STEEL

Which lasts longer, painting or galvanizing?

The best corrosion protection from using zinc that can be provided for steel is galvanizing. In this process, the sacrificial zinc metal is applied by dipping the steel in molten zinc. This process not only coats the steel with the zinc but the iron and zinc melt together to alloy into a very tight bond. The zinc film is also impervious to moisture penetration. With paints, the molecules of the binder resin are much larger than the water molecule and water can eventually migrate through the coating. But with galvanizing, the zinc atoms pack next to each other with no space between them for the water to penetrate through.

Galvanizing is not always the best system. In high chloride, highly acidic or alkaline environments, the zinc can degrade too rapidly and not provide the life that is desired. In some of these cases, coating the galvanized surface can overcome this problem.

Applying zinc to the steel by painting just glues the zinc particles to the steel surface. On tubular pieces, the inside is not typically painted which leaves it open to corrosion, whereas, with galvanizing, the molten zinc also coats the inside and protects it.

Why not just galvanize everything?

Galvanizing tanks are only so large and not everything can fit into them. The longest one currently is 82 feet. Design details needed to accommodate the thermal expansion that come with dipping the piece in a tank filled with 850ºF molten zinc are very important. These expansion forces have been known to break 1-inch thick plates.

These forces are even worse when a part is too big to be dipped completely and needs to be double-dipped. In this process, one side of the part is dipped and coated then turned over and the other side is dipped. In the transition area between the hot expanded part and the still cool undipped part, huge forces are created. The hot part can expand by several inches while the cool part does not.

How should I write plan sheet notes if I am planning to paint over galvanizing?

These details are now in Item 446 of the 2004 Standard Specification book.

COLORS

What do I have to put in the plan notes (color, top coat type, etc)?

Item 446 tells you what systems are available and what paints are covered by them.

If you want to use System I, you must state this. If you do not want the painter to choose the appearance coating type (acrylic latex or urethane,) this must be stated or if it is not addressed, the painter has his choice. This is where you choose the color unless you want to go with the standard default of concrete gray.

If this is a repaint project, then either System II or a Special Protection system must be noted. If System II is specified, then the choice of appearance coat types (742 or acrylic latex) should be specified, as well as, the color desired.

If a Special Protection system is to be specified, then more details will be necessary. This will include the cleaning type, the paint system, the paint material requirements, its dry film thickness, the color of the appearance coating and any other pertinent details. These systems can be variations on existing coatings, like some of the choices listed in DMS-8101, or a completely different coating.

If the steel to be painted is new and is painted in the shop, then Systems II, III, & IV or a Special Protective system can be specified. The appearance coating type and color needs to be specified. If a Special Protective system is specified, then the same details as above need to be supplied.

<u>What colors are available and how should I specify a color?</u>

Specifying the color to meet Federal Standard 595B Color is more generic than specifying some paint company color, like Sherwin Williams color XXXX or Devoe color YYYYY. The fan deck for the 595B colors should be available through the Federal General Services Administration. The Stock Number is 7690-01-162-2210.

<u>What are differences between colors?</u>

There are generally two types of pigments. These are the mineral pigments and the organic type. The mineral pigments are refined minerals like red iron oxide (barn red) which is purified rust. This pigment does not fade with time because it is already in the chemical state that it wants to be in. Generally, these pigments are the duller, more earth-tone types.

The organic pigments are chemically created compounds that are colored. These pigments tend to be the more brilliant and distinctive colors. They are very complex chemically and the laws of thermodynamics direct that they will degrade and lose color. Some of these, like phthalo blue, (which is one of the most used blue pigments) are very stable. But some of the reds and yellows do not have a very long life. Some reds fade within a few months in sunlight.

<u>Lightfast vs Non-lightfast colors?</u>

Lightfastness of a color pigment is the measure of its capability to resist fading on exposure to sunlight. The more lightfast a pigment is, the more durable it is.

As a general rule, the brighter and more distinct a color is, the more noticeable it is when it fades. Mineral pigments, tans, browns, dull yellows, and dull reds are those that are derived from mineral sources. Barn red is purified rust and will not change color regardless of how long it is exposed to weather. Fire engine red is organic (not mineral) and will fade much more rapidly. Because it starts out brighter, the decrease will be more obvious.

[See Appendix B for a questionnaire to give to the contractor to have the paint supplier answer about using lightfast pigments.]

<u>What else can cause color durability problems?</u>

Another weakness in color durability is how the pigment is incorporated into the paint. If it is dispersed as a dry pigment during the manufacturing process, it will last longer than if it is added as a tint color. This process is called “dry grinding.”

Tint colors are color pigments that come predispersed in a liquid medium and then are added to a tint base. The tint base is a paint that is made with everything except its color. This way the paint companies do not have to make a whole warehouse of different colors. Some tint colors are called “universal” colorants because they are formulated to be able to be mixed into many different types of paints. Because of the wetting agents or soaps that help this mix, these tint colorants are not as durable as dry grinding.

Another type of colorant is one that is dispersed specifically in the same resin system that the base is made from. These do not need the huge load of wetting agents to allow the mixing and thus do not weaken the paint after it is tinted. We require these for the acrylic latex paints used as appearance coatings. These colorants are dispersed in acrylic latex systems and mix in readily with no damage to the durability.

In choosing colors, it is better to choose three or four possible choices. Then ask the supplier which of these is made with the fewest different tint colors needed to attain the color.

<u>How do we require mineral pigments?</u>

This is the wording that is used in DMS-8110 for the opaque concrete sealers. (The portion left out in the second paragraph did not relate to pigments.)

“Use laminar silicates, titanium dioxide, inorganic oxides, and other mineral pigments for toning. Use of organic pigments … in the formulation are not permitted.”

Can I specify 742 in colors other than gray?

The 742 Gray Appearance Coat was designed as a flat gray paint. It was designed for use as an appearance coat for painted steel but has been used for concrete also. Over the last few years more people wanted to add colors to their bridges and they had the paint companies make the 742 in different colors. Unfortunately, due to the resin system, this paint fades to flat and gray over time. When this paint starts out as flat and gray, this degradation is not very noticeable. But when it is made in more distinct colors, this fading is more pronounced.

Another problem with this paint is that when it is applied in different thicknesses, the thicker coats are glossier than the thinner ones. In gray, this is not very noticeable but it has become more of a problem with the deeper colors.

The acrylic latex coating was evaluated and added to the list of acceptable coatings because it is designed to be used in different colors. The base resin system is durable and does not yellow or degrade as much as the 742, so it will not drag the color down.

If you are painting standard gray, then the 742 could be used. If you want other colors, switch to the acrylic latex appearance coating.

CONCRETE PAINT

Why paint concrete?

Concrete is usually painted for esthetic reasons to give it color and a more uniform appearance. Unfortunately, with the painting comes the maintenance involved with repainting and the awareness by the public when the paint is peeling or fading.

Sometimes concrete is painted to reduce salt and water from penetrating to the reinforcing bars, causing the concrete to spall. For this we can use an opaque concrete sealer or acrylic latex on top of a silane or siloxane.

Why not just mix colors into the concrete?

While this seems to be a great and maintenance free system, the people whom I have talked to who have tried it were not satisfied. It seems that each concrete pour ends up being a different shade of the color. It is possible that within each pour or truckload of concrete there can be variation. This can come from the color pigment not being dispersed well in the initial mix. Then as the surface is troweled or worked, it disperses which develops the color. This leaves a surface with different shades depending on how the color is developed by the working.

What are silanes & siloxanes?

Silanes and siloxanes are a category of chemicals based on the silicon atom. Silanes are compounds with a silicon atom at its core and two or three groups on one side of it that anchor the molecule to the surface and another chemical group on the other side that repels water. When it is applied to a surface, it anchors itself chemically and orients the water repelling part outward to stop water from getting to the surface.

Siloxanes are combinations of several silane molecules. They work the same way but are larger molecules. They may not have the same effectiveness to penetrate but they are less likely to evaporate before they can anchor themselves on a hot surface. The smaller silane molecule can evaporate before it anchors if the surface is too hot. Since both silanes and siloxanes react with inherent moisture chemically bound to the surface, they are susceptible to moisture problems when applied. If the concrete has too much moisture in the pores, the silanes will react with it and not penetrate any farther or anchor to the concrete.

Although these compounds repel liquid water from penetrating the surface, they allow the concrete to breathe. This stops excess water from entering the concrete but still allows it to come to equilibrium.

These materials can be specified by using Item 428 in the 2004 Standard Specification book.

Concrete Paint: Latex vs 742 vs opaque concrete sealer?

There are three paints that are used on concrete. The 742 (also steel appearance coat) is used in some areas. This one is the least recommended because of its durability problems. On concrete, the ultraviolet rays of sunlight degrade it faster. It is more durable when used on steel beams under a bridge deck where the sunlight cannot get to it. It is also completely porous to water and does not stop water from migrating through it.

The next category is the acrylic latex paint. It is immune to the ultraviolet rays in sunlight and to the alkalinity of the concrete. This paint allows moisture to penetrate it. If the reason for painting is to stop moisture, apply a treatment of silane or siloxane before painting. The acrylic latex paint is a film-forming coating, so it is noticeable in thickness when applied. This means that it will hide some of the texture differences in the concrete.

The “opaque concrete sealer,” or stain, as it used to be called, is a solvent-borne coating that penetrates into the concrete and provides color and protection. These are silicone/acrylic coatings. This means that they have a silicone component that function like the silanes & siloxanes to prevent liquid water from penetrating the concrete. The acrylic component glues the color pigment to the surface. Because these are typically thinner and do not form a strong film, they do not hide irregularities well. However, they do not peel easily.

Why are stains now called “opaque concrete sealers”?

In 1999, when the Environmental Protection Agency’s rule for limiting the amount of VOC’s (volatile organic compounds or solvents) was placed in effect for architectural and maintenance coatings, they limited the amount of solvents in different categories of coatings. The opaque stains were limited to a much lower level than the existing products. The suppliers raised the silicone component of their stains enough to pass testing as a sealer. So the stains were recategorized as “opaque concrete sealers,” which allows a much higher limit of solvents. This higher level of solvents is necessary for the paints to penetrate the concrete well. The change in properties allowed the change in category and the name.

Why does paint peel from concrete?

As late as the 1980’s, there were still paints approved for painting concrete that degraded on an alkaline surface, like concrete. These would peel after a couple of years, even if the surface was cleaned properly and the paint applied correctly. Some of these textured paints are probably still on structures around the state and are still peeling.

Since that time, the two main reasons for peeling are the grout slurry applied by the concrete finishers and poor surface preparation by the painters.

The grout slurry of cement, sand, and latex are applied by the crews that finish the concrete and this is done to hide any form marks or other repairs. Unfortunately, sometimes this material is applied without properly cleaning the concrete. If the form release agents are not properly removed and this water-borne grout is applied, it will not adhere properly. When paints dry, the loss of the solvent or water in the film causes shrinkage in the paint film which creates stress on the surface that it is applied to. If this grout is not adhered well because the form release agent or the curing membrane was not removed, the shrinkage of the drying of the paint film will cause the grout to disbond and peel.

If the surface does not have the grout applied and the curing membrane is not removed, the paint will adhere to this curing membrane. Since it is not adhered to the concrete, the curing membrane is pulled off the concrete by the paint shrinkage and the paint is blamed for peeling.

ANTI-GRAFFITI COATINGS

What are Anti-Graffiti Coatings?

These are coatings applied to a surface to facilitate the removal of graffiti. They do not repel graffiti or prevent it from disfiguring the surface.

TxDOT has two types of Anti-Graffiti coatings, which are described in Departmental Material Specification DMS-8111, Anti-Graffiti Coatings. The coating application and graffiti removal are specified in the 2004 Standard Specification book in Item 740.

In DMS-8111, the two types of anti-graffiti coatings described are the Type I – Sacrificial and Type II – Permanent. The Type I – Sacrificial coating is a wax based coating that is removed using a hot water power wash and as the wax melts away, it carries the graffiti with it. After the removal, another coat of the Type I must be applied again for the next use. The Type II – Permanent is a two package coating that chemically resists the solvents or chemical removers that are used to remove the graffiti from the surface of the anti-graffiti coating. The graffiti cannot be removed from this coating with power washing.

There are great hopes that the next generation will be available in the near future. This would be a Type III which would be a permanent coating that resists the graffiti and if applied, the graffiti will be easily removed with a low pressure power-wash or maybe even brushing with a broom.

What are the problems with specifying these on a project?

The biggest problem with the use of Anti-Graffiti coatings is the communication between the construction office that has them applied as part of a project and the maintenance personnel who has to remove the graffiti some time later.

The Type I – Sacrificial coating must be removed with hot water wash and another coat applied. If they remove the sacrificial coating without applying another coat, there will be no protection for the next impact of graffiti.

If the Type II is used, the crew needs to use a solvent or chemical graffiti remover. Often they will show up with the power-washer and try to use water pressure to remove the graffiti. This will not work. But they may persevere and finally blast the anti-graffiti coating off the surface which now leaves no coating to facilitate removal of the next graffiti impact.

The other tactic that is used often with either coating is to paint over the graffiti, regardless of the anti-graffiti coating that is on the surface. If this is the practice that will be used, it is recommended that the surface be painted with a paint that is designed to be recoated rather than spending extra money on anti-graffiti coatings that are not used properly. The 742 or acrylic latex appearance coatings used on steel surfaces can easily be painted over with the same paint. The 742, acrylic latex, or opaque sealer that are used on concrete also can be painted over with themselves.

It is not recommended that any of the two package system appearance coatings, urethane or some of the newer materials like polyurea or polysiloxane, be painted over to cover graffiti. The graffiti should be removed from these coatings by the solvent or chemical remover method.

<u>Can I paint my concrete with opaque sealer then coat it with clear Type II Anti-Graffiti coating?</u>

It has been done but it is not recommended. So far the reports of these systems failing have not shown up, but it is expected. The opaque sealer (or the acrylic latex) is a soft non-rigid coating and applying a rigid two package urethane (Type II – Permanent anti-graffiti coating) over it is not a good idea. It is like the difference between laying a plate of glass on top of a mattress or laying the glass on the floor and placing the mattress on top of it. This situation is the glass on top of the mattress situation.

If the surface of the concrete is to be painted in color, it is much better to apply the colored Type II directly to the concrete in one coat. With this, one gets both the desired color and the anti-graffiti protection.

APPENDIX B

PAINT DURABILITY QUESTIONNAIRE

This questionnaire can be provided by the project engineer to the contractor to be answered by the paint supplier. The intent is to assure that TxDOT gets light- and weather-fast color pigments used in appearance coatings that will not fade in a short time. This may include communication of the paint supplier with the company technical support to answer these questions.

<u>PAINT DURABILITY QUESTIONNAIRE</u>

Questions to be taken to the <u>paint supplier by the contractor</u> to assure that the weatherability of the color pigments used in the appearance coating will provide durability of the final color.

Will “universal” tint colors (those designed to be used with any solvent- or water-borne paint) be used?

Is one of the colorants “Iron Blue”?

Are any of these other colorant types to be used?

Mono Azo (Pigment yellow 1, 3, 73, 74)

Diarylide (Pigment yellow 83, 12, 13, 17)
[Some versions of PY83 (opaque version) are lightfast.]

BON Reds (Pigment red 48, 52, 57, 60)

Toluidine Red (Pigment red 3)

DNA Orange [Dinitraniline orange] (Pigment orange 5)

If the answer is <u>“**yes**”</u> to any of these questions, have someone who is familiar with the light- and weather-fastness of pigments and tints contact the project engineer. They will discuss modifying the color to one that is acceptable and is made with color pigments that will not fade quickly.