American Iron and Steel Institute

Steel Bridge Construction: Myths & Realities

D432-07

With anticipated improvements in understanding of the performance of steel bridges and the continuing development of new technology, this material might become dated. It is possible that AISI will attempt to produce future updates, but it is not guaranteed.

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Introduction

This booklet is an updated and expanded version of the original document published in the mid-90s. Its purpose is to dispel some of the "myths" or misconceptions surrounding the use of steel in bridge construction. These myths often arise out of past experience and don't take into account changes in technology, improvements in materials and products or updated design and construction practices.

Adhering to these myths can limit the competitiveness of steel solutions, lead to misuse of steel products or prevents designer and owners from taking advantage of viable options when it comes to providing crossings.

The original document focused primarily on signature bridges of steel plate girder construction. This new document has been expanded to include prefabricated/modular steel bridges using steel rolled beams and hollow structural sections and also corrugated steel pipe and corrugated steel plate as viable materials for bridge construction.

The information is presented so that choices of framing materials can be made with more accurate knowledge and in the most rational way possible. What follows is not intended to be an exhaustive treatise on the technical aspects of steel bridge design but rather to help designers and owners take full advantage of steel in their search for viable solutions. To the extent possible we have provided references as back up and as sources for additional information.

Other sources of technical assistance are:

- National Steel Bridge Alliance: *www.steelbridges.org*

- American Iron and Steel Institute: *www.steel.org*

- National Corrugated Steel Pipe Association: *www.ncspa.org*

- American Galvanizers Association: *www.galvanizeit.org*

Some people feel that once in place, concrete bridges (reinforced and prestressed) last forever and that steel bridges are slowly corroding away. Indeed the perception is that concrete is an inert material which is less vulnerable to the environment than structural steel. First, virtually all steel bridges include concrete components such as deck and/or substructure. In many cases what is labeled deterioration of a steel bridge in fact, involves the concrete components. Concrete deterioration is a subject that has been widely researched but not so widely discussed. According to the Organization for Economic Co-Operation and Development (OECD) some of the important causes of deterioration of concrete in bridges are:

- Chloride contamination by de-icing salts, saline air and seawater;
- Sulphate attack;
- Thermal effects (freeze/thaw action);
- Poor quality concrete;
- Insufficient concrete cover;
- Lack of maintenance;
- Alkali-silica reactions;
- Ineffective drainage;

Any combination of these factors, such as the use of deicing salts in a freeze/thaw climate with ineffective bridge drainage, (not an uncommon situation in the Northwest, Midwest and Northeast portions of the country), can greatly accelerate the deterioration of the bridge, be it concrete or steel.

One item mentioned above, alkali-silica reaction (ASR), has been cited by the Strategic Highway Research Program, as a major cause of cracking and deterioration in concrete structures in the United States. ASR is a reaction inherent in concrete that causes it to expand and crack based on three elements in the concrete: 1) reactive forms of silica or silicate in the aggregate, 2) sufficient alkali (sodium and potassium), primarily from the cement and 3) sufficient moisture in the concrete. The combination of the silica and alkali produce a gel reaction product. When this gel reaction product encounters moisture it expands resulting in cracking of the concrete. In arid desert-like regions of the southwestern United States the lack of moisture causes the ASR gel reaction product to shrink, which also produces cracks in the concrete. Although the symptoms of cracking and distress may be caused by external factors such as freezing and thawing, corrosion of reinforcing steel or plastic shrinkage, ASR is a process that occurs within the concrete itself.

Stein Rostam in "Concrete International" made another in-depth presentation of concrete deterioration. In his article Rostam describes carbonation, the process by which CO_2 is absorbed by concrete gradually reducing the alkalinity to a point where reinforcing steel loses the corrosion protection afforded by an alkaline concrete.

Rostam also described chloride intrusion that attacks concrete in marine environments and whenever salt is used as a deicing agent. In the latter case the concrete is also subjected to freeze shock causing small cracks that gradually allow chloride laden moisture to penetrate the body of concrete and attack the reinforcing steel. The result — spalling and loss of reinforcing itself-.may not be evident initially.

An article in the April 2007 issue of the *Journal of Protective Linings and Coatings* titled "Concrete Bridges: Heading Off the Impending Durability Burden," Bob Kogler of Rampart, LLC makes the following points:

- The demands of increasing age, traffic loading and the increased use of road salts has made the durability of bridge structures of all types more difficult.

- The increase in the number of bridges using pre-stressed concrete structural elements has led to a large number of bridges where the high-strength steel prestressing strands are protected from the environment and corrosion by only an inch or two of concrete cover.
- Corrosion of steel strands is a major factor in a significant number of bridges in the FHWA Bridge Management Information System inventory being classified as structurally deficient.
- There is a long-overdue need to consider protective coatings for concrete structures as well as targeted corrosion prevention solutions for new and existing structures.

The American Concrete Institute also recognizes that concrete structures are subject to deterioration. It recommends sealing of concrete surface to reduce permeability, considered to be the single most important factor affecting the rates of deterioration from reinforcing bar corrosion, carbonation, alkali-silica reaction or freeze-thaw cycle, all of which may be occurring simultaneously. When this type of internal deterioration occurs it is very serious; the solution is expensive repair or bridge replacement. Such hidden defects in a concrete bridge are often extraordinarily difficult to detect and can lead to catastrophic collapse such as happened in 2006 to a bridge in Quebec, Canada. Built in 1970 the collapse was blamed on misplaced or missing or short rebars; probably at the girder dapped ends, something virtually impossible to detect once the bridge was completed.

Structural steel deterioration on the other hand is visible and any signs of corrosion are clearly apparent which creates the impression that steel is maintenance prone. However, steel is easily repairable at almost any stage of corrosion and over the years has shown a remarkable tolerance to lack of maintenance.

REFERENCES:

Organization for Economic Co-Operation and Development, 1989 Road Transport Research Report; "Durability of Concrete Road Bridges", *Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures*, 1991, Strategic Highway Research Program.

Rostam, Steen. "Service Life Design — The European Approach," Concrete *International*, July 1993

Kogler, Bob. "Concrete Bridges: Heading Off the Impending Durability Burden," *Journal of Protective Linings and Coatings*, April 2007.

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MYTH: Concrete bridges outlast steel bridges.

REALITY: There is no credible statistical evidence to support the notion that concrete bridges outlast steel bridges.

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In comparing the relative durability and service life of concrete vs. steel bridges, attempts have been made to show that concrete outlasts steel when in fact the first major prestressed concrete highway bridge (the Walnut Lane Bridge in Philadelphia) was replaced after a service life of approximately forty years. Of course, there are examples of ill maintained and badly deteriorated steel bridges that have also been replaced. There are also many steel bridges with over 100 years of service life that are still performing adequately.

Perhaps the truest picture is presented in an exhaustive study conducted at Lehigh University in 1992 by Professors David Veshofsky and Carl Beidleman. They analyzed deterioration rates for, at the time, the approximately 577,000 bridges listed in the Federal Highway Administration (FHWA) National Bridge Inventory. Their conclusions were 1) that superstructure material type was not an indicator of the life expectancy of a bridge, 2) age is the

primary determinant of deterioration and 3) average daily traffic is the second most important determinant of deterioration.

More recently in an article titled "Enduring Strength" published in the September 2003 issue of Civil Engineering the authors point out existing and potential problems with post-tensioned concrete bridges. Corrosion of post-tensioning tendons was found in a significant number of recently constructed bridges in Florida and other states. Extensive non-destructive testing and inspection by use of a fiberscope, performed at considerable expense, revealed corrosion of strands because of improper grouting procedures and exposure of strands at bridge joints to saline atmosphere or de-icing chemicals.

It seems that bonded prestressing tendons are susceptible to errors that are difficult to detect and that can lead to serious structural problems. Once again, problems with steel bridges are usually ones of details such as joints and bearings.

REFERENCES:

Veshofsky, David and Beidleman, Carl R. "Comparative Analysis of Bridges Deterioration Rates," ATLSS Program-NSF Engineering Research, Lehigh University

Poston, Randall W., Ph.D., Frank, Karl H., Ph.D. and West, Jeffery, Ph.D. "Enduring Strength," *Civil Engineering*, September 2003

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MYTH: Weathering steel performs only under ideal climatic conditions

REALITY: Weathering steel performs successfully when designed and detailed according to the published FHWA and Industry guidelines for its use. There are many cases of weathering steel bridges not conforming to the guidelines that are also performing well.

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When used properly, uncoated weathering steel is by far the most cost-effective material for bridges when considering either first or long-term costs. Over the years there have been some isolated problems due to a lack of understanding of the material and its subsequent misuse. The fact remains that weathering steel is acceptable in most locations of the country. Because of isolated problems, however, it became clear that guidelines on the use of weathering steel were needed so that owners could enjoy its economic benefits with confidence.

FHWA GUIDELINES

In 1988 the FHWA conducted a "Weathering Steel Forum" to establish these guidelines. This forum brought together state departments of transportation to discussed their positive and negative experiences with weathering steel bridges. The outcome of this forum was the FHWA "Technical Advisory — Uncoated Weathering Steel in Structures" in 1989. (These guidelines, although still valid, are currently being reviewed by FHWA and supplemented with more data.) In accordance with these FHWA guidelines, there are four considerations that must be taken into account when considering the use of weathering steel:

- Environmental and Site Conditions
- Location
- Design Details for Proper Drainage
- Maintenance

Environment

An evaluation of atmospheric and site conditions at a particular site should be made before uncoated weathering steel is considered. The steel industry offers a free service to help owners evaluate such factors as marine atmosphere, annual rainfall, prevalence of fog, and atmospheric and industrial pollutants in order to determine whether site conditions are suitable for the use of uncoated weathering steel. Some of these factors such as saline atmosphere can adversely affect the performance of any bridge material.

Location

Grade separations over depressed roadways in urban environments subject to heavy road salt application and with long and deep approach retaining walls that produce a tunnel effect, can cause salt behind vehicles to be lifted off the roadway and deposited on the bridge above. This can result in excessive corrosion of weathering steel. (**Figure 1**) There are however, innumerable unpainted weathering steel bridges used in standard overpasses with more than 30 years of successful performance.

Figure 1: Grade Separations Problems
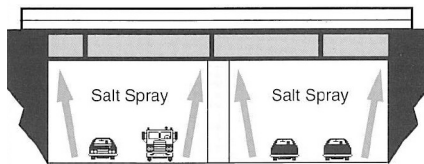
Design Details

The single most important factor affecting the performance of uncoated weathering steel involves design details that assure proper drainage, thereby minimizing the exposure of the steel to water and deicing salts from the roadway above. The FHWA Technical Advisory fully explains proper design details. Here are some of the highlights:

Joints — Bridge joints, when possible, should be eliminated (see section on bridge joints) because they add to problems of corrosion, rideablity and maintenance of all types of structures. Where joints are used, assume they will leak and provide proper drainage for them such as sloped drains under the expansion joint. The FHWA recommends that steel be painted underneath the joint for a distance of 1½ times the girder depth to protect against the effects of leakage. Once again, there are many examples of bridges with more than 30 years of successful performance without painting. So, proper detailing is both important and effective.

Drainage — Roadway and deck drainage should be diverted away from the superstructure and substructure. Maximizing space between scuppers increases the velocity of water running through them that will allow the flow to flush away debris. Downspouts should not contact the steel members and drains should not be routed through closed box girder sections where leaks can go undetected.

Maintenance and Inspection

Uncoated weathering steel bridges, like all bridges, need to have effective inspection programs. Because of the unique nature of uncoated weathering steel, inspectors need to know the difference between the desired oxide coatings and excessive rust scaling. Information and further assistance on this is available from AISI. Maintenance programs should include:

- Cleaning troughs of joints and resealing of deck joints.
- Cleaning and painting of steel only in the zone under bridge joints or repainting (if necessary).
- Removal of dirt and debris that hold moisture and maintain a wet surface condition on the steel. Such conditions do not allow the steel to develop its protective patina.
- Maintaining screen covers over drains.
- Removal of nearby vegetation that prevents natural drying of the steel surface.

CONCRETE STAINING

Staining of the concrete substructure can occur with uncoated weathering steel. Most of the problems occur during construction before placement of the bridge deck after which time the steel is protected. This is true even under bridge joints that usually remain weather tight long enough for the protective patina to form on the steel. In certain environments the patina can form in as little as one year. In extremely arid climates the oxide may never form completely. Generally speaking, it takes about three years of alternate wetting and drying for the protective oxide to form completely.

Protection of the pier caps and abutments during construction prior to deck placement is key. This can be accomplished by temporarily wrapping them with polyethylene (**Figure 2**). Another solution is to seal the concrete to prevent penetration by the stain. Clear sealers such as silane, siloxane, polyurethane and liquid silicone can provide at least two to four years of protection for this purpose.

Figure 2: Concrete Staining
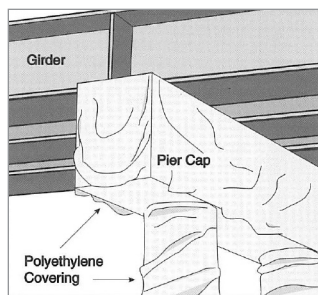
If corrosion protection of the concrete pier caps or abutments is desired any of the above sealers can be combined with a clear or pigmented polyurethane topcoat. Such a system should provide 25 to 30 years of protection.

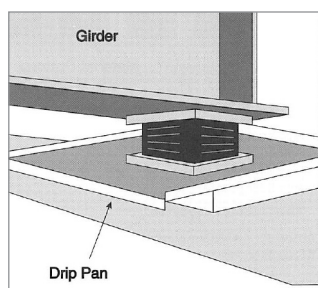There are also details that help divert the water away from the concrete such as drip pans in **Figure 3**. However, this method may be ineffective if the piers are very wide or tall as wind can carry diverted water back to the concrete surfaces.

Figure 3: Drainage Details

EXAMPLE BRIDGES

The environmental considerations in the FHWA guidelines are not intended to be a limitation on the use of weathering steel; given proper consideration the guidelines may be exceeded in certain cases. There are numerous examples of weathering steel bridges that are performing exceptionally well under atmospheric conditions more severe than those recommended in the FHWA guidelines.

For example, a series of ten weathering steel bridges traverse the mountainous region from San Juan to the southern shore of Puerto Rico, carrying route PR52 over gullies and grade separations. The atmosphere is a hot and humid tropical climate with prevailing salt-laden winds and approximately 100 inches of rainfall a year. These bridges, in service for over twenty-five years in this questionable location and atmosphere, have performed exceptionally well without any major maintenance problems.

Another example is a section of the New Jersey Turnpike that is close to the ocean, crosses many salt marshes and passes through one of the worst area of industrial pollution in the country. These bridges have been in operation for years and continue to perform very well.

In some cases a weathered appearance may not be the first choice but this should not prevent owners from benefiting from the economies of weathering steel. In these cases the recommendation is to blast clean and paint the outside surface of the fascia girders only.

Given recent positive experience and the overwhelming short and long term cost benefits of weathering steel, its use deserves careful consideration by all owners and in fact several states use uncoated weathering steel as their default specification for steel bridges unless there is a clear reason not to.

REFERENCES:

AISC Marketing, Inc. "Uncoated Weathering Steel Bridges," Vol. 1, Ch. 9., *Highway Structures Design Handbook*, January 1993

"Uncoated Weathering Steel in Structures," *FHWA Technical Advisory (T5140.22)* October 3, 1989

American Iron and Steel Institute "Performance of Weathering Steel in Highway Bridges," Robert L. Nickerson, 1995

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MYTH: Optimization by weight is the best approach to economical design.

REALTY: Although this may be true in some cases, savings in material may sometimes be more than offset by increases in fabrication cost; in certain instances, adding weight may provide the least cost solution.

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In the past, it was often sufficient to find the least weight solution and assume that this would also be the most economical. However, over time material and labor costs can fluctuate due to global or national economics and can also vary regionally. As a result the designer needs to be more aware of the balance between more or less material and the impact on fabrication time i.e., the number of detail pieces and shop operations involved.

FLANGE PLATES

One example involves flange plates that represent a significant portion of material costs. The amount of labor involved in fabricating flanges can vary significantly as a result of design. If one understands the most economical way of making up flange material in the shop, this variance is easier to understand. The most efficient way to make flanges is to weld together several plates of varying thicknesses received from the mill. After welding and non-destructive testing, the individual flanges are

"stripped" or gang cut from the full plate (**Figure 4**). This reduces the number of welds, individual run-off tabs to start and stop welds, the amount of material waste and the number of X-rays for non-destructive testing. The obvious objective, therefore, is to *keep flange widths constant* within an individual shipping length by varying material thickness as required. This is also beneficial when utilizing metal stay-in-place deck forms.

Because most fabricators will generally purchase plates in minimum widths of 72 inches to obtain size discounts, it is best to repeat plate thicknesses as much as possible. In the example shown in **Figure 5**, there are too many different plate thicknesses. It would have been better to increase the thickness of some plates in order to combine widths to get to the 72" purchasing width. The thicker plates don't allow this but at least the design/cost equation has been satisfied to the extent possible. Furthermore, without combining, each splice will have to be individually rather than gang welded. (When combining plate widths fabricators must allow for ¼" width loss between burns.)

Said another way, larger order quantities of single plate thicknesses cost less because they often allow the fabricator to satisfy requirements for minimum order quantities thereby eliminating tonnage surcharges. Similar sizes of flanges obtained during preliminary design should be grouped to minimize the number of thicknesses of plate that must be ordered. For example, if preliminary design is optimized with eight thicknesses of 1¼, 1⅜, 1½, 1¾, 1⅞, 2, 2⅛ and 2½ inch, consider reducing to four plate thicknesses of 1¼, 1½, 1⅞ and 2½ inch.

The discussion of flange design leads to the question of how much additional flange material can be justified to eliminate a width or thickness *transition*. As a result of discussing hundreds of designs with fabricators some rules of thumb seem to apply. The AASHTO/NSBA Steel Bridge Collaboration has summarized those guidelines in the table and example below.

Weight Saving Factor Per Inch of Plate Width for ASTM A-709-Gr. 50 Non-Fracture Critical Flanges Requiring Zone 1 CVN Testing

Multiply weight savings/inch x flange width (length of butt weld)

Thinner Plate @ Splice (inches)	Thicker Plate @ Splice (inches)						
	1.0	1.5	2.0	2.5	3.0	3.5	4.0
1.0	70	70	70				
1.5		80	80	80	80		
2.0			90	90	90	70	70
2.5				100	100	80	80
3.0					110	90	90
3.5						110	110
4.0							130

The following example demonstrates the use of the table. Evaluate splicing a plate 16" x 1" x 35' to a plate 16" x 1½" x 35' versus using a plate 16" x 1½" x 70'. The weight saved by adding the splice is equivalent to the weight of a plate 16" x ½" x 35' (16" x 0.5" x 3.4 pounds/sq. inch x 35' = 952 pounds) or about 950 pounds. The weight savings needed to justify adding the splice is determined by using a factor of 70 pounds per inch from the table, times the plate width of 16", resulting in a value of 1,120 pounds. Because the actual saving is 950 pounds the table indicates that it is more economical to extend the 1½" plate for the full 70' than to add the shop splice.

When making flange transitions, there are two additional things to keep in mind: 1) It is good design practice to reduce the flange cross-sectional area by no more than approximately one-half of the area of the heavier flange plate to reduce the build-up of stress at the transition, and 2 One should avoid making flange width transitions within a field piece but if a transition in width must be provided, shift the butt splice a minimum of 3" from the transition as shown in **Figure 6**. This makes it much easier to fit run-off tabs, weld and test the splice and then grind off the run-off tabs.

Figure 4–6: Plate Burning
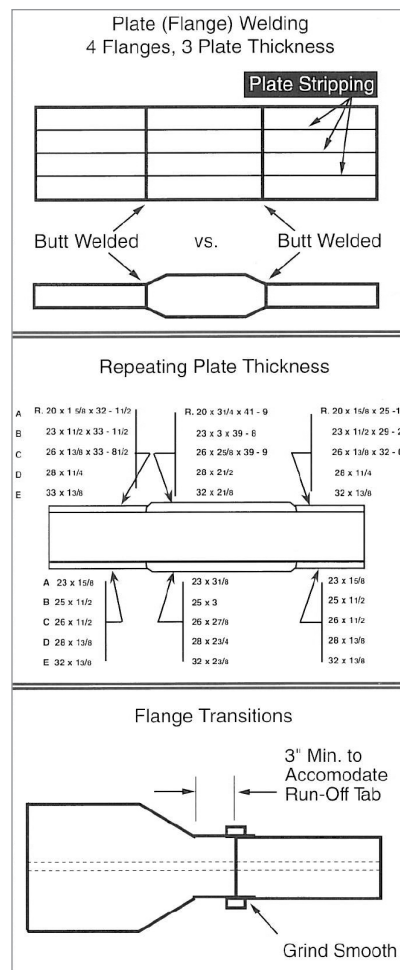
WEB PLATES

Web design is another area that can have a significant impact on the overall cost of a plate girder. From the standpoint of material costs, it is usually desirable to make girder webs as thin as design considerations will permit. However, this may not always produce the greatest economy since fabricating and installing stiffeners is one of the most labor intensive of shop operations. Once again, here are some guidelines applying to use of stiffeners:

- Generally avoid using web thickness less than ½".
- Cross frame connections will act as web stiffeners. The LRFD Specification does not prescribe cross frame spacing. If web stiffeners are spaced at three times the girder depth or less a girder is considered to be fully stiffened. Therefore, if cross frames are located at three times the girder depth or less a girder is considered fully stiffened.
- Transversely unstiffened webs are generally more economical for web depths approximately 50 inches or less.
- Generally, partially stiffened webs are most economical for a typical plate girder.
- Intermediate transverse stiffeners should be placed on only one side of the web and should be cut back a minimum of one inch from the tension flange to accommodate painting. The distance between welds must be limited to between 4 to 6 times the thickness of the web to prevent crippling of the web in the gap. Transverse stiffeners should not bear on both top and bottom flanges. Tight fitting of transverse stiffeners is very time consuming because each one has to be individually cut and ground to fit at each location.
- Longitudinal stiffeners should generally be avoided but when used in conjunction with transverse stiffeners on longer spans with deeper webs, they should preferably be placed on the opposite side of the web from the transverse stiffener. Where this is not possible such as at intersections with cross-frame connection plates, the longitudinal stiffener should not be interrupted for the transverse stiffener.

CONSTRUCTIBILITY

Designers should also be aware that least weight designs also have an effect on contractibility. Field pieces need to be stable during lifting and setting in place. As a general rule, the unsupported length in compression of the shipping piece divided by the minimum width of the flange in compression in that piece should be less than about 85. Generally, good design practice would indicate a minimum flange thickness of ¾ inches and a minimum width of 12 inches.

In summary the most economical and most practical design is not necessarily the one with the least weight but rather the one with the lowest cost after taking into account fabrication costs, transportation and erection limitations. The recommendation is to first recognize that different fabricators and constructors will prefer different procedures and secondly, show alternate types of details so that they can choose those that most closely match their capabilities.

It should also be empahsized that the guidelines cited in this discussion may vary from job to job and location to location. It is recommended that when in doubt the bridge designer consult the National Steel Bridge Alliance or local fabricators who are potential bidders to discuss individual project requirements.

REFERENCES:

AASHTO/NSBA Steel Bridge Collaboration Document G 12.1–2003, "Guidelines for Design for Constructibility"

Guzek, Thomas P. and Grzybowski, John R. "Reducing Bridge Fabrication Costs," *Modern Steel Construction*, September 1992

Mion, Roy L. and Grubb, Michael A. "Cost-Effective Design of Steel Girder Bridges," *Proceedings, National Symposium on Steel Bridge Construction*, *1993* AISC Marketing, Inc.

**MYTH:** Steel Is Not Recommended For Short Spans.

REALITY: Due to changing designs and prices for both steel and concrete members, the relative economics of span and cost of each material has also changed. In many cases, the most economical steel span may be close to or the same as for the concrete design. In some cases where 250-foot to 350-foot segmental prestressed concrete spans are used, the most economical steel spans may be shorter than for concrete.

There is a perception in bridge design that because steel has a greater strength to weight ratio than concrete, to be competitive, the steel design should have the longer spans. Thus designs often appear with fewer, longer spans for the steel alternate than for the concrete. The truth is that the new prestressed concrete bulb-tee members are thinner and more efficient than the old sections and have strength to weight ratios more closely approaching those for steel. As a result in the 130-foot to 170-foot span ranges the more competitive steel design may have spans close to those of the concrete design.

Most importantly, in determining the most economical span arrangement, it is meaningless to just compare the cost of the steel superstructure with that of the concrete. One must look at total bridge costs including the substructure. In fact, it is the cost of the substructure for each design that usually determines the most economical span arrangement; if substructure costs are relatively high, the argument is for longer spans thereby eliminating costly piers and foundations. If these costs are low, shorter spans are more efficient to reduce the cost of the superstructure. The optimum span arrangement can only be determined by comparing cost curves for superstructure, substructure and total structure for each material as shown in **Figure 7**; the optimum span falls at the minimum or low point of the total cost curve. In the case illustrated, the most economical span would have been around 165 feet.
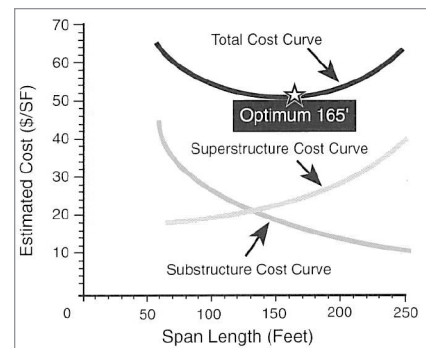
The validity of such an analysis is only as good as the accuracy of the cost data. Often rule-of-thumb estimates are applied for substructure costs, which turn out to be highly inaccurate thereby leading to improper conclusions about most economical span lengths.

From looking at designs that included steel and concrete alternates in moderate span lengths of 130 to 140 feet, simply reducing the steel span lengths to those called for in the concrete design reduced the steel quantities and costs by 30–45%. For example, in a case in the Midwest, the concrete design called for nine equal spans of 133' 9" and the steel design had eight spans ranging up to 180' in length. Changing the steel design to the same 133' 9" span arrangement as for concrete reduced the steel quantities and cost by almost 37 percent.

Often designers concentrate on optimizing individual spans by minimizing the number of lines of girders and in so doing will generally reduce superstructure weights by 5–10%. While important, it is the careful determination of span arrangement that can add significant savings.

If substructure costs have such a substantial impact on the most economical span arrangement, it is logical to make pier designs as efficient as possible. Without going into an in-depth discussion of pier design, there are a couple of guidelines which will help reduce costs; smaller stems with increased pier

cap cantilevers are more compact and easier to form and, constant shapes (sections) allow contractors to use readily available and re-usable forms.

Figure 8: Jointless Abutment Detail

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MYTH: Bridge joints are essential.

REALITY: Experience has shown that jointless bridge decks can be designed to provide a durable and cost-effective structure.

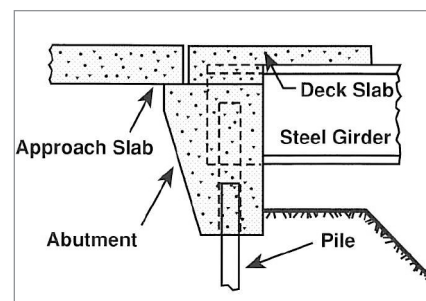. . . . .

For many years bridges had been designed as a series of simple spans with a corresponding number of expansion joints. Without the help of modern computers and calculators, designers often found the analysis of continuous-span bridges to be a tedious task. Therefore, designers sometimes adopted a simpler approach of designing multiple simple-span bridges. Although bridge expansion joints relieve secondary stresses of the superstructure from thermal and moisture changes of the deck, this solution caused more problems than it solved! Expansion joints cause structural deterioration problems because of:

- Leakage of road salt runoff onto the superstructure and substructure
- Corrosion and deterioration of beams, bearings and bridge seats under the joints
- Need to maintain and periodically replace the joints

These bridge maintenance problems can be avoided by designing continuous-span superstructures without joints by making them integral with the substructure. The integral connection with the substructure relieves the superstructure of the secondary stress through the foundation instead of relieving the stress by the use of expansion joints (**Figure 8**).

Many states have a long history of using jointless bridges some of which are California, Colorado, Idaho, Indiana, Iowa, Kansas, Missouri, Nebraska, Ohio, North Dakota, South Dakota, Tennessee, Virginia and Wisconsin. The State of Tennessee designs mostly jointless bridges, using expansion joints only when absolutely necessary. In the 1950's, the DOT started building joint-free structures of short length. Over time they have been expanding the number of structures and their lengths with no known serious problems attributed to the absence of a joint. The Tennessee DOT has constructed steel structures over 400 feet in length with no joints and up to 2,800 ft in length without deck joints, except at the abutments. When utilizing jointless bridges, proper details should be provided to accommodate the relative movement of the integral abutment and the approach slab and pavement.

In over 40-years experience many savings have been realized in initial construction costs by eliminating joints and bearings and in long-term maintenance expenses from the elimination of joint replacement and the repair of both super and substructure.

REFERENCE:

National Steel Bridge Alliance. "Integral Abutments for Steel Bridges," *Highway Structures Design Handbook*, Volume II, Chapter 5

There is no written history which shows how and why bearings for steel bridges evolved into the expensive and elaborate systems we often see today, and why bearings for prestressed concrete bridges have remained simpler and much more economical. One can surmise that many years ago when steel was the only choice for bridges of any significance, the fabricator supplied not only the superstructure, but the bearings of fabricated steel as well. As spans increased and bridges became more sophisticated, the bearings became more elaborate and expensive too. On the other hand, prestressed concrete bridges started with shorter spans and simpler bearing pads of unreinforced neoprene. Starting from a simpler initial concept, these bearings tended to remain simple even as prestressed concrete structures increased in span and sophistication. It was as if the development of bearings for the two types of bridges moved forward on two separate tracks.

The use of one type of bearing for steel bridges and another for concrete designs may thus have been born at least partly out of tradition. One thing is certain; there is often a large difference in the cost of bearings between steel and concrete designs for the same bridge. In fact, in a Midwestern project, the bearings for the steel alternate cost almost $300,000 more than for the concrete design.

This difference in design approach, according to Prof. Charles Roeder, PE, of the University of Washington, is also partly a result of the AASHTO Specification. Even though the coefficients of expansion for steel and concrete are very close, the specification treats temperature movements much more liberally for concrete than for steel. Depending upon how one interprets the specification language those movements are usually twice and sometimes four to five times those for concrete. The argument is that concrete has more mass and takes longer to heat up than steel and thus, does not exhibit the same movement. Prof. Roeder maintains, however, that a steel superstructure rarely acts alone but rather in conjunction with a large concrete mass, the deck.

Even within the existing AASHTO Specification, there are ways to conform with the improved bearing pads available today. Gilbert Blake, PE of Wiss, Janney, Elstner Associates, Inc. states that the industry has come a long way from the original unreinforced neoprene pad of limited capabilities. Today the steel reinforced elastomeric, fiber reinforced elastomeric and performed fabric pads meet the load carrying capacity requirements and have the ability to take significant multidirectional displacements. In addition they are more economical and not subject to locking up in comparison to various traditional steel bearings. For most applications these pads should be the first consideration in bearing design for steel bridges (**See Figures 9 and 10**).

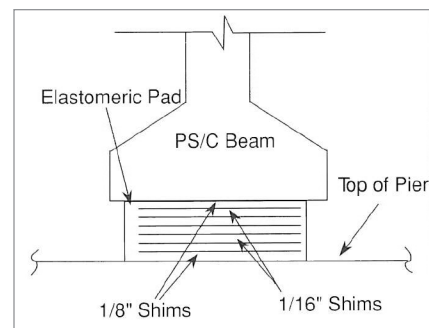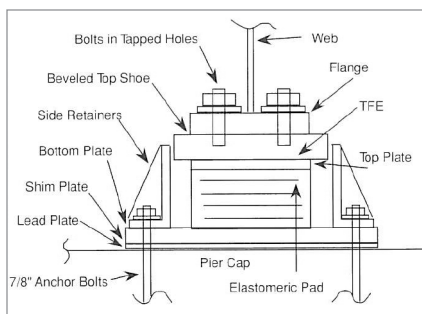Figure 9: Elastometic Bearings for Concrete Alternate

Figure 10: Elastometic Bearings for Steel Alternate

Bolts in Tapped Holes — Web
Beveled Top Shoe — Flange
Side Retainers — TFE
Bottom Plate — Top Plate
Shim Plate
Lead Plate
Pier Cap
7/8" Anchor Bolts — Elastomeric Pad

REFERENCES:

Stanton, John F., Roeder, Charles W., Campbell, Ivan T. (Dec. 1992). "High Load Multi-Rotational Bridge Bearings," — Draft final report. *National Cooperative Highway Research Program, NCHRP 10-20/A*, Washington, D. C.

"Steel Bridge Bearing Design and Detailing Guidelines,". *AASHTO/NSBA Steel Bridge Collaboration Document G9.1*

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MYTH: Painting or repainting of steel bridges is an insurmountable problem.

REALITY: There are cost-effective ways of repainting existing bridges. For new construction, there are modern high-performance coatings which comply with EPA standards and which can provide a minimum service life of 25 years prior to first paint maintenance.

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Few subjects have received more attention or created as much controversy as the issue of paint. This has been created in large part by stringent federal and state EPA standards prohibiting the use of lead-based paints and limiting quantities of volatile organic compounds (VOC's) in other paint systems. In addition, removal of lead based paints from older structures falls under strict rules for containment and worker protection.

EXISTING BRIDGES

In light of regulations, the owner is faced with what to do for both existing and new bridges. First, for older bridges, the real issue is what to do with the existing lead-based paint. This is a problem that cannot be ignored, but it should be recognized that there are reasonable options in terms of what can be done. According to Eric Kline of KTA-TATOR, Inc., a fundamental principle is not to let older bridges deteriorate to the point where the only course of action is complete removal and containment of lead-based paint and subsequent repainting. Beyond that, he says there are three options for existing bridges, in order of preference:

1. Do nothing. If the surface condition of the bridge does not allow repair and overcoating of the existing system, delay eventual paint removal and repainting as long as possible. In this manner, funding for eventual removal and replacement of the system can be budgeted.

2. Repair. If the surface condition of the bridge allows, repair and overcoat the existing system. Repairs may include spot or zone treatment. Spot treatment entails dealing with localized areas, while zone treatment involves dealing with definable areas; e.g., clean and paint 1½ times the web depth on either side of the expansion dam. Naturally, coating strategies may include combinations of both spot and zone painting. Note that it is much easier for a contractor to accurately provide a cost for zone cleaning and painting, since the parameters affecting his costs are more easily defined. It may be convenient to request a cost to provide spot repairs on a "per square foot" basis and an add-on cost for a zone cleaning and painting effort.

3. Repair/Replace. When delay is no longer possible, remove and contain the lead-based paint and recoat the structure with a high-performance VOC compliant paint system.

Clearly, the most expensive option is the final one calling for complete removal, containment and repainting. Therefore it benefits an owner to do everything possible to extend the service life of the coating system in place prior to a remove-and-replace effort. The most cost-effective effort is often to repair and overcoat the existing system. There are recognized methods for evaluating the condition of the existing system, and its suitability for overcoating. Data on the following are helpful in determining suitability:

1. The extent and distribution of corrosion or coating deterioration

2. The number of coating layers and the total thickness of the existing coating

3. The adhesion characteristics of the system in order to determine its suitability for overcoating.

4. The condition of the substrate beneath the coating (mill scale, rust or abrasive blast cleaned)

Experience has shown that a coating system can be upgraded by overcoating if it exhibits less than 10 percent deterioration/corrosion, has a thickness of 5–20 mils and has satisfactory adhesion. Naturally, after meeting these criteria, "test patches" of the proposed surface preparation/coating application should be undertaken prior to making a commitment to total overcoating.

If the topcoat is peeling but layers beneath are intact, preparation of the surface by hand, power tools, pressure washing or water jetting or even open nozzle (brush off) blast cleaning may be necessary to remove the top coat layers before overcoating.

Removal of the topcoat(s) alone can be a pivotal consideration in the decision process. It may cost as much to remove, contain, and dispose of the debris as it would to simply remove the entire system. Careful cost analysis will indicate which option is preferable.

Overcoating or Recoating Material Selection: The question remains as to what type of paint systems to use for overcoating? There are several possibilities that have emerged after many years of laboratory and field testing. A list of some such materials is shown below:

- Alkyd (lead free)
- Calcium Sulfonate Alkyds
- Epoxy Mastic/Urethane
- Moisture Cured Urethane
- Waterborne Acrylic

Information about the systems named above can be obtained through NEPCOAT or AASHTO NTPEP SSC.

NEPCOAT is an affiliation of northeastern states (Maine, Vermont, New Hampshire, Rhode Island, Massachusetts, Connecticut, New York, New Jersey, and Pennsylvania). The affiliation was formed for the purpose of developing testing criteria for protective coatings used on highway bridge steel. The Overcoat Program was designed to comparatively test products to determine their performance under field conditions. The NEPCOAT Qualified Products List (QPL) currently lists three overcoat systems (see NEPCOAT at www.nepcoat.org). NEPCOAT also has a testing program for the approval of three-coat or two-coat options for new or bare steel (See NEPCOAT QPL A, B, and C).

In general terms, overcoating systems should impart low shrinkage stresses during curing and high solids content to minimize solvent penetration and softening of the underlying paint systems.

AASHTO NTPEP SSC The American Association of State Highway and Transportation Officials (AASHTO) oversees a materials testing branch known as NTPEP (National Transportation Product Evaluation Program). NTPEP is comprised of highway safety and construction materials project panels. These panels are made up of state highway agency personnel with the objective of providing

quality and responsive engineering for the testing and evaluation of products, materials, and devices that are commonly used by the AASHTO member departments of transportation. In 1997, the Structural Steel Coatings (SSC) Panel (under the Construction Materials Project Panels) was created to develop a standard specification, a corresponding project work plan, and a reporting system for testing of industrial coating systems for use on bridge and highway structures.

In 2002, the Provisional PP30 specification became AASHTO Specification R31, "Standard Practice for Evaluation of Coating Systems with Zinc-Rich Primers." AASHTO R31 is a comprehensive specification that describes the testing criteria for evaluating coating systems containing zinc-rich primers for use on iron and steel surfaces. The testing criteria are comprised of four areas: compositional properties, accelerated corrosion and cyclic weathering, physical testing, and two-year atmospheric exposure.

AN OVERCOATING CAUTION: Anytime new coating material is applied over old coatings, some risk of a disbonding-type adhesion failure is inherent. Risk management is therefore necessary. Risk management is undertaken by the above data gathering/ analysis and by the preparation and application of candidate coatings to test sections of the structure(s) in question.

If the condition of the coating is beyond overcoating, delaying any action as long as possible may be the prudent course. The idea is to hold out as long as possible so as to take advantage of funding opportunities and yet not risk excessive damage to the steel.

COST: The cost of any of these options vary by region and individual job conditions, but recent history indicates that that costs have been reduced as the technology needed to handle the containment and worker safety issues has become widespread.

The financial benefits of overcoating are incontestable. This procedure can be from 45–80% less expensive than complete removal. The costs of recent coatings projects vary greatly. According to KTA-Tator, Inc., the average cost of overcoating ranges from $4 to $6 per square foot, as opposed to complete removal and replacement at $12 to $14 per square foot. The life expectancy difference between the two coating methods is approximately 10 to 13 years. Stated another way, a new system over bare steel would last at least 25 years; with overcoating, the owner would enjoy another 12 to 15 years of protection.

NEW BRIDGES

The question of painting new bridges should not be surrounded with as much drama and anxiety as for repainting of older existing structures. Painting a new bridge today does not entail confronting the same environmentally sensitive problems as now associated with the older lead-based paint system. When it comes to new construction, we now benefit from great strides made in high-performance coatings, which are also in compliance with federal and state EPA standards. A list of some of the systems used on new steel is shown below:

- Zinc/Epoxy/Urethane
- Zinc/Epoxy/Siloxane
- Zinc/Polyaspartic
- Moisture Cured Urethane (zinc primer)
- Waterborne Acrylic
- Fluoropolymer
- Thermal Spray Coating (metallizing) with and without seal coats

Information about individual coating systems for new steel can be obtained from the sources already cited. What kind of performance can be expected?

According to Eric Kline of KTA-TATOR, Inc., the organic and inorganic zinc-rich primer coating systems, if properly applied, can be considered permanent protection which will be in place for the life of the structure. With an intermediate coat

and topcoat applied, the first required maintenance should not be needed for at least 25 years. At that time, with spot cleaning, spot priming and the addition of another topcoat (approx. 2–3 mils), another 15–20 years of service life should be anticipated. At the end of that period, the same process would be repeated several times with the same anticipated results. Coating life can achieve 100 years with the original zinc rich primer still in place.

When constructing a new steel bridge today, an owner now benefits from a number of environmentally friendly coatings with greatly extended service life. In the near future it is expected that, coating technology will continue to evolve allowing the development of coating systems that are even longer lasting and more economical.

In addition there are a number of notions about painting that increase cost without improving performance.

EDGE GRINDING

The idea of edges of beams being ground to a $\frac{1}{16}$–inch radius before shop painting is probably rooted in the traditional belief that coatings draw thin on sharp edges due to the forces of surface tension during drying. Reduced thickness would then lead to corrosion failure. This is not true for bridge paints commonly specified today.

Even though this was true when most industrial coatings were oil-based, it was not true when these coatings were applied on rolled edges. Rolled edges have rarely been shown to require any additional preparation for painting, as the rolling process leaves a rounded edge, although it might not be a $\frac{1}{16}$–inch radius. Even when edges are sheared or burned, grinding to a $\frac{1}{16}$–inch radius is not necessary for paint performance.

Highly pigmented zinc-rich paints do not flow away from the edge, and in addition, provide galvanic throwing power to protect any edges or areas not coated. Also, these materials resist corrosion undercutting. Therefore, the requirement that burned or

sheared edges always be ground to a minimum $\frac{1}{16}$–inch radius is questionable. Edge radiusing requirements in fabrication specifications are not only very expensive but offer undetectable improvements in corrosion resistance.

Another important fact is that technological advances by coating manufacturers have yielded other high-build low-solvent and solvent-free epoxy formulations that have modified rheological properties and better resist flow when applied to edges. The high-solid (VOC compliant) products in use today have greatly reduced wet film volume shrinkage and drawback from edges.

Long-term salt fog testing by one coating manufacturer indicated no difference in performance between rolled beam un-ground, sheared plate untreated and sheared plate radiused edges when coated with an inorganic zinc-based coating system. Extensive testing by NASA on dozens of inorganic zinc rich coatings showed no edge breakdown on saw-cut and sheared edges primed with inorganic zinc after 18 years in a salt spray exposure site.

Improved specification language should include provisions that reflect the following:

- Sharp edges, such as those created by flame cutting and shearing, shall be broken before surface preparation. (Breaking edges can be accomplished by a single pass of a grinder in order to flatten the edge.)
- Usually the rolled edges of angles, channels and rolled beams are presumed to need no further rounding. (If sharp edges occur the single pass of a grinder can flatten them.)
- Machine fillet welds are considered a paintable surface with no further treatment required. Only weld spatter need be removed.

SURFACE IMPERFECTIONS

Another common myth is that surface imperfections such as ridges, slivers, fins or hackles must be ground flush since they are also sharp edges. Such

anomalies are surface imperfections on rolled sections and plate. They result when small (usually less that ½ inch) areas of the steel surface are not well bonded to the surrounding surface are bent upwards during blast cleaning. It is typically only necessary to cut off the head of isolated hackles with no further grinding. An exception could occur if there were extensive hackles in a small area. In such instances some further attention could be warranted.

RE-PROFILING OF BLAST-CLEANED SURFACES

Blast cleaned surfaces that are subsequently ground do not need to be re-profiled to achieve effective coating performance. A study undertaken by SSPC: The Society for Protective Coatings (formerly Steel Structures Painting Council) has shown that steel that had been blast cleaned, ground and recoated performed well as well in salt fog tests as steel that had been re-profiled and recoated. Limited test data and extensive field experience have shown that the small areas (approximately 1 square inch and smaller) that have not been re-profiled perform as well as surfaces that were re-profiled before coating. While the SSPC blast cleaning specifications for Commercial (SP6), Near-White (SP10) and White Metal (SP5) address the necessity of correcting damage to the profile resulting from operations that take place after blast cleaning, common sense requires the recognition that small areas covering only a few square inches may be ground and not re-profiled without adversely affecting coating performance. Likewise, common sense dictates that if large areas are involved, re-profiling may be required.

SHOP-APPLIED FINISH COATS

Many have the idea that applying all paint coats in the fabricator's shop results in superior corrosion resistance as compared with the traditional shop-prime/field-top coat approach. While quality control and accessibility to inspection is easier in the shop some important sacrifices in corrosion resistance result.

The history of multi-coat shop systems is still being recorded. More and more specifiers are calling for this approach as they replace the lower performing vinyl field coats with epoxy and urethane topcoats. Vinyl topcoats were "dry-fall" and could be sprayed without fear of drifting and adhering to vehicles. This is not necessarily true with the epoxy and urethane coatings, an argument for their application in the fabricator's shop. Nevertheless, these latter coatings are being sprayed extensively in field maintenance applications.

The Michigan DOT has been the most influential proponent of multi-coat shop systems. In the early 1980s, the Michigan DOT began requiring their three-coat system of inorganic zinc/epoxy/polyurethane to be applied in a controlled shop environment. This lead to handling damage problems associated with fabricators', haulers' and erectors' unfamiliarity with handling finish-painted steel and topcoated inorganic zinc that could be dry to the touch but not necessarily cured hard. The remedy for excessive handling damage selected by Michigan was to change the paint system to epoxy zinc-rich/epoxy/polyurethane. This system is more resistant to handling damage and is not likely to be applied too quickly. Michigan's decision to change from the inorganic zinc primer to the organic zinc primer was based on similar results of accelerated laboratory corrosion testing of both systems. *The fallacy here is that the inorganic zinc primed steel provides substantially improved corrosion resistance when it is allowed to weather two months or more before topcoating.* This condition was not present for the accelerated test panels where each coat was applied back-to-back in the laboratory. This difference in inorganic zinc system performance is well documented by NASA and some coating suppliers.

This change in coating procedure has had profound effects on costs of fabricated steel. In order to achieve a Class B surface for slip-critical connections, faying surfaces are primed with inorganic zinc. These surfaces must dry and then be masked from application of the coating system. This effectively adds a fourth coat in the shop and then requires additional field coating of the connection plates and fasteners, normally done during final paint touch-up. Field touch-

up should not require spot blasting and full system application but rather, a spot prime of epoxy mastic or similar high-performance surface-tolerant product, followed by a spot application of the finish coat. A glossy poly-urethane finish can be difficult to tie in uniformly and invariably will not look as good as its full coat application.

The quality of the field-applied topcoats over the inorganic zinc has little bearing on the long-term corrosion resistance of the system. Providing for their application in a better painting environment while eliminating the weathering of the inorganic zinc primer, or replacing it with an organic zinc primer, results in lower corrosion resistance. The most important coating, the inorganic zinc primer is still best applied in a controlled shop environment. It is damage resistant, has a Class B surface rating for slip-critical connections, maintains its corrosion protection for many years and does not have a finite recoat "window."

Many states have adopted the Michigan system in their new bridge construction specifications. This is understandable because Michigan had, and perhaps still has, the most comprehensive testing program for evaluating performance of coating systems in the development of their qualified systems list. The Michigan DOT materials laboratory has done a great service to our industry with its technical findings. Invariably there are circumstances where finish coating in the fabrication shop is prudent. It is important, however, to balance the costs and benefits of this approach and understand the history of this practice before making a wholesale policy decision. In summary, with a multi-coat shop system:

- Corrosion resistance is reduced from that of a shop-applied inorganic zinc/field-applied topcoat system.
- Fabrication costs are increased substantially.
- Field coating costs are not completely eliminated because of the need for touch-up.
- Aesthetics may be compromised because of the difficulty in blending and matching glossy topcoats during field touch-up.

REFERENCES:

Corbett, William D. "The Future of Bridge Coatings; A National Qualification System for Structural Steel Coatings," *Journal of Protective Coatings and Linings*, January 2004

Kline, Eric S. and William D. Corbett, KTA-TATOR, Inc. "Beneficial Procrastination: Delaying Lead Paint Removal Projects by Upgrading the Coating System," *Journal of Protective Coatings and Linings*, March 1992

Lowes, Robert. "Encapsulation. Make That Overcoating," *Painting & Wallcovering Contractor*, May–June 1993

"Illinois Saves $1.5 Million With Bridge Overcoat," *Better Roads*, July 1993

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MYTH: Bridges at the end of their calculated fatigue life or those experiencing localized fatigue problems have to be replaced.

REALITY: Fatigue life applies only to details. Localized fatigue problems can generally be fixed quickly and easily with no reduction in live load capacity or life of the bridge.

During the life of a steel bridge structure, certain details may exhibit fatigue cracking. These localized fatigue cracks do not mean that the entire structure has exceeded its service life. Many fatigue cracks can often be easily repaired by drilling holes at the tip of the cracks to stop crack propagation, if the driving force is removed or in other cases, bolting splice plates over the crack. After this retrofitting is performed, there is no reduction in live-load capacity or remaining service life of the bridge. Much of this retrofitting and repair can be completed without interrupting traffic.

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Much of the determination of the life of a steel bridge structure rests upon the methods of calculating the fatigue life. The current AASHTO "Guide Specifications for Fatigue Evaluation of Existing Steel Bridges, 1990" *gives the best available procedures for estimating the remaining fatigue life of a detail (not the entire bridge)*; that is, the number of years before substantial fatigue cracking will occur at that detail. It should not be assumed by designers that the fatigue life of a detail is over when it reaches the remaining life calculated by the procedures in these guide specifications. Actual lives of some details are expected to exceed the calculated lives by large amounts. There are large inherent uncertainties in predicting the fatigue life of a detail. There is a huge amount of scatter in fatigue test data, and there are uncertainties in calculating stress ranges and in estimating truck volumes. As mentioned previously, even if fatigue cracking has started at certain details in a bridge that does not necessarily mean that the useful life of the bridge is over since it may be feasible to make suitable repairs to these details. A good example of this is the Yellow Mill Pond Bridge on I-95 in Bridgeport, Connecticut. In 1970, cracks at the ends of cover plates were discovered only twelve years after the bridge was in service. An additional fatigue problem on another bridge was cracks that had developed along a longitudinal fillet weld used to attach a lateral connection plate to the edge of the floor beam flange. Grinding and peening the weld terminations repaired these cracks. A 1992 field inspection of these repairs to the Yellow Mill Pond Bridge indicated that no new cracks occurred in the cover plates or at the connection plates. The cost savings resulting from shot peening and grinding welds compared to the cost of replacing an entire structure is obvious.

REFERENCES:

Fisher, John W. "Executive Summary: Fatigue Cracking in Steel Bridge Structures," Center for Advanced Technology for Large Structural Systems, Lehigh University, *ATLSS Report No. 89–03*, July 1989

Fisher, John W., Yen, Ben T. and Wang, Dayi. "Fatigue of Bridge Structures — A Commentary and Guide for Design, Evaluation and Investigation of Cracks", Center for Advanced Technology for Large Structural Systems, Lehigh University, *ATLSS Report No. 89–02*, July 1989

American Association of State Highway and Transportation Officials. 1990. *Guide Specifications for Fatigue Evaluation of Existing Steel Bridges*

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MYTH: Modular prefabricated short-span steel bridges are only temporary structures.

REALITY: Modular prefabricated short-span bridges, as compared with so-called panel bridges, are typically permanent structures.

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Modular prefabricated bridges of the type shown in Illustration 1 started as portable structures but over the years have evolved as fully permanent ones. They meet all the standards of permanent structures such as AASHTO Specifications, ASTM material standards and the AWS Welding Code. Additionally, all welding is done in the shop under favorable conditions for quality control. Thousands have been installed for private companies as well as various Federal agencies, states, municipalities and counties.

Illustration 1: Santa Fe National Forest

MYTH: Modular prefabricated short-span steel bridges are limited to a one-size-fits-all scheme.

REALITY: Modular prefabricated short-span steel bridges are custom engineered to meet individual specific requirements.

Modular prefabricated short-span steel bridges rely on a concept of modular units that are bolted together in the field thereby eliminating any field welding and greatly increasing speed of construction. The modularity permits installation with small crews and light equipment, which also increases speed of construction and also lessens the impact on the environment. Beyond that each bridge is custom engineered to meet specific loading requirements and even skewed alignments and slopes.

MYTH: There are limited options with modular prefabricated short-span steel bridges.

REALITY: There are numerous options available.

Modular prefabricated short-span steel bridges can be designed with various guardrail (and railing) types and deck types including concrete (cast-in-place or precast modular), gravel, asphalt or timber depending on individual requirements. Furthermore they can accommodate any alignment whether straight, skewed or curved. Structural elements can be of painted, galvanized or weathering steel.

MYTH: Timber decks on modular prefabricated short-span steel bridges do not hold up and the timber treatments leach harmful chemicals into the environment.

REALITY: Treated timber continues to be used as a decking material for permanent bridge installations. Properly treated and detailed the treated timber will last the design life of the bridge. Timber treatment continues to advance with preservative choices and manufacturing techniques to minimize impact on the environment.

Treated timber has been used as a bridge deck material for generations. Like all materials, treated timber continues to evolve to provide better long-term performance. Individual plank decking has been supplanted by panelized-engineered deck systems. The deck systems are designed for all modern loading combinations. Built from glued-laminated or doweled-laminated wood, the deck designs include fastening systems to both connect the panels directly to steel stringers and to interconnect the panels (**See Illustration 2**). Improved details provide a deck often covered with an asphalt wearing surface for relatively maintenance free durability.

Illustration 2: Steel Superstructure with Treated Timber Deck and Rails

Crash-tested railing systems developed by industry and federal agencies attach directly to the deck panels. The entire system is shop fabricated for better quality control and either shop or field assembled allowing fast installation without the need of falsework.

More preservatives are being offered. Environmental impact with regard to the surrounding site and the facility user are a high priority in preservative selection. Preservatives must balance long term ability to fight decay while limiting the exposure of undesirable chemicals. Modern treating techniques follow Best Management Practices developed to fixate the chemicals in the wood and clean the wood surface prior to leaving the treating facility. Components are detailed to be fabricated prior to treatment thereby minimizing field cutting and drilling. This reduces the chance of cut-offs and saw dust from entering the environment.

Treated timber can be used as an engineered material with many complements to a steel structure.

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MYTH: Steel is not competitive for simple-span bridges less than 140 feet in length.

REALITY: Prefabricated modular steel bridges compete favorably with other materials when considering the greater use of shop labor vs. field labor, the speed at which they can be installed and the significant reduction in time required to close a given roadway to the public.

....

Shop labor is generally less expensive than field labor and it is easier to control quality than in the field. Shop-fabricated modular elements also increase the speed of construction. Illustration 4 shows an entire unit being placed with light handling equipment. The positive impact of speed can be summed up in the following points:

- If an owner is under court order to pay for environmental mitigation costs at a bridge site a prefabricated modular bridge could greatly lessen or even eliminate those costs.
- Shorter installation/construction time reduces the daily cost to an owner for traffic control, e.g., traffic control devices, maintenance of control devices, flagging, lighting, temporary roadways and maintenance of detours.
- Delay-related user costs (traffic delays, increased commuting times, increased mileage for use of detours) can be lessened by the use of prefabricated modular steel bridges. Although typically not the responsibility of the owner, reducing these costs benefits the motoring public significantly and helps the image of the owning agency.

Illustration 3: Promontory Bridge

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MYTH: Corrugated steel pipe or corrugated steel plate bridges do not last, as they tend to rust out.

REALITY: With proper attention to design details and appropriate coating one can expect a service life up to 100 years.

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Originally corrugated steel pipe and plate was only furnished with standard 2 oz. per square foot galvanized coating. In certain aggressive environments

(from aggressive soils or infiltration of road salts) galvanized material had a shortened service life. Now 4 oz. is readily available providing increased durability. The introduction of Aluminized Type II and polymer coatings has allowed corrugated steel material to be used in even more aggressive environments with a much extended service life. In fact, the National Corrugated Steel Pipe Association (NCSPA) has durability guidelines that show Aluminized Type II with a minimum service life of 75 years and polymer coated a minimum life of 100 years within certain environmental ranges.

Corrugated steel plate is supplied with a standard 3 oz. per square foot galvanized coating.

Another aspect of durability has to do with abrading of the invert from particulate material in the water. There are various ways of protecting against this such paving with asphalt or concrete or burying the invert in the ground. Another option is to use an arch or a three-sided box that avoids the streambed.

Regarding these coatings it should be noted that in the case of galvanized and Aluminized Type II there is a metallurgical bond between the base metal and the metallic coating and therefore it is impossible for the two to separate as in peeling. In the case of polymer coatings the polymer film is bonded to the galvanized coating before fabrication and meets AASHTO Standard M 246 for adhesion.

REFERENCES:

National Steel Corrugated Steel Pipe Association. *Corrugated Steel Pipe Design Manual*, 2007

National Steel Corrugated Steel Pipe Association. CSP Durability Guide, American Association of State Highway and Transportation Officials. *Standard Specification for Steel Sheet, Metallic-Coated and Polymer Pre-Coated, for Corrugated Steel Pipe, M 246-05*

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MYTH: Reinforced concrete pipe lasts forever.

REALITY: Concrete pipe is susceptible to deterioration from aggressive soils and road salts as well as lack of soil stability.

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There are many examples of reinforced concrete pipe bridges with spalling and wall deterioration due to aggressive soils or road salts attacking the concrete thereby exposing the reinforcing steel to corrosion. This can cause structural problems very quickly such as shown in **Illustration 5.**

Illustration 3: Corrugated Steel Pipe Retrofit of Concrete Box Culvert

Reinforced concrete pipe uses a stab-joint type joining method that is susceptible to gaps and openings if the bedding has any imperfections or there is differential settling or movement in the soil after installation. By contrast, corrugated steel pipe is manufactured with re-corrugated ends and uses corrugated external band couplers to provide a positive and durable connection. In fact, these joints provide the highest tensile and moment properties of any pipe system.

In the case of corrugated structural plate bridges the individual plates are connected with high-strength bolts uninterruptedly along the full length of the structure assuring continuity throughout the entire structure.

REFERENCE:

AASHTO LRFD Bridge Construction Specification, Section 26, Second Edition 2004

Illustration 4: Multiple Span Box Culvert

Illustration 5. In the case shown precast concrete footings were placed on the sides of each arch and the corrugated steel plate structure in preassembled sections was placed on the footings. *The space in between is completely free of any obstructions.*

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MYTH: Corrugated steel is flexible and not appropriate under high fills.

REALITY: Corrugated steel material for bridges has the ability to perform under fill heights exceeding 100 feet.

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Corrugated steel pipe and corrugated steel plate structures are flexible and gain their strength by transferring loads through the steel walls and into the surrounding side backfill zones. Thus they can work under fill heights exceeding 100 feet. As a rigid material, high-strength reinforced concrete pipe has to structurally carry its loads within its walls and can withstand loads imposed by fill heights less than half of that.

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MYTH: If corrugated steel pipe or corrugated steel plate is used as a bridge one is obliged to disturb the natural waterway.

REALITY: There are options available to avoid disturbing the waterway.

....

One way to liberate the natural waterway is to bury the structure invert in natural streambed material. This has to be done after installation. Another way that avoids disturbing the waterway altogether is to use a single arch or *series of arches* as shown in

....

MYTH: Corrugated steel pipe and corrugated structural plate bridges cannot compete with comparable reinforced concrete structures.

REALITY: Corrugated steel bridges compete very favorably with reinforced concrete bridges.

....

When comparing the cost of a bridge either of corrugated steel pipe, corrugated steel plate or reinforced concrete one must consider material lead times, shipping costs, installation cost and installation speed all of which contribute to the true total cost of a bridge. Corrugated steel pipe is light and quick to move into place. Corrugated steel plate, used for bridge applications requiring diameters up to 80 feet have an advantage of being able to be pre-assembled completely or in sections adjacent to the roadway and then placed with light equipment thereby minimizing roadway closure time. Reducing roadway closure time reduces costs for traffic control as described in a previous section. Even though these costs are not part of the bridge per se they nevertheless must be born by owner. When considering all these elements corrugated steel bridges compete very favorably with other materials.

MYTH: Although able to provide long-term protection under adverse environmental conditions, galvanizing on plate-girder and rolled-beam bridges is prohibitively expensive.

REALITY: Due to the relatively stable price of zinc metal over the past 20 years, the initial cost of hot-dip galvanized plate girder and rolled beam steel for bridges is very competitive with painted steel and even less expensive in many cases.

When life-cycle costs (initial + maintenance costs) are considered, the selection of hot-dip galvanizing for corrosion protection is even more compelling. Hot-dip galvanized steel is durable and maintenance-free for 50 to 60 years or more, while most paints require significant and costly maintenance at 10 to 15 year intervals, depending on the paint system selected. Unfortunately, the selection of a paint corrosion protection system by an architect, engineer, or project owner is often made based on the priorities of initial cost, historical preference, and established specifications, in that order. While all are important elements of the decision-making process, the painted project's life-cycle cost for the duration of the design life is often two to five times greater than the initial cost. This suggests the top priority in the analysis should be the determination of life-cycle costs.

Calculating life-cycle cost is a complex process because it requires the use of exponential financial equations that impute the time value of money. Simply put, life-cycle calculations put the cost of future maintenance in today's dollars, considering what inflation means to future value of money and what interest could be earned on the money used. To simplify the calculation process, the American Galvanizers Association developed a life-cycle cost calculator using paint cost data (See footnote 1) and average galvanizing costs (See footnote 2) collected in national surveys. This calculator is available at: *www.galvanizingcost.com*.

To demonstrate the advantages of using hot-dip galvanized coatings to protect girder and beam steel, consider the following data for a typical project:

100 ton project	C-3 industrial environment	250 ft² per ton of steel
50-year service life	shop applied by spray	3% inflation/6% interest
$0.22 US/lb. (See footnote 3)	$0.516 CAN/kg (See footnote 3)	

Using these data as input to the Life-Cycle Cost Calculator (www.galvanizingcost.com), the tables below compare initial cost and life-cycle costs for hot-dip galvanizing with three common and comparable paint systems.

Initial and Life-Cycle Cost (LCC) Comparison of HDG to Paint Systems

UNITED STATES

SYSTEM	INITIAL COST/ft²	TOTAL LIFE CYCLE COST	LCC COST per ft²	AEAC A per ft²
HDG	$1.76	$ 44,000	$1.76	$0.11
IOZ/epoxy/polyurethane	$3.07	$189,346	$7.57	$0.48
Epoxy/Epoxy	$1.97	$181,845	$7.27	$0.46
Epoxy/Polyurethane	$2.17	$205,284	$8.21	$0.52

CANADIAN

SYSTEM	INITIAL COST/m²	TOTAL LIFE CYCLE COST	LCC COST per m²	AEAC (See A below) per m²
HDGB	$21.11	$49,029	$21.11	$1.34
Inorganic zinc/epoxy	$23.01	$143,217	$61.65	$3.91

A   Average Equivalent Annual Cost

B   1 US dollar = 1.06933 CAN dollar

FOOTNOTES

[1] KTA Tator, *Expected Service Life and Cost Considerations for Maintenance and New Construction Protective Coating Work*, Helsel, Melampy, Wissmar, 2006

[2] American Galvanizers Association, National Survey, 2006

[3] May vary due to regional differences, processing schedules, and London Metal Exchange price of zinc metal.

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