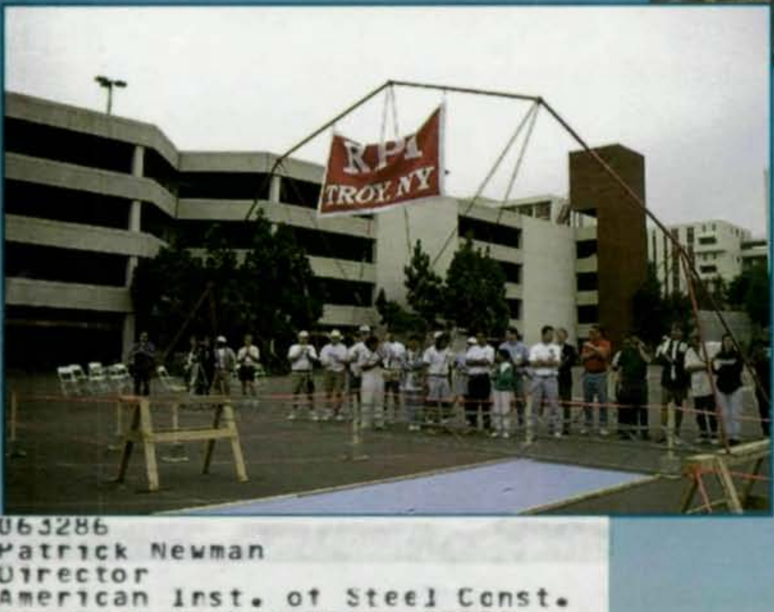
MODERN STEEL CONSTRUCTION

September 1994

$3.00

Bridge Construction



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Common weld washers are made of 16 gage material (0.0598") and have a 3/8" diameter hole. The weld should overfill the hole to produce a visible weld diameter of about 1/2". **Typical 3/8" x 16 gage washers are not recommended with deck design thicknesses equal to or greater than 0.028" (22 gage). Weld washers are recommended for welding in deck panels thinner than 0.028".**[1] Most form deck products are furnished in metal thicknesses ranging from 0.0269" (23 gage) to 0.0149" (28 gage) and therefore weld washers are recommended. The table is based on typical form deck material (grade E) such as used by **United Steel Deck, Inc.** to produce **UFS, UF1X, UFX-36, and UF2X** which are form decks ranging in depth from 9/16" to 2" - a catalog is available on request. A 70 ksi electrode was used for the tables.

Deck Gage	Thickness	Design Uplift (Tensile) Value, lbs. [2]			IV
		I  Weld washer/ one deck thickness.	II  Weld washer/ two deck thicknesses.	III  Weld washer/edge lap (at support)-- weld is eccentrically loaded.	Shear Value, lbs. [3] Shear strength values are based on a safety factor of 2.75.
28	0.0149"	740	850	510	435
26	0.0179"	760	890	530	565
24	0.0239"	810	980	560	855
23	0.0269"	830	950	580	1030

Values in I, II, and III are based on a safety factor of 2.5 but includes a 33% increase for wind loading.

(1) Luttrell, L.D. (1993),"Arc Puddle Welds and Weld Washers for Attachments in Steel Deck", Steel Deck Institute, P.O. Box 9506, Canton, Ohio, 44711.

(2) LaBoube, R.A. and Yu, Wei-Wen (1991), "Tensile Strength of Welded Connections", Final Report, Department of Civil Engineering Center for Cold Formed Steel Structures, University of Missouri-Rolla, Rolla, Missouri, 65401.

(3) Luttrell, L.D. (1987), "Diaphragm Design Manual, Second Edition", Steel Deck Institute, P.O. Box 9506, Canton, Ohio, 44711.

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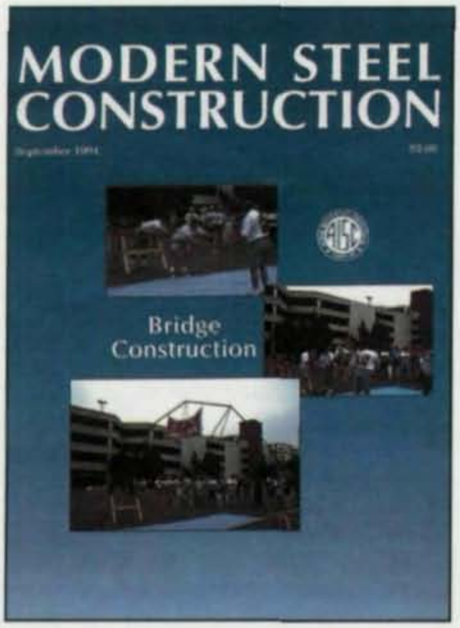

The student team from RPI took first place in this year's National Student Steel Bridge-Building Competition. The story behind these hard-working students begins on page 18.

FEATURES

DEPARTMENTS

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A Level Playing Field

FOR MUCH OF THE PAST DECADE, I'VE PLAYED ON A RECREATIONAL SOFTBALL TEAM. We recently ended our summer season on a down note: We lost to a team we had beaten twice a summer for at least the last five years. The loss was pretty inevitable. The game fell on a bad weekend for us; many of our top players were out of town—in fact, we could only field nine players versus the normal 10 on a softball team. In that respect, the playing field clearly was not level.

Too often, the playing field also isn't level in the competition between steel and concrete bridge designs. Probably the two biggest areas where steel is penalized are in the bearings and in the substructure. The former is the more common problem. Whereas concrete bridges are almost always designed with inexpensive, simple elastomeric bearings, steel bridges too often are designed with expensive, complex pot bearings or weldments.

Inequitable substructure design is an even more expensive problem. When bids are received for steel and concrete bridges, it is important to consider not only superstructure costs, but also substructure savings. Because steel bridges are lighter than their concrete brethren, substructure costs are often substantially less for steel bridges. Because bridge contracts are awarded on total costs, it is unreasonable to assume that the same substructure would be used on both the steel and concrete alternates.

In his article on post-tensioning steel bridges (see page 38), Leo Spaans, a designer best known for his concrete bridges, addresses some of these problems. As Spaans states: "In many alternate bid situations, the substructure for the steel alternate has the same number of piles and the same amount of reinforcing steel as the concrete alternate—despite the fact that the steel alternate is considerably lighter."

Another way for steel to increase its competitiveness is to make sure that designers and detailers are using the most cost-effective details. To help, Walter Gatti, one of the country's most respected detailers, offers more than a dozen examples of good and bad details, with explanations of what can be done to reduce costs (see page 24).

Only when the best designs in both steel and concrete are presented will the playing field be truly level. And that's a winning situation for everyone. **SM**

STEEL INTERCHANGE

Steel Interchange is an open forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine. If you have a question or problem that your fellow readers might help you to solve, please forward it to *Modern Steel Construction*. At the same time, feel free to respond to any of the questions that you have read here. Please send them to:

Steel Interchange
Modern Steel Construction
One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001

Answers and/or questions should be typewritten and double-spaced. Submittals that have been prepared by word-processing are appreciated on computer diskette (either as a Wordperfect file or in ASCII format).

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principals to a particular structure.

Information on ordering AISC publications mentioned in this article can be obtained by calling AISC at 312/670-2400 ext. 433.

The following responses from previous Steel Interchange columns have been received:

In designing composite steel girders in accordance with the LRFD Method, it has been well established that significant reductions in beam sizes can be achieved. However, in my experience, I have found, in some cases, the most economical girder sizes may be unsafe during unshored wet concrete construction. This can occur when the metal deck runs parallel to the girder and, in my judgement, does not afford significant lateral restraint to the top flange of the girder in compression. For this condition, the unbraced length is the spacing between the beams supported by the girder. Significant reduction in the non-composite moment capacity can occur due to lateral torsional bucking which may not be adequate for the unshored wet concrete construction.

No criteria for this serviceability problem or guidance appears to be given in the LRFD specification. I would like to know whether there has been any testing or research to demonstrate that metal deck, parallel to the girder does indeed provide adequate restraint or that checking the beam size for the temporary construction condition, should be carried out as outlined above.

The optimization of composite steel plate girder design increases the potential for both flange and web buckling to occur during construction. Higher-than-allowable compressive stresses and resultant buckling are almost certain to occur in these girders if stability during construction is ignored by the designer. This problem is not limited to those girders designed under the LRFD Specification. Since the use of composite construction became prevalent, engineers have identified the problem of instability in steel girders during construction.

In failure cases observed in Pennsylvania, metal deck pans provided insufficient bracing against lateral flange buckling at critical sections. Evidence indicates that the failure of the welds connecting the pans to the flange can be expected to occur prior to flange buckling. Although increasing the capacity of this connection may be considered, the designer should be concerned about the quality control of such a critical connection, as well as the associated cost-effectiveness of this approach. More significantly, metal deck pans will do nothing to prevent excessive web buckling which is as likely to occur (but not as likely to be detected) in the compressive region of unstiffened webs.

AASHTO's Standard Specifications for Highway Bridges, 15th Edition, addresses the problem of construction instability in Article 10.50, by limiting both the compression flange shape factor (b'/t ratio) and the lateral-torsional buckling moment capacity (M_r) for composite girders subject to non-composite dead loads. In some instances, states have introduced their own criteria for checking this condition, which are typically more conservative than AASHTO. Pennsylvania, in particular, has gone to great lengths to develop their own design parameters. In general, the AASHTO criteria is a widely accepted check for the stability of girders during construction.

The AASHTO criteria can be met by reducing the length of deck pours, increasing the size of the steel girder section, reducing the distance between lateral brace points (i.e. diaphragms or cross beams), or by a combination of these methods. Limiting the length of deck pours is critical for continuous girders, where temporary positive moments from the wet concrete may be much larger than the final positive dead load moment that will exist after the entire deck has been placed.

The methods used to mitigate construction-related stresses should be determined by the designer after consultation with contractors and fabricators as to the economics of the various alternatives. When the methods chosen require control of the construction process (i.e. deck pouring sequence), this should be clearly indicated on the construction plans. As the question accurately illustrated, the need to address construction-induced stresses will become more critical as we continue to optimize and refine our design methods.

Daniel G. Faust, P.E.
Delaware River Port Authority
Camden, NJ

The use of channel sections or other lightweight narrow flange sections as girts supporting non-bearing exterior wall assemblies against wind load is common practice. How is lateral instability of the unsupported compression flange accounted for when the wall is subject to outward pressure due to suction at the leeward face of the building? These outward forces are equal to or greater than the inward forces.

There are two basic approaches to this problem. The answer depends on the designers degree of conservatism in treating the role of sag rods that are often required to minimize the excessive girt sag under own weight.

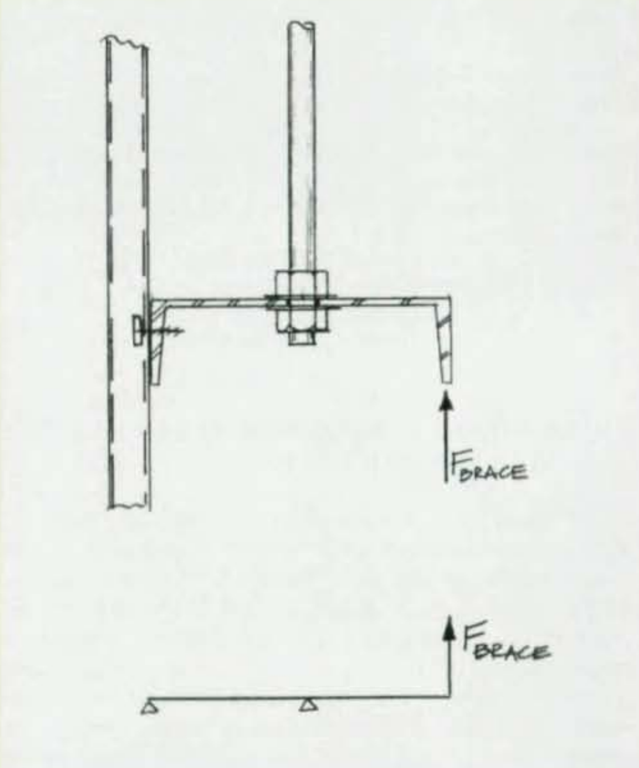
One approach, obviously a very conservative one, considers the interior flange as completely unbraced from column to column. The channel sections designed under this scenario are usually so heavy that it often makes sense to use wide flange girts instead.

Another approach recognizes a restraining action of sag rods and considers the channels laterally supported at each sag rod location. The number of sag rods may have to be increased to provide enough bracing points to maximize the allowable bending stresses in channels (AISC ASD Spec. Chapter F). This seemingly unconservative approach has been used for decades and withstood the test of time.

An attempt to rationalize this practice can be made as follows. For the channel girt to buckle under wind suction loading, its interior flange must move vertically. At the point of the sag rod attachment this movement is prevented, as it is at the exterior flange stabilized by the wall siding fasteners. It is widely recognized that the compression flange of a flexural member may be considered braced if the brace can resist a force of about 2% of the flange compression. If this force can be developed by the web cross-bending, the required bracing is present.

The figure illustrates the assumed model of the web acting as a cantilever beam. The width of the web effective in this action is determined by engineering judgement.

It is worth mentioning that cold-formed C- or Z-girts may often be more economical than structural



channels. These members are usually continuous over the column supports, since lap splicing can be easily made, and the points of inflection are frequently assumed to be braced laterally.

Alexander Newman, P.E.
Maguire Group, Inc.
Foxborough, MA

New Questions

Listed below are questions that we would like the readers to answer or discuss.

If you have an answer or suggestion please send it to the Steel Interchange Editor, Modern Steel Co nstruction, One East Wacker Dr., Suite 3100, Chic ago, IL 60601-2001.

Can threads on anchor bolts be either rolled or cut? Is one method better than the other?

What is the most efficient and cost-effective way to connect a steel wide flange girder to a concrete column?

Jake Roth
Roth Metal Works, Inc.
Brooklyn, NY

Practical Design Takes Center Stage At NSCC

Lawrence G. Griffis

WHILE NOT AN OFFICIAL THEME, THE UNDERLYING SUBJECT AT THIS YEAR'S National Steel Construction Conference was clearly "Designing buildings that work." This tone was set initially by Dan Cuoco, P.E., who stated that good design doesn't stop at the drawing board, but rather that good design must include ease of constructability.

The conference, held May 18-20 in Pittsburgh, attracted nearly 700 designers, fabricators and educators. Cuoco, a principal with Thornton-Tomasetti Engineers in New York City, opened the conference's first session with a detailed description of the design and construction of the Anaheim Arena and the Chicago Stadium (see May 1994 MSC for more on these projects). One of Cuoco's key points was that in designing a structure, an engineer must consider how the project can be fabricated and erected—a process that he dealt with extensively on the two projects described in the session.

Following Cuoco, Lawrence G. Griffis, P.E., this year's T.R. Higgins award winner, presented a paper on composite construction. Griffis, senior vice president and director of structural engineering for Walter P. Moore and Associates in Houston, is the first design engineer to win the award since 1989. His paper included an overview of the history of composite construction, advantages and disadvantages, design responsibility, design considerations (such as wind load, diaphragm action, drift criteria and differential column shortening), and the use of LRFD in composite column design. He then concluded with several case studies of highly successful composite projects designed by his firm and a brief discussion of the future of composite construction. A condensed version of his paper is scheduled to appear in the October issue of MSC. Also, Griffis will give at least six lectures around the country this fall and winter.

The general sessions during the next two days were equally exciting. On Thursday, Leslie E. Robertson of Leslie E. Robertson Associates, New York City, and John Daly of Karl Koch Erecting Co., kicked off the days events with a fascinating discussion of last year's World Trade Center explosion. Using photos and a videotape, the two men walked their audience through the catastrophe and explained how the building was so quickly put back into operation. "It was the largest incendiary device in U.S. history," according to Robertson. Despite that, the damage to the steel structural system was superficial. One diagonal was blown off completely, another was bent, and a large steel box column had a hairline crack. Instead, most of the damage was to non-structural elements, such as piping, conduit and concrete slabs and walls. Though the structural system was left intact, the bomb did rip a 120-ft. wide hole in the floor and even devastated the floor three levels above the explosion.

The structural system remained completely stable, so much so that the building reacted to a fierce wind storm just days after the bombing exactly as it had during similar storms prior to the bombing. This wasn't surprising since the tower was designed to withstand the maximum possible windstorm—a design load four times greater than the design requirements in an earthquake zone such as Los Angeles.

Interestingly, Robertson pointed out that the tower had been designed to withstand sabotage of the perimeter columns—as well as the impact of a fully fueled 747 jumbo jet.

On Friday, Robert Nickerson of NBE Ltd., Hampstead, MD, presented a discussion on the Myths and Realities of Life Cycle Costs, a greatly condensed version of a session he has been presenting to state bridge departments around the country for the past year. Nickerson was followed by Michael Engelhardt, a professor at the University of Texas at Austin, who discussed the steel moment connection failures discovered after the Northridge Earthquake (see June 1994 MSC for an extensive commentary on this subject).

STEEL NEWS

Hot Seminars

THE MOST POPULAR SEMINAR DURING THE THREE-DAY CONFERENCE was "Steel Interchange—Connections," which featured Geoffrey Kulak from the University of Alberta and Omer Blodgett from The Lincoln Electric Co. The two men each began the session with 20 minute presentations on bolting and welding, respectively, and then took questions from the audience for the rest of the session, which attracted a combined audience during two separate presentations of more than 200.

Another hit was "Lean Engineering" with Mark Holland of Paxton & Vierling Steel Co. and Samir A. Lawrence from The Ralph M. Parsons Co. The session showed how projects can be improved through a close working relationship between a fabricator and engineer. As Lawrence put it, "Lean engineering...embraces and promotes productivity through reengineering the work process, intrinsic motivation and technology." He urged that engineers and fabricators work closely together in a partnering agreement, including: agreement on the capabilities of the detailing programs by incorporating these capabilities in the design; agreement on all pre-approved standard connections; usage of AISC Connections per Volume II (ASD or LRFD); and continuous coordination between CAD systems and the detailing program to reduce the use of non-intelligent graphics in order to automate the shop drawings further.

Other well-attended sessions included: Lawrence G. Griffis from Walter P. Moore and Associates and Robert J. Wills, Jr., from AISI (see May 1994 MSC) discussing "Wind Damage and Design Load Requirements;" "Electronic Data Transfer" with Sayle Lewis of Fluor Daniel and Harry Moser of DuPont; and "Building Innovations" with Tom Sputo, a Florida-based consulting engineer (see May 1994

MSC), and Neil Wexler of Neal Wexler, P.C. (see April 1993 MSC).

Busy Exhibits

AS ALWAYS, THE EXHIBIT HALL, WITH MORE THAN 60 EXHIBITORS, generated a lot of interest—and not just because of the large fabrication equipment on display. One of the most interesting displays was for a product called SCATS (Standard Commodity Accounting and Tracking System). This PC-based system uses bar code technology to produce current status reports on steel products—from the fabricator to the galvanizer to the erector. For information on this product, circle #81 on the reader service card near the back of this magazine.

Another fascinating new product was a pen-based detailing system from CadVantage called, appropriately, PENVANTAGE. This graphical interface works with the CadVantage Structural Detailing System and allows users to enter data with a pen and tablet. Even for touch typists, entry is much faster. The company had a working version operating at the conference that grabbed a lot of attention. For information, circle #45.

Several other vendors also were showing software for fabricators. Baresel Corporation, a three decade old detailing firm, was demonstrating Compu-STEEL, its Windows-based detailing system featuring extraordinarily clean and easy-to-use interfaces. For information, circle #82. Other software vendors included CDS (circle #63), Structural Software Co. (circle #42), Cadex Oy (circle #83), Dogwood Technologies (circle #96), EJE Industries (circle #46), SteelCAD International (circle #84), Timberline Software (circle #85) and Softdesk (circle #86).

While numerous detailing software vendors demonstrated their products, very few structural engineering software producers showed their wares. Ram Analysis demonstrated the latest

version of RAM-STEEL (circle #41) and Metrosoft showed off their Robot V6 system (circle #51).

Perhaps the biggest category of exhibitors were fastener manufacturers. Garnering the most attention was Huck International, which showed the prototype of its blind bolting system (see February 1994 MSC) called Ultra-Twist, which is to replace welding for joining structural tube sections (circle #87). Other bolting and fastener exhibitors included: St. Louis Screw & Bolt (circle #36); LeJeune Bolt Co. (circle #73); Mid South Bolt & Screw (circle #74); NSS Industries (circle #88); Lohr Structural Fasteners (circle #89); Atlas Bolt & Screw Co. (circle #90); Haydon Bolts (circle #91); Elco (circle #92); and ITW Buildex (circle 93).

In addition, several related products were on display, such as J&M Turners direct tension indicators (circle #94) and elastic tension indicators from Universal Loading Spring (circle #95). On the welding side, only The Lincoln Electric Co. was present (circle #97).

Disappointingly, very few paint manufacturers were present. Con-Lux Coatings offered literature on its range of paint products, including a handy Pocket Paint & Coatings Applications Guide (circle #98), and Southern Coatings offered a wide variety of literature (circle #99).

A related booth was one for The Wheelabrator Corp., which produces surface preparation machines for plate, structural steel and tubing (circle #100). While Wheelabrator did not have an operational machine on display, though several other large equipment manufacturers did, including Behringer Saws (circle #101), Nitto Kohki (circle #102), Daito Seiki Co. (circle #103) and Peddinghaus (circle #104).

Next year's conference will be held in San Antonio on May 19-21. For information on the upcoming show, circle #4 on the reader service card in this magazine.

Northridge Update—Part II

The number of steel buildings structurally damaged by the Northridge Earthquake is now estimated at more than 100, though there were no collapses and most of the buildings remain functional.

Last month we discussed short-term research. Medium- and long-term research also is planned.

Medium-Term

A national workshop on seismic steel building research needs has been proposed by AISC to the federal agencies, affiliated steel industry organizations, researchers and the design profession for this fall. A panel of about 60 to 100 expert participants will be invited for two days of break-out sessions to identify specific research problem statements, their estimated time and cost requirements. The ensuing workshop report will establish the consensus priorities and coordinated needs for seismic steel research for the longer-term.

NSF previously announced a post-Northridge research program. Many proposals have been submitted on a variety of topics (including all building materials).

Long-Term

It is expected that in six to 18 months, at least some of the recommendations contained in the published workshop report will be funded through a combined pooling of federal, industry and professional resources. These are expected to be major, multi-year undertakings that will target such issues as overstrength levels and ductility demands for steel structures, fast strain-rates, floor slab and column load effects, alternative ductile moment frame systems, semi-rigid (PR) design, lateral framing redundancy, role of drift control, seismic force code limits and other design assumptions.

In all the Northridge follow-up activities, AISC has been working closely with the Structural Steel Producers Council and AISI. The steel producers have been supplying the steel and fabricators are producing the test specimens. Contacts have been established with the National Institute for Standards & Technology, Structural Engineers Association of California, and other agencies for their input and assistance.

Learning The Ins And Outs Of Bridge Design

Students from RPI erected their winning bridge in less than eight minutes

WITH A NOD, THE CONSTRUCTION SUPERVISOR SIGNALED THE CREW TO BEGIN ERECTION of a replacement bridge across a raging river. The action was fast and furious—and finished in just over 2 minutes. No, it's not a scene from an Ayn Rand novel. Rather, it was the national finals of the Student Steel Bridge-Building Competition held this past May at San Diego State University.

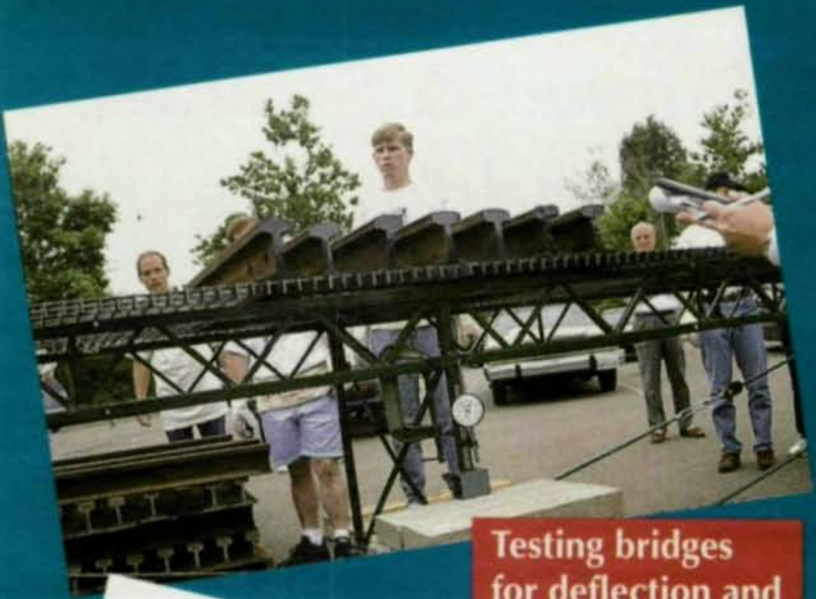
The competition, sponsored by AISC and ASCE, is open to all ASCE student chapters. Student teams design, fabricate and erect a 1:10 scale model of a replacement bridge across a "river valley in a mountainous rural region." The bridges are judged on stiffness, lightness, construction speed, aesthetics, efficiency and economy. The bridges are built atop two construction horses 20' apart with a blue-paint, 7-ft.-wide river running in between. A winding "road" on each side provides the only access to the construction site from a staging area. Students are penalized for leaving the access road or touching "water". The bridges are measured for lateral load against a 100-pound force and a vertical load of 2,500 lbs.

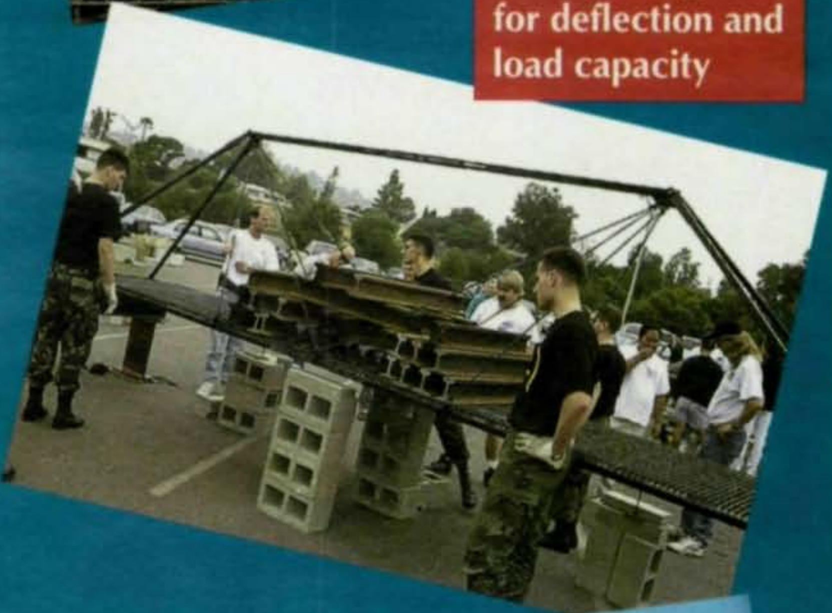
"THIS IS THE PREMIER STUDENT ENGINEERING EXERCISE," according to Michael Orlandella, a professor at Southern College of Technology in Marietta, GA, whose students took second place at the nationals for the third consecutive year. "The biggest thing is that students learn about the management of a project—how to put it all together. They need to find sponsors, design the project, fabricate it, and erect it. They have to write real letters to professionals. And they learn how to work with steel—the student who did the welding on our bridge had never welded before." It's a real-life exercise of the skills taught in steel design classes.

The student team from Oregon State University, which finished fifth in the 21-school competition, was typical. "We started with four different designs, before settling on this one," said Steve Trautwein, a graduating senior and the project leader. "We initially considered a king post, but we realized it would be difficult to erect since the top of the bridge would be in the middle of the river." **ERECTION CONSIDERATIONS PLAYED AN IMPORTANT ROLE IN THEIR DESIGN,** which ultimately consisted of 25 pieces (according to the rules, no piece could exceed 40 lbs. or overall dimensions of 5-ft.-6-in. x 7.5-ft. x 7.5-ft.) and four cables.

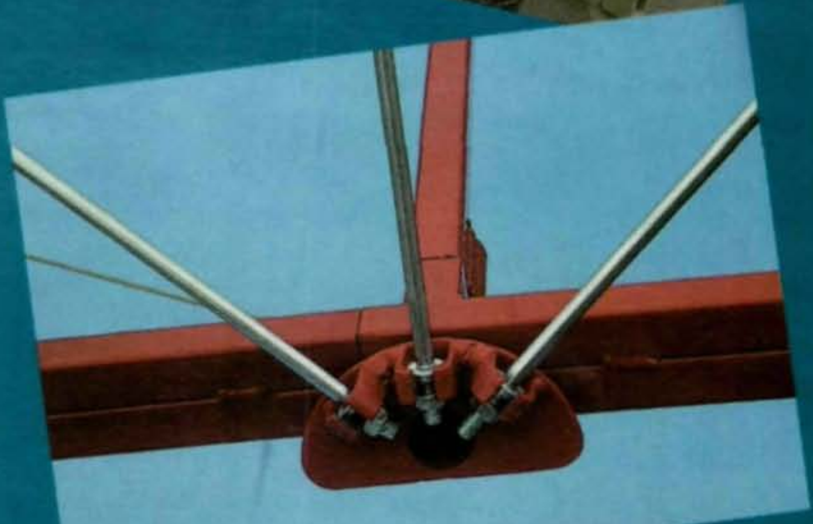
Students from the State University of New York at Buffalo, which finished tenth, also gained a new appreciation of connection details. "You can put anything on paper, but it won't be successful if it can't be put together in the field, said Chris Ballard, the team leader. "Originally, we had all male-female connections since they could be erected so quickly, but then we discovered it was impossible to erect the last piece!" Learning doesn't stop once the bridge is designed and erected for the first time. Buffalo's skewed arch bridge was designed as a truncated pyramid and had taken second place in their regional competition held a few months earlier. "Since then, we took 10 lbs. off of it, improved our erection speed, and cut down on our deflection." The bridge ultimately weighed 289 lbs. and deflected only .27 in. under a 2,500 lb. load.

By comparison, the lightest bridge, Michigan State University's, weighed only 84.5 lbs., and three other bridges,





Testing bridges for deflection and load capacity



A fast erection connection

Southern Tech's innovative bridge took second place

from Southern College of Technology, Oregon State University, and Portland State University, weighed 89, 90 and 99 lbs., respectively. The winner in the construction speed category was Southern College of Technology, which took 10.28 man-minutes (a raw speed of two minutes, three seconds multiplied by the number of workmen plus any construction penalties), followed closely by the N.J. Institute of Technology at 12.45 minutes and the University of Kansas at 16.71 minutes.

For stiffness, the winner was the University of New Hampshire, which deflected only 0.195 in. under a 2,500-lb. load, followed by Rensselaer Polytechnic Institute (0.227 in.) and San Diego State University (0.239 in.). For efficiency (calculated as incremental vertical deflection divided by 0.10 plus the total weight divided by 100), RPI took first with a score of 3.44, followed by San Diego State at 3.66 and Portland State University at 3.74.

In the economy category (calculated as total weight time 1000 plus construction time times 3000 plus penalty costs), the winner was the Southern College of Technology at \$119,845, followed by Oregon State University (\$143,398) and the University of Florida (\$161,248).

Based on the above mentioned categories, RPI and Southern

A failure can be a learning experience too

College of Technology tied, with RPI winning first place overall based on a higher aesthetic ranking.

IN ALL, 21 SCHOOLS COMPETED IN THE NATIONALS OUT OF THE MORE THAN 140 SCHOOLS THAT COMPETED IN 17 REGIONAL COMPETITIONS. Students came from as far away as New York and Florida, and both West Point and the Air Force Academy were represented. And while males dominated the competition, at least five of the schools included female team members.

Judges were recruited from a large number of local engineering firms, including McDaniel Engineering, Boyle Engineering, Barrett Consulting Group, RHG Wong, R2H Engineering, Northrup, and the California Regulation Water Quality Control Board.

"I think this competition will make me a better engineer," said Ballard, who, along with his teammates, missed graduation day exercises in Buffalo to be at the competition. "I can now identify with the construction sequence and I have a better understanding that good designs have to look good after fabrication and not just on the drawing board."

And the drawing board process can be very involved, as students at Cal State—Long Beach discovered. "We did three different designs and analyzed them all using Algor and RISA 2D software to find the one with the least weight-to-strength ratio," said Tim Slegers, one of the four-man team from Long Beach, which finished in the middle of the pack at twelfth place. **"IT WAS GREAT TO LEARN HOW A PROJECT IS DONE—FROM START TO FINISH.** And we'll pass some advice on to next year's team. For example, we know we can save some time by using slip joints."

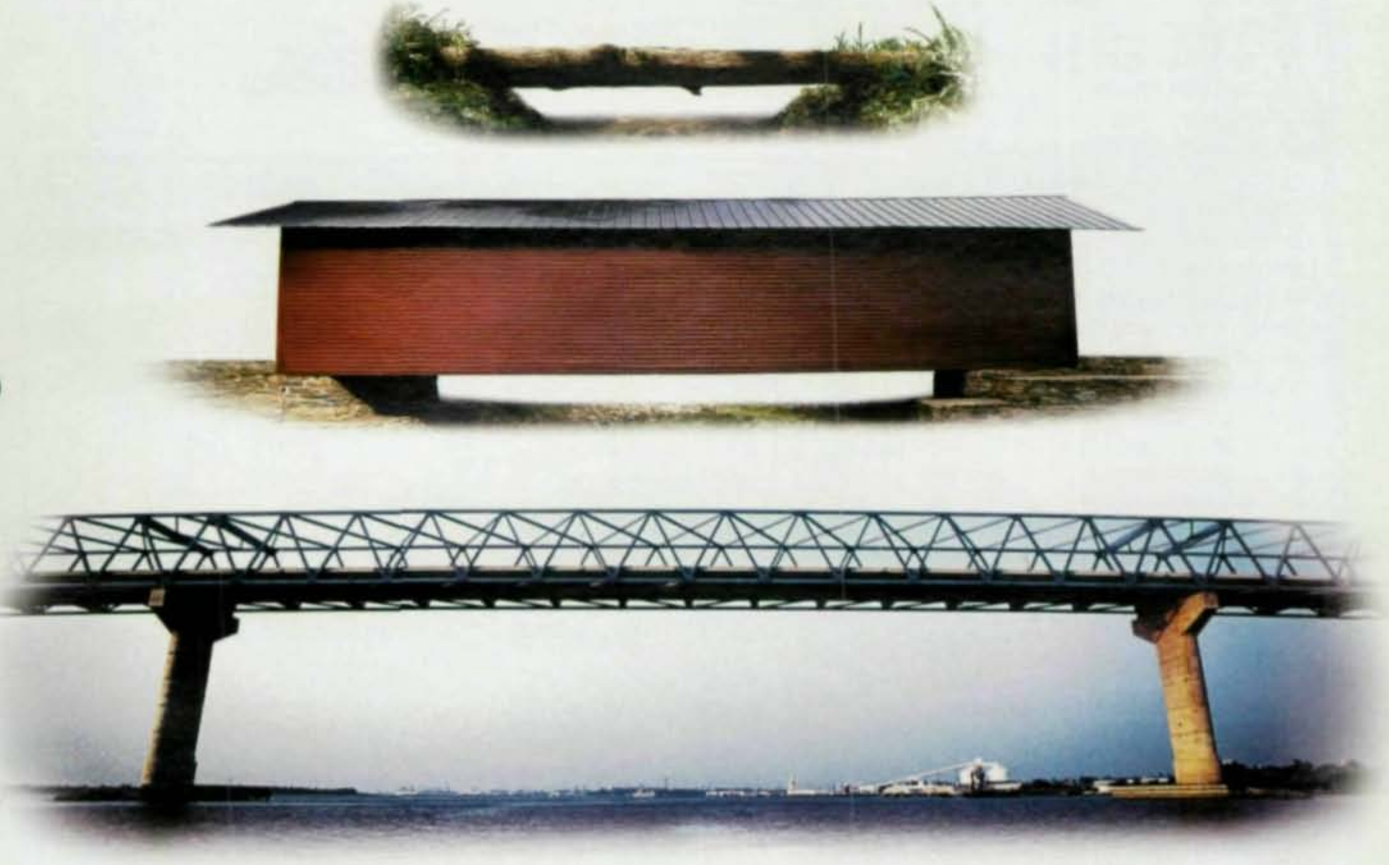
The idea of seeing something that they designed spring to life was an important aspect for many of the students. "This was the first time I saw something go from my head to AutoCAD to completion," explained Kevin Kasner, student leader from San Diego State University and one of the competition's organizers. "Working as a team—and learning how to get the most out of the team—was also a great experience."

Of course, not everything went perfectly. Michigan State University's team knew their bridge was right on the edge—it had failed three times under different test conditions. And it failed again as the loading neared 2,500 lbs. "It was still a great learning experience," according to Guy Nelson, a senior at MSU. **"WE LEARNED A LOT ABOUT BUCKLING, CONNECTIONS AND FABRICATION."**

Oregon State's Steve Trautwein echoed that opinion. "Two years ago, our school's bridge failed. The guy who designed it said he learned more in those two seconds than he had in any design class. You can design on paper and it looks good, but as soon as it fails, you realize you really need to pay attention to details."

If you're interested in watching a regional competition featuring your alma mata, either contact that school's engineering department or AISC's Fromy Rosenberg at 312/670-5408 (fax: 312/670-5403).

Fortunately Some People Are Never Satisfied.



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LS-B4(7/94)

Economical Steel Bridge Details

A look at what does and doesn't work based on 35 years of detailing experience on more than 2,000 bridges

By Walter Gatti, President
Tensor Engineering Co.

(This information was originally presented at the 1993 National Symposium on Steel Bridge Construction.)

At one time, reducing steel construction costs meant finding the least weight design. And today, the very sophisticated computer programs that every engineer has at his disposal often include an optimization routine that will reduce the weight of the structure by selecting the minimum size of material.

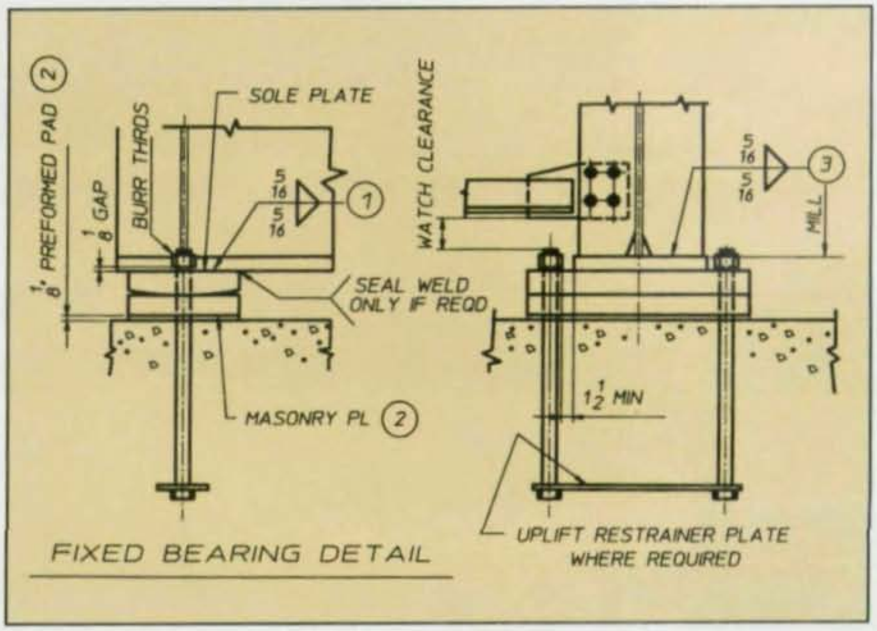
Unfortunately, least weight no longer necessarily equates to lowest cost. As steel material prices have declined, fabrication costs (i.e. labor costs) have risen. Today, an economical design includes low weight, but also accounts for the constructability of details.

Fabrication costs for detail material are six times more expensive per pound than main material. Every piece that is not rectangular will require a hand-made tracing template, layout on the piece, or maybe a burning table program. All of these are high-cost items. Every hole must match a hole in another piece; minor changes in the design or layout of connections in order to standardize detail pieces can lead to major savings in fabrication and erection costs.

During the past three-and-a-half decades, Tensor Engineering Co., a detailing firm with a national practice, has detailed more than 2,000 bridges and more than a million tons of structural steel. In the last decade alone, the 22-man firm has provide fabrication and erection drawings for more than 400,000 tons of steel.

We are constantly trying to simplify details in order to eliminate those that breed mistakes. Our knowledge has been acquired through layers of scar tissue and unpleasant phone calls from fabrication shops or field erectors. Through this exposure, however, we have developed a large amount of information about which details work—and which don't.

Detail 1: Fixed Bearing Detail



Fixed Bearing Detail:

1. Weld sole plate to flange in longitudinal direction only.

2. Wherever possible, use an elastomeric pad in lieu of a masonry plate. This is the same simple detail that works well for concrete girder bridges.

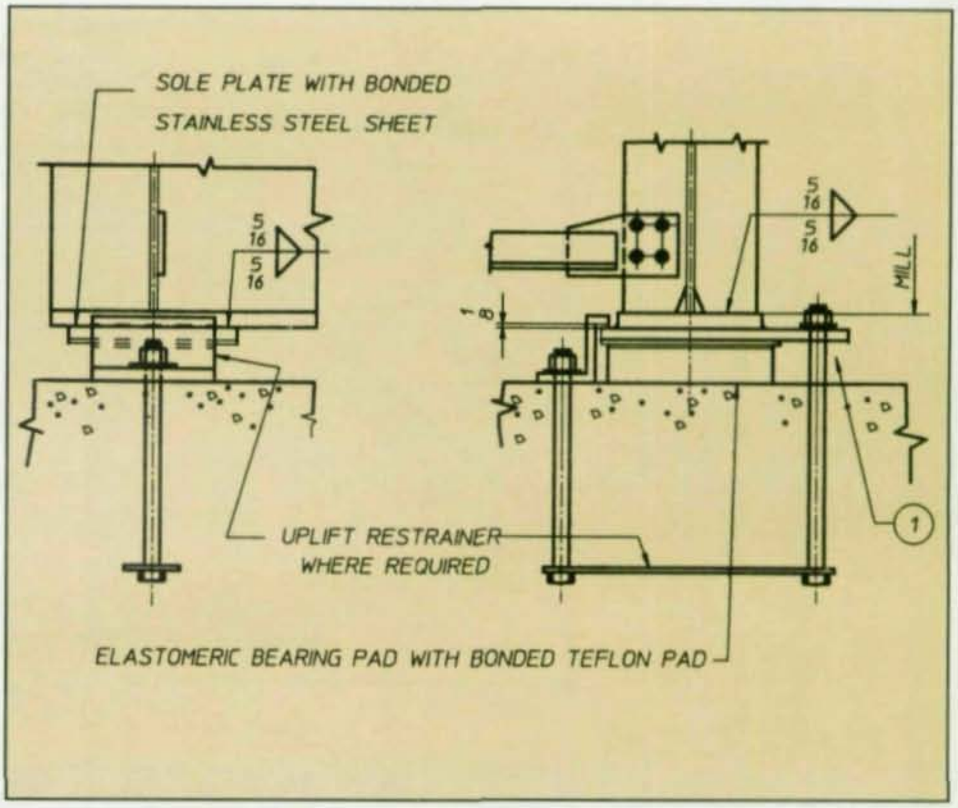
3. Fabricators prefer to mill and fillet weld bearing stiffeners to flanges rather than use full penetration welds, which require joint preparation, multiple weld passes and non-destructive testing—all of which greatly increase costs. Also, full penetration welding causes distortion of the bottom flange and an out of plane bearing area. ***Avoid full penetration welds where possible.***

Expansion Bearing

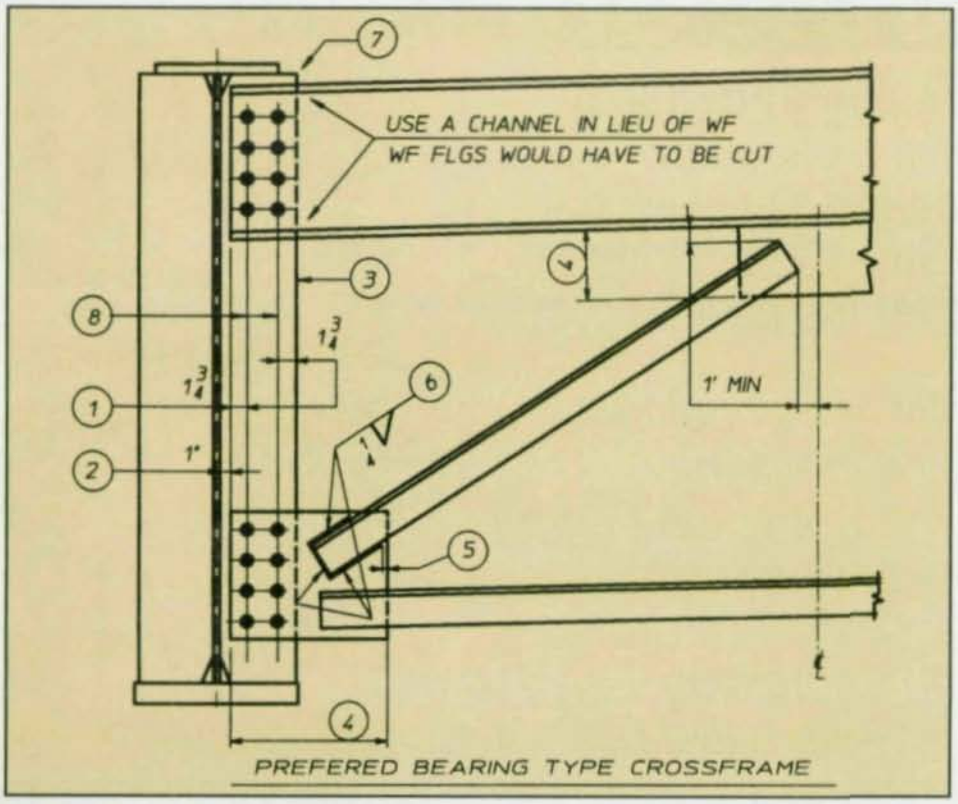
1. When uplift restrainers are not required, anchor bolt would project through the sole plates. The sole plates should have slotted holes to allow for movement.

2. Again, concrete girders only have an elastomeric pad under each bearing, regardless of span length—a very cost effective design. Steel girders can—and should—be designed the same way.

DETAIL 2: EXPANSION BEARING



DETAIL 3: PREFERRED BEARING TYPE CROSSFRAME



Bearing Type Crossframe

1. Provide for preferred edge distances (e.g. $1^3/_4$ for $^7/_8$ diameter bolts).

2. Provide $^3/_4$-in. minimum clearance from edge of fillet weld.

3. Include allowances for notes 1 & 2 in determining stiffener width (normally a $7^1/_2$-in. wide stiffener is required for the connection shown).

4. Keep gusset plates rectangular.

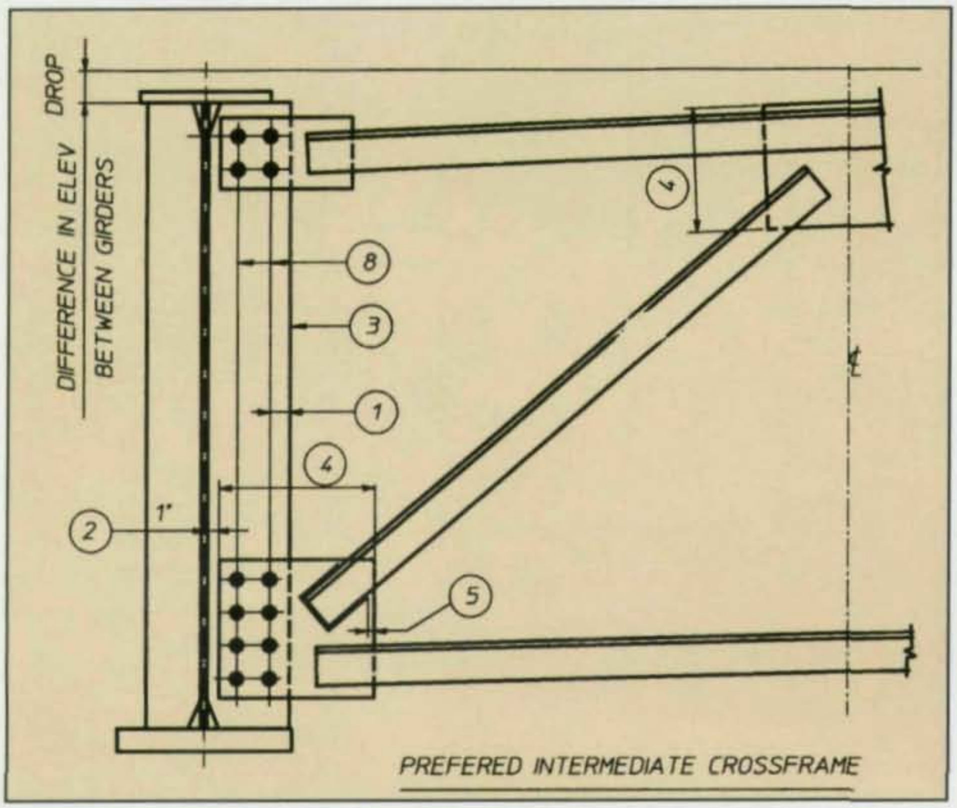
5. Terminate welds $^1/_2$ in. short of edge.

6. Avoid all around welds; omit weld on far side.

7. Avoid clips on stiffener and lower channel to avoid coping.

8. Permit the use of oversized holes for crossframe connections.

DETAIL 4: PREFERRED INTERMEDIATE CROSSFRAME



Intermediate Crossframe

Use K-type crossframes wherever possible because:

- All frames can be easily made in a jig since the connection stiffeners on both girders are the same;
- All welding is done from the near side and therefore assembly does not have to turn over;
- All crossframe top gusset plates, bottom gusset plates and center plates are identical;
- Changes in the geometry of the frame can be easily accommodated by moving one side of the jig for differences in the drop.

Uneconomical Detail

Avoid this detail for the following reasons:

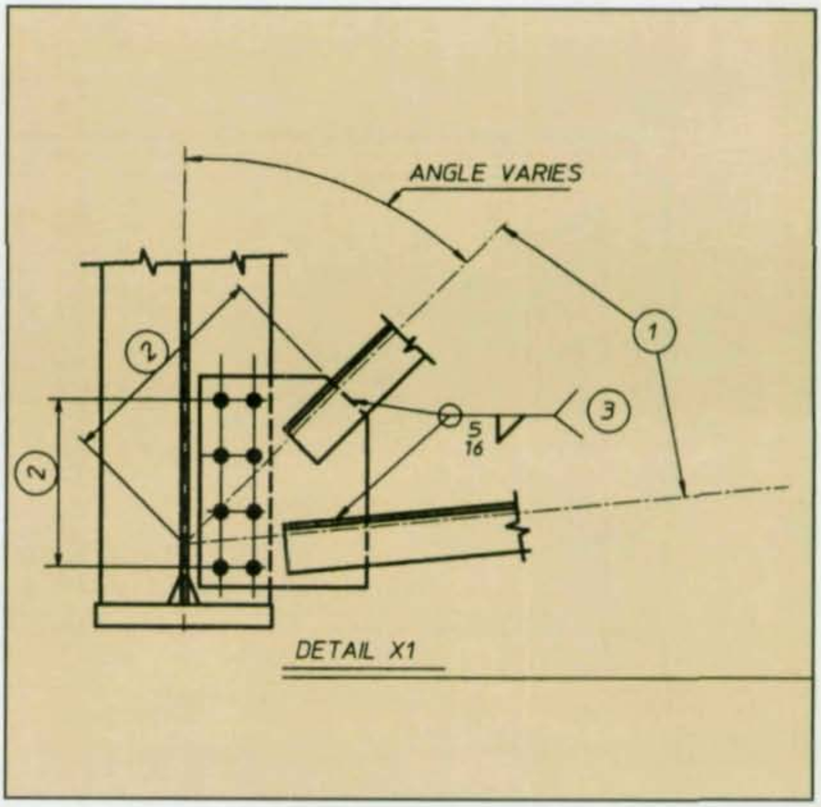
1. The members are positioned with the CGs intersecting a common WP. It would be more economical to design the connection to account for the eccentricity.

2. As the drop changes, the angle varies, which changes the dimensions, requiring a new stiffener and connection plate layout & mark.

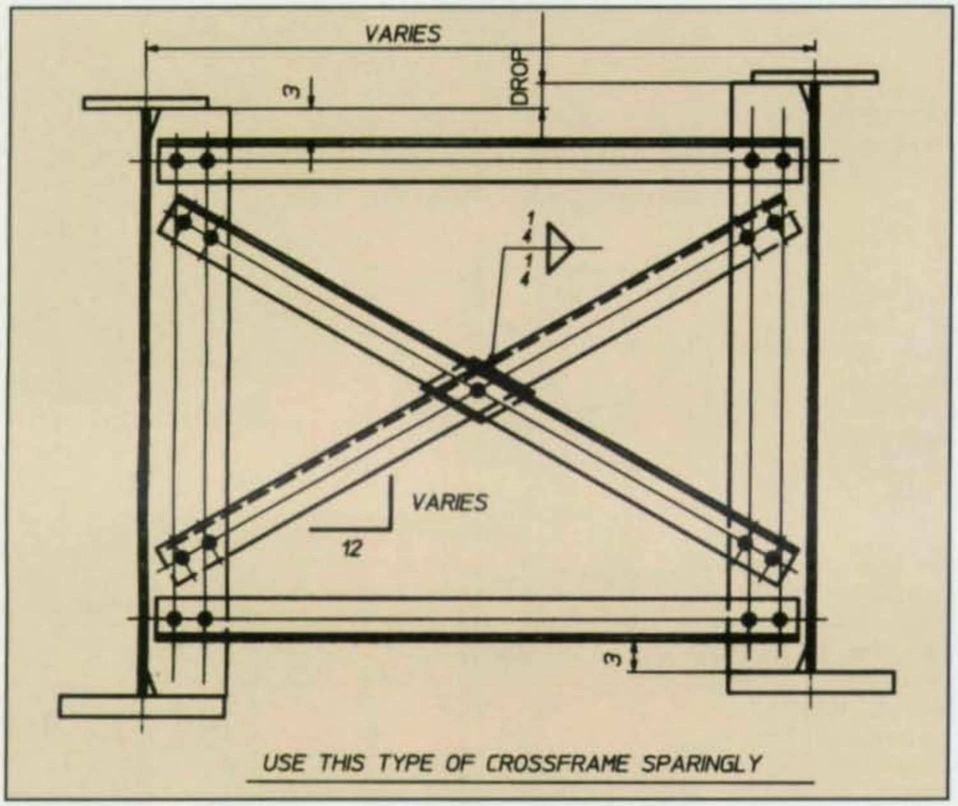
3. All around welding creates undercutting problems. The weld perpendicular to the angle does not increase the capacity of the weld but might decrease strength of the member. Remember, if these angles were bolted, the edges would be tight, though not sealed. Why require sealing when the field connection is bolted tight and is acceptable?

•Duplication will reduce the chance of errors and fabrication costs. This detail requires a different layout for each plate, while the rectangular plate on the previous page does not.

DETAIL 5: UNECONOMICAL DETAIL



Detail 6: Difficult Crossframe



Crossframe

This detail may be economical to fabricate if C/C girders and drops are the same, but it's still difficult to erect.

1. Knocked down crossframes require more shop & field handling. They are difficult to erect due to the increased number of different pieces to track, handle & hoist.

2. Due to varying drops and varying distances between girders, this frame would require a different layout for each stiffener, diagonal, strut and fill plate.

•***The chance of an error on this type of frame is many times greater than a jigged cross frame.*** The frame below and the one on the next page is more economical.

Better Crossframe

This crossframe offers numerous advantages:

•All stiffeners have the same layout and mark.

•No layouts are required since connection plates are rectangular.

•Weight of material is similar since angles are cut back.

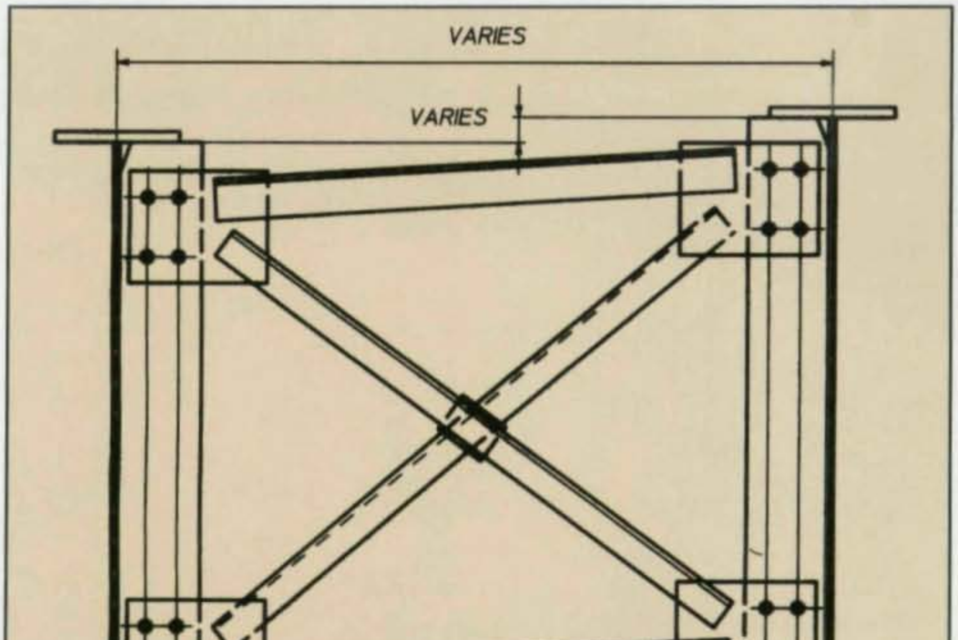
•All angles can be cut without layouts.

•Erection is much faster due to fewer erection pieces.

•Since frames are jigged, the chance of field misfits is minimized.

•All plates can be stack drilled or multiple punched since the hole patterns are identical.

Detail 7: Better Crossframe



Economical Crossframe

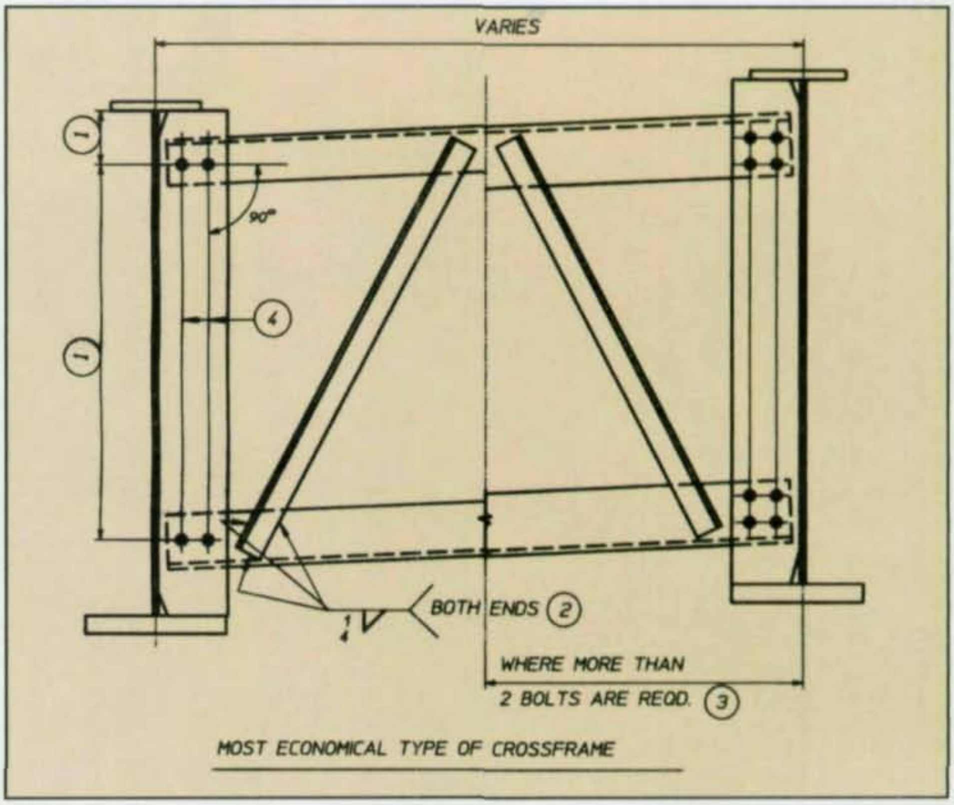
This type of crossframe requires only four components.

For the most economical results:

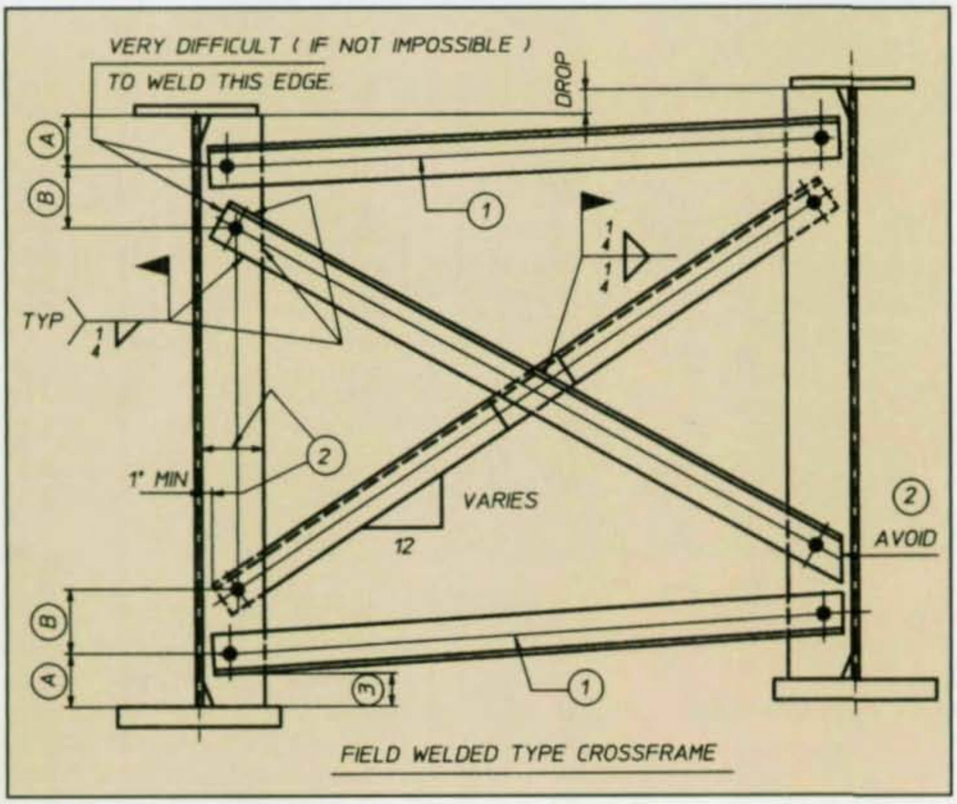
1. Keep these dimensions the same and slope the struts.
2. Keep all welding on one side.
3. Increase the size of the struts as required for load.
4. Use oversize holes to maximize the slope of holes in struts.

•The same method could be used for X type crossframes.

DETAIL 8: BEST CROSSFRAME



DETAIL 9: FIELD WELDED CROSSFRAME

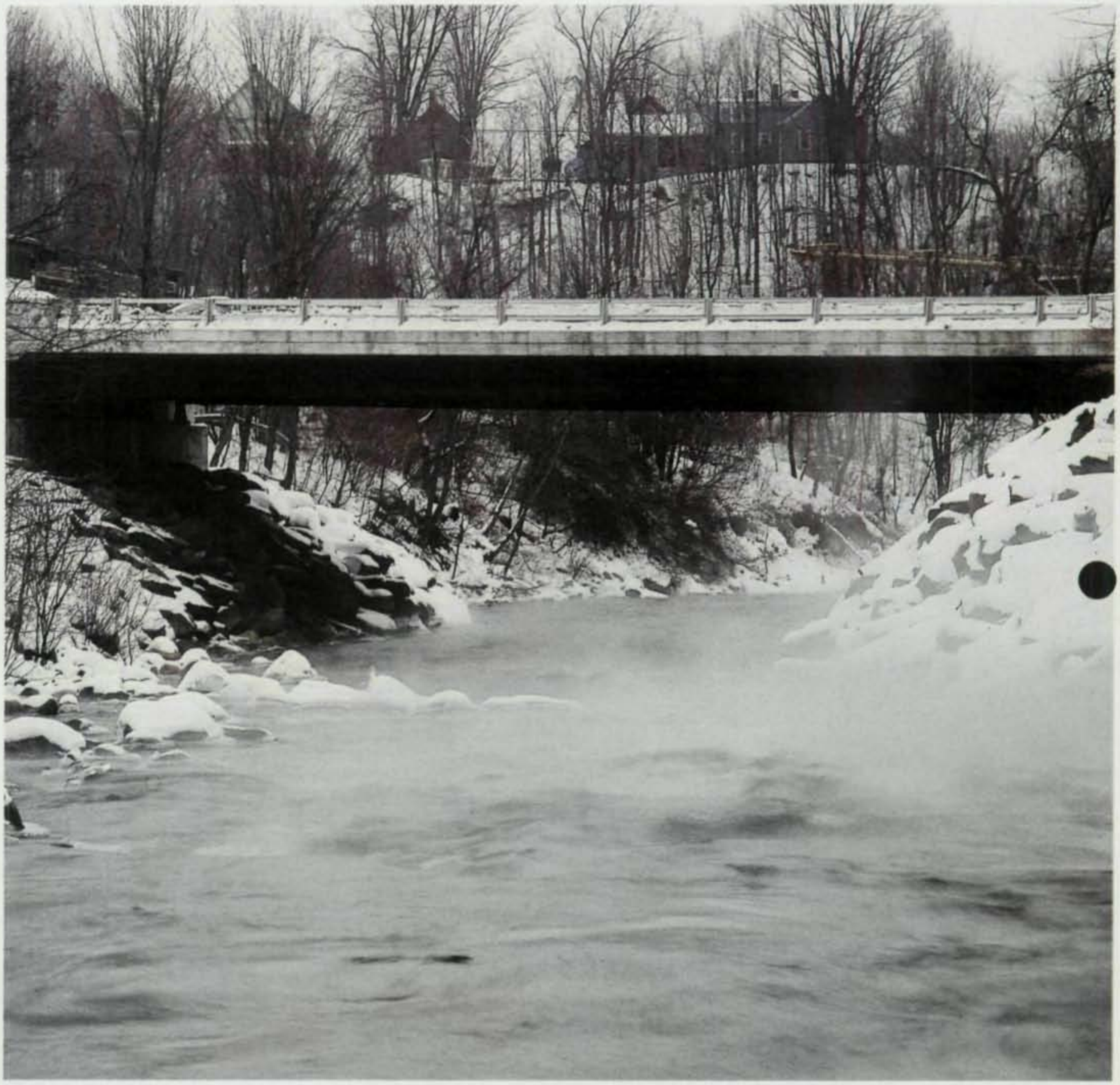


Field Welded Crossframe

In states that prefer field welded crossframes, this method is preferred, providing the following are incorporated:

1. Slope the top and bottom struts and keep dimensions A & B the same on each girder to make the stiffeners identical.
2. Do not cut members on this skew. It is more economical to make the stiffener wider since most fabricators use an anglematic machine that punches and cuts the angle square automatically. A beveled cut requires a burning operation at each end.
3. Make dimension A large enough to provide room between the strut and flange to allow clearance for field welding.

Weather.



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Third, nature. Weathering steel blends with its

Or not.

surroundings, and minimizes disturbance to the natural environment.

In a state with so much weather and so much natural beauty, it's no wonder that Maine already has built 80 weathering steel bridges, and has more on the drawing boards, specifying ASTM A588.

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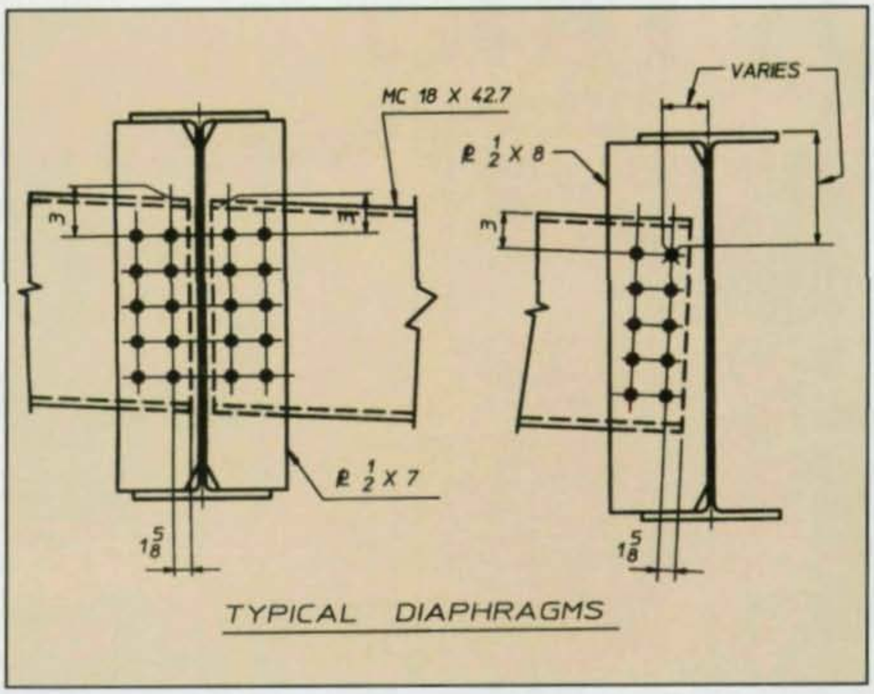
DETAIL 10: TYPICAL DIAPHRAGMS



Typical Diaphragms

Holes for diaphragm connections connections can be located either normal to the stringer web and connection plate or normal to the channel. The method used is based on the fabricators equipment, fabrication methods, and material cost comparisons.

Holes Normal To Plate

Advantages include:

• Connection plates are identical.

• Plate width and weight are minimal.

Disadvantages include:

•Diaphragm ends have a skewed cut.

•Diaphragm layouts change with variable cross slopes.

Holes Normal To Channel

Advantages include:

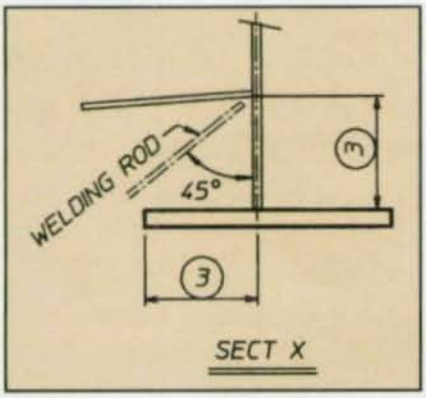
•Diaphragms can be alike.

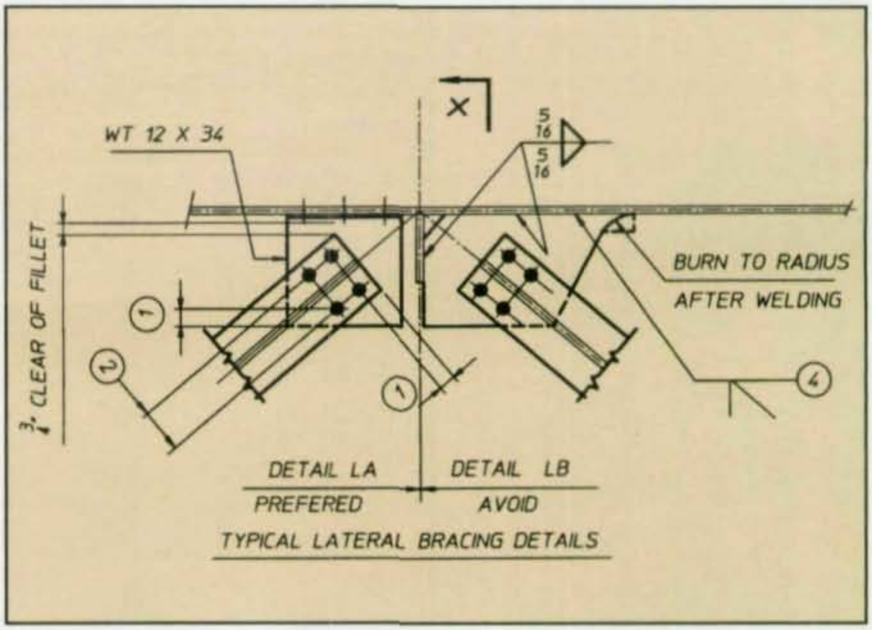
•Diaphragm ends and hole patterns are square to the diaphragm.

Disadvantages include:

•Wider connection plates are required and plate layout changes with each slope.

Detail 11 and 11A: Typical Lateral Bracing Details (bolted)





Typical Lateral Bracing Details (Bolted)

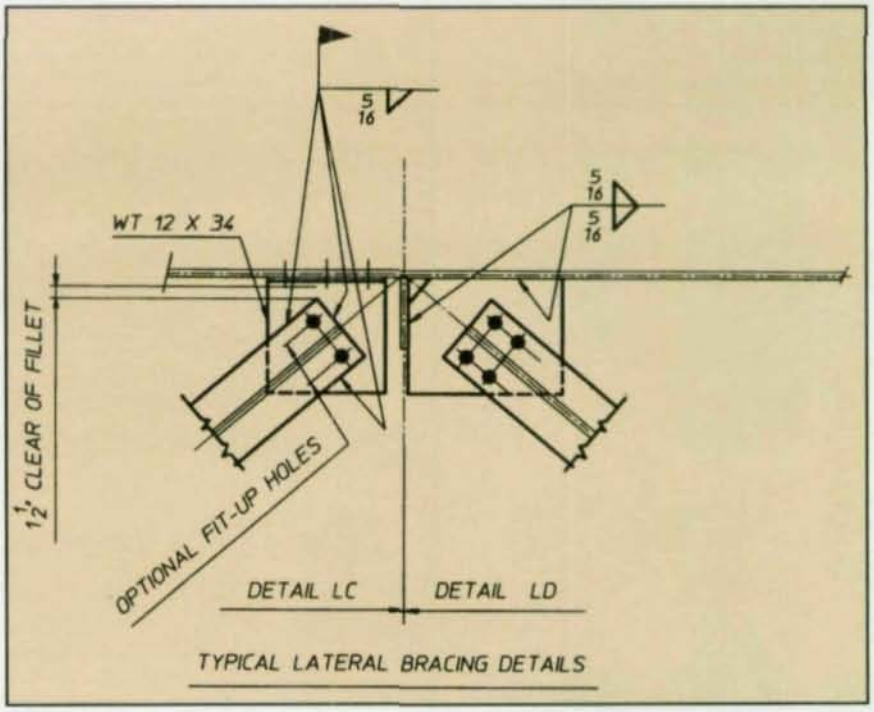
Detail LA is preferred since it can be used in tension zones and meets fatigue criteria. For the most economical design, use standard tee sizes and cut the steel square.

1. Use preferred edge distances for the oversized hole size.
2. Use oversized holes in both plies of the material.

Detail LB has been used where plates are welded in a tension zone. This detail costs more to fabricate due to extra burning and grinding. It is more economical to use Detail LA in lieu of this detail.

3. When lateral gussets are welded, provide clearance to allow fillet welding on both sides.
4. Avoid full penetration welding.

Detail 12: Typical Lateral Bracing Details (Field Welded)

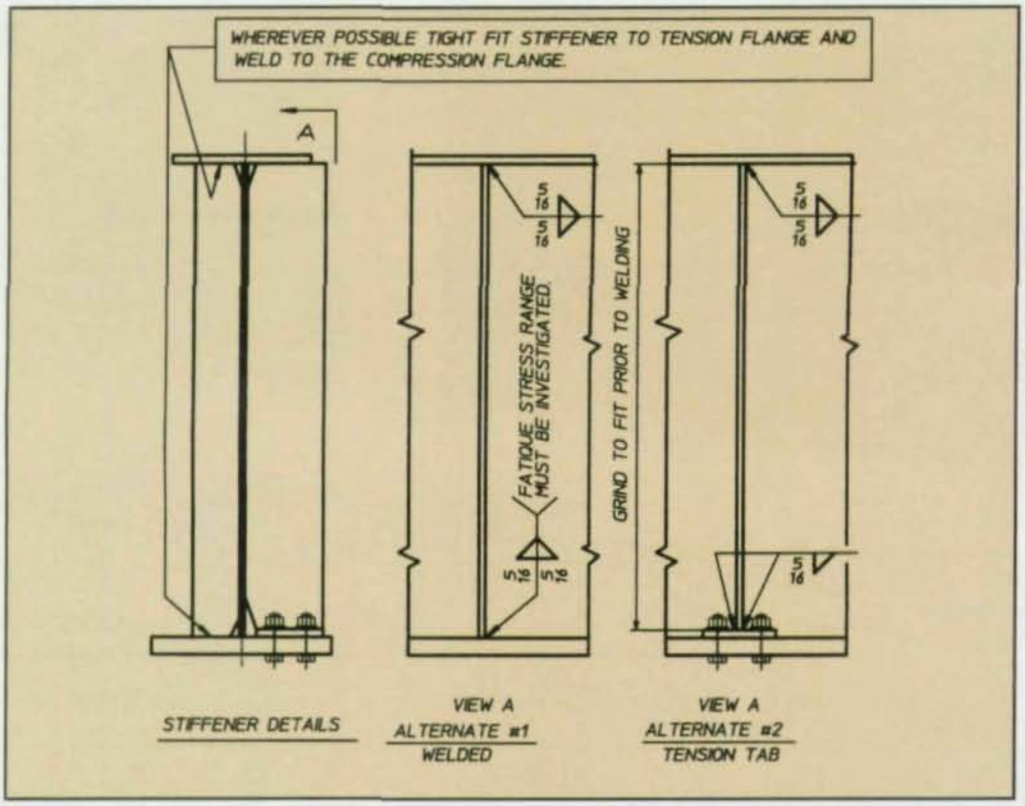


Typical Lateral Bracing Details (Field Welded)

Where field welding is preferred, Detail LC is a practical solution. The gusset plate could be a tee as shown, or a welded gusset plate as shown in detail LD. Fit-up holes could be furnished as an option.

Detail LD is an economical detail to use where the stresses and fatigue criteria allow welding to the web. The welds should always terminate $^1/_2$ in short of the end of the plate.

DETAIL 13: STIFFENER DETAILS



Stiffener Details

Wherever possible, it's more economical to remove the tab plates. Instead, it's more cost effective to weld the stiffener to the tension flange. Between bolts, tab plates and fabrication costs, it adds approximately $75 to $100 per tab plate.

Welded Crossframe

(Note: Design should show alternates for both welded and bolted.)

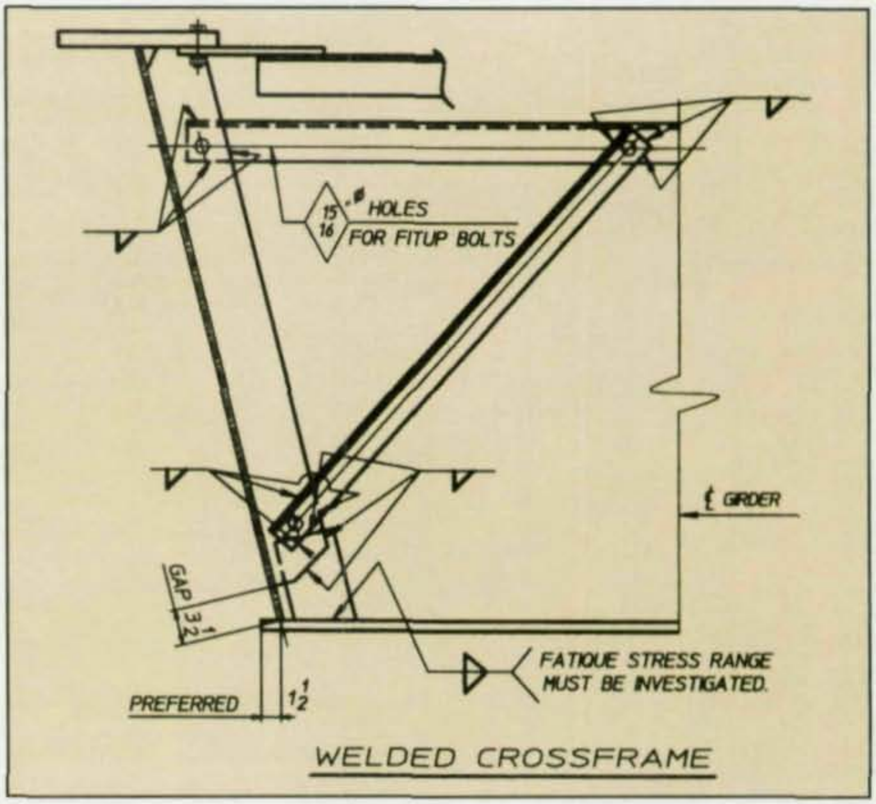
• $1^1/_2$-in. (minimum) bottom flange extension is required for flux support and the welding machine to track on.

• Web, top flange, and stiffeners are usually fabricated as a sub-assembly prior to fitting to the bottom flange.

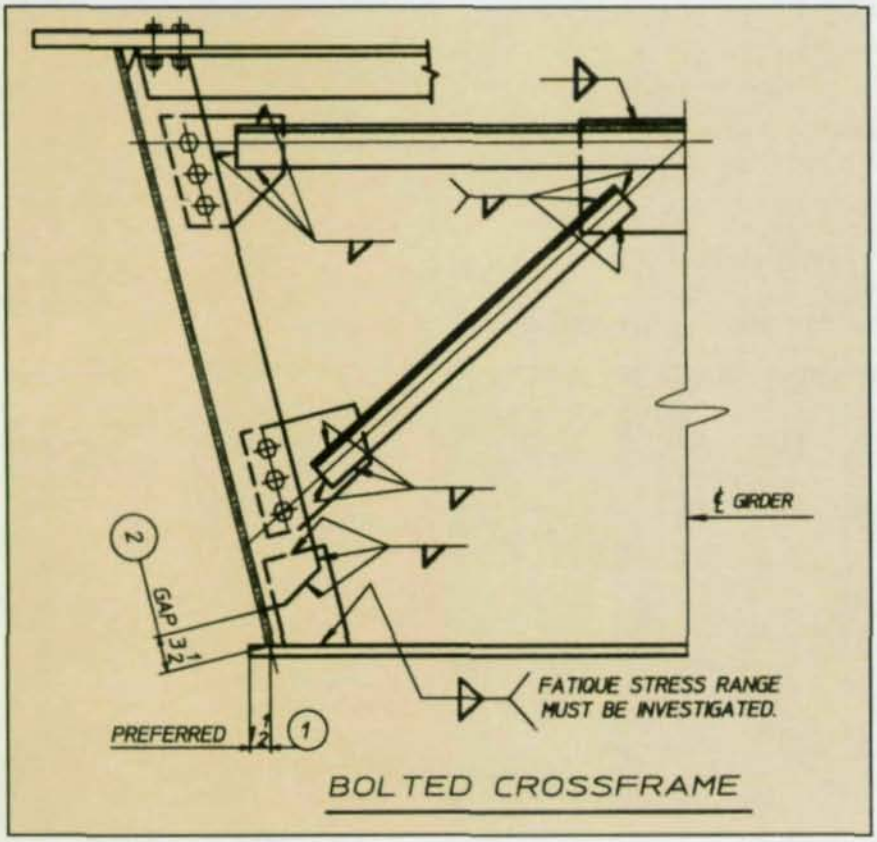
• The crossframe members are then bolted to the web/top flange sub-assembly, which helps shape the final girder assembly. (Alternately, the crossframe is built in a jig as a sub-assembly, fit-up and welded; then it is bolted to the web/top flange sub-assembly.)

• The web/top flange sub-assembly with the crossframes bolted in place is then fitted to the bottom flange plate, which has been blocked to its cambered shape. The web to bottom flange plate welds are then made. The $3^1/_2$-in. gap at the bottom allows the web-to-flange welding to be made without interruptions.

Detail 14: Welded Crossframe (Alternate)



Detail 15: Bolted Crossframe



Bolted Crossframe

(Note: Design should show alternates for both welded and bolted.)

• $1^1/_2$-in. (minimum) bottom flange extension is required for flux support and the welding machine to track on.

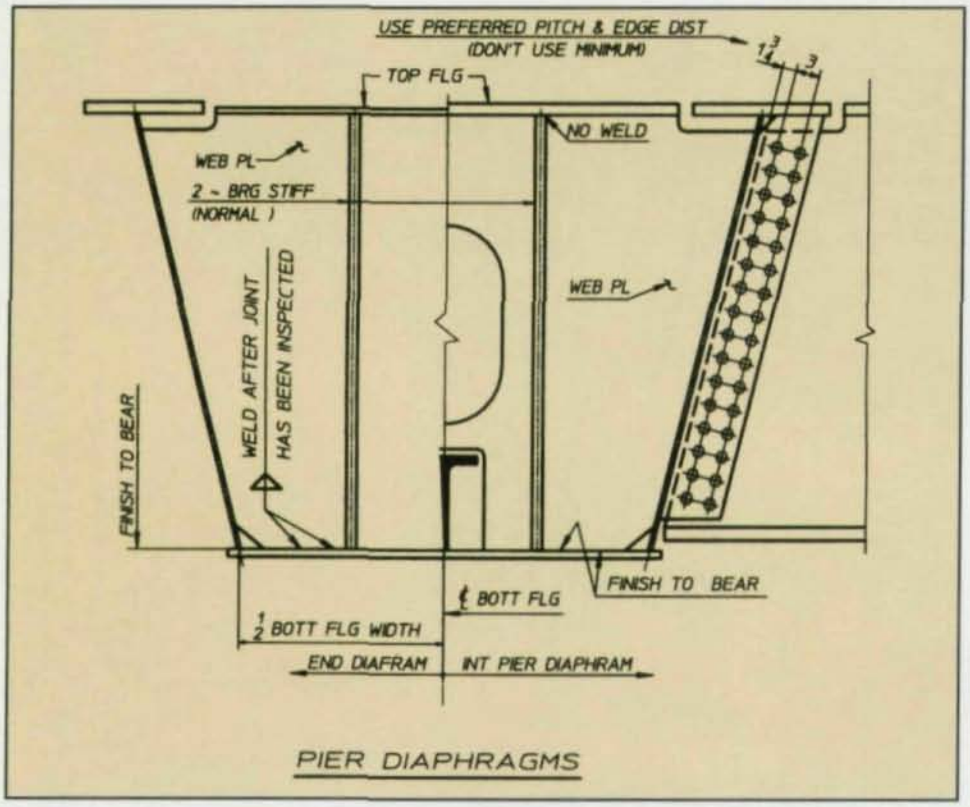
• Web, top flange, and stiffeners are usually fabricated as a sub-assembly prior to fitting to the bottom flange.

•The crossframe is built in a jig as a sub-assembly, fit-up and welded (note that all welding is made from the near side).

•The crossframe sub-assembly is then bolted to the web/top flange sub-assembly, which helps shape the final girder assembly.

• The web/top flange sub-assembly with the crossframes bolted in place is then fitted to the bottom flange plate, which has been blocked to its cambered shape. The web to bottom flange plate welds are then made. The $3^1/_2$-in. gap at the bottom allows the web-to-flange welding to be made without interruptions.

DETAIL 16: PIER DIAPHRAGMS



Pier Diaphragms

• Avoid full penetration welds; finish to bear with fillet weld is preferred.

• Use preferred edge distances and pitch, not minimums.

• The diaphragm is made up as a sub-assembly and then fitted to the bottom flange and web assembly.

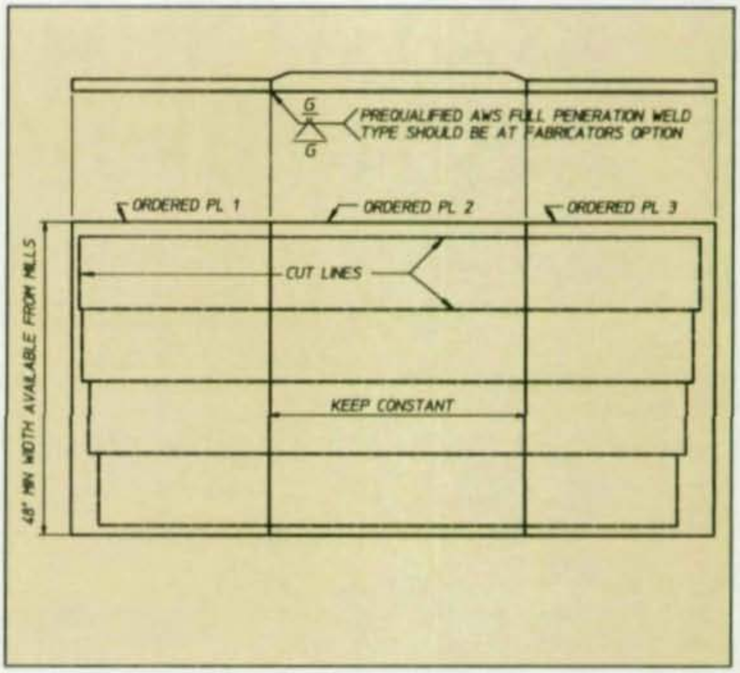
Slabbing & Stripping

Steel mills no longer roll plates in widths less than 48 in. As a result, fabricators are required to combine plates and nest them in order to economize and reduce scrap. This should be considered when the designer selects the sizes of flange plates on adjacent transverse girders.

The ordered plate ends are prepared, as shown. The individual flange plate assemblies are then flame cut to their finished widths by multiple torches. Non-destructive testing is performed prior to the flange plate assemblies being welded to the web plates.

- Avoid transitions in flange width in any one girder (vary thickness instead).
- Keep flange plate thicknesses the same for adjacent girders and keep the lengths on the center flange plates the same.
- Avoid changing flange plate thicknesses. It may be more economical to extend the thicker flange. The cost of a splice may exceed the material costs.
- Slabbing and stripping works for both straight and curved girders.

Detail 17: Slabbing and Stripping



Post-Tensioning Cuts Steel Bridge Costs

Plate girder bridges can be made more economical through the use of post-tensioning

By Leo Spaans, P.E.

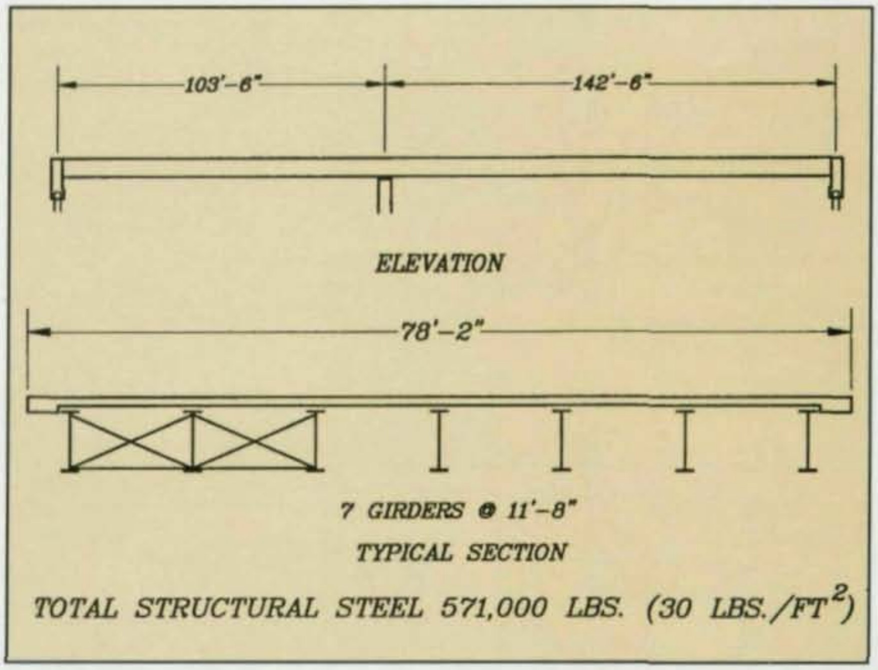
(This paper is adapted from a talk given by the author at the 1993 National Symposium on Steel Bridge Construction.)

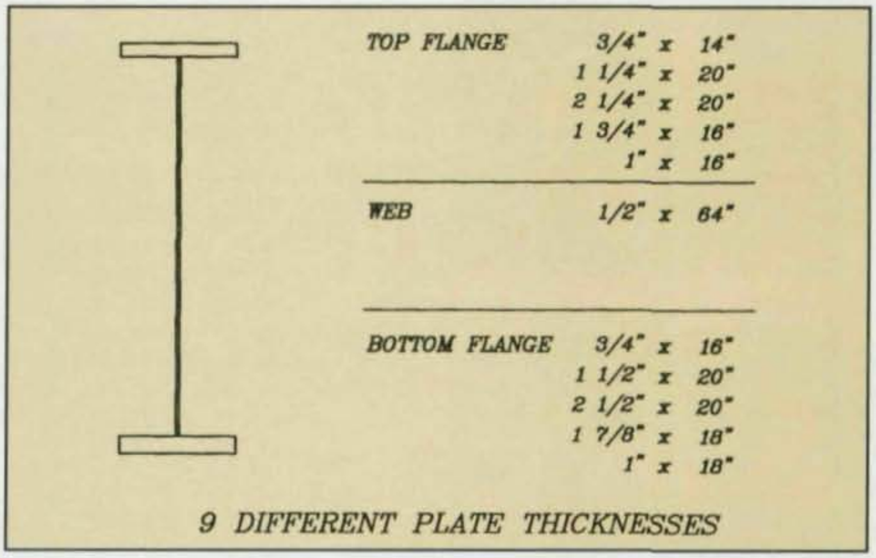
WHILE ONCE DOMINANT FOR BRIDGES WITH 90-FT. TO 250-FT. SPANS, conventional welded plate bridges have slowly been losing ground to prestressed concrete bulb T girder bridges. However, the adaptation of many of the same techniques—especially post-tensioning—that make bulb T girder bridges so successful can result in an even more economical steel structure.

For illustration purposes, an existing welded plate girder design was re-examined. The bridge was originally designed with two spans (103.5 ft. and 142.5 ft.) with welded plate girders spaced 11-ft.-8-in. on center and steel usage totaling 30 lbs. per sq. ft. of deck area. The abutments consisted of integral end bents, which eliminated the use of bearing and expansion joints. In the past, this would have been considered a good, efficient design.

The particular contractor on this job, however, had a preference for concrete and, subsequent to the start of construction, re-engineered the project using a concrete bulb T girder design. Since this design was at least as cost effective as the standard steel design, the state DOT allowed the change.

One of the primary problems with the plate girder bridge was the use of nine different plate





A two-span welded plate girder design proved to be too expensive, in part because it utilized nine different plate thicknesses.

thicknesses, which meant a lot of fabrication labor at the flange width/thickness transitions and for the filler plates at the bolted splice connections. While using different plate thicknesses minimizes material use, it increases labor costs. And, as the steel industry has been pointing out for some time, the wages are going up while steel prices are flat or, in some cases, have dropped.

Subsequently, a post tensioned plate girder design was evaluated to see what savings could be made in both material and fabrication cost. This alternate design had the following criteria:

1. The girder must have a uniform cross-section to eliminate labor intensive splices and flange transitions.

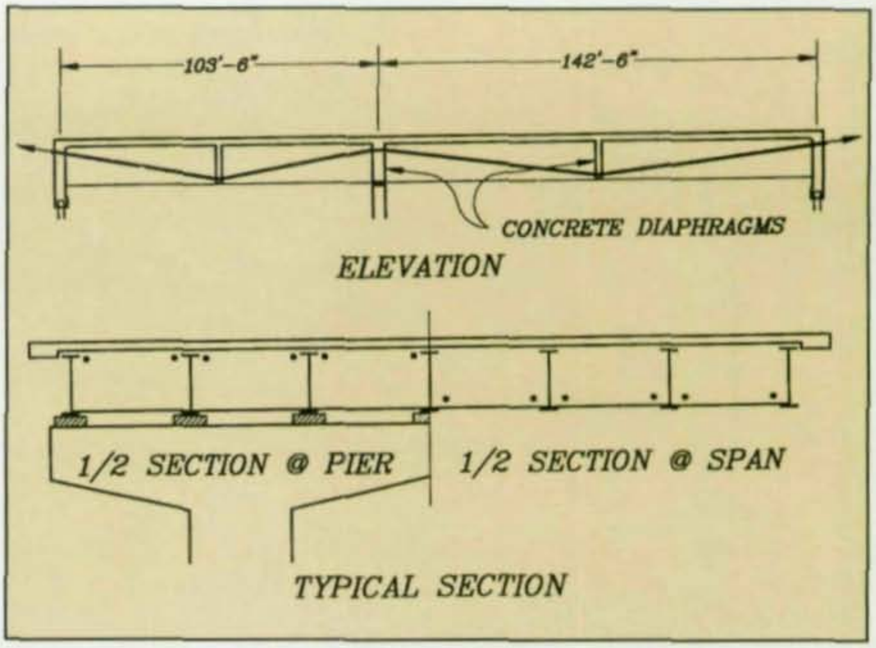
2. It must use external post tensioning tendons protected by polyethylene pipes with grout and concrete diaphragms for deviation points along the length of the structure.

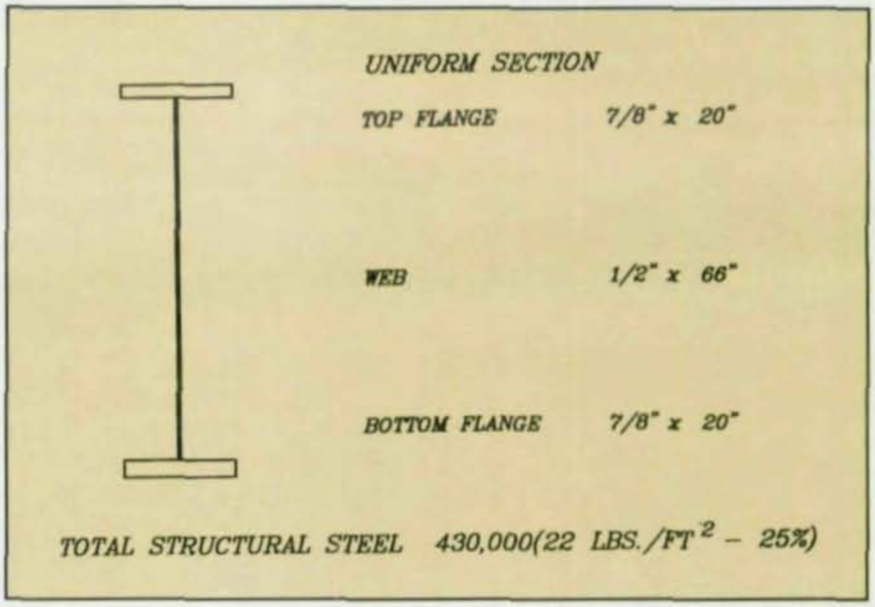
3. The girder section must be strong enough to carry the dead load of the deck slab. This will prevent the need for special temporary girder support and also will allow for safe future deck replacement. Tendons will be stressed after the deck has cured.

4. The design must avoid, at all costs, attaching the post-tensioning anchors to the steel girders. Instead, the tendons must be anchored and supported by the concrete diaphragms. (Earlier post-tensioned steel structures were hampered by designs that attached the post-tensioning anchors to the steel girders. The savings in material efficiency was more than off-set by the cost of the extremely labor-intensive detailing for affixing the post-tensioning anchors.)

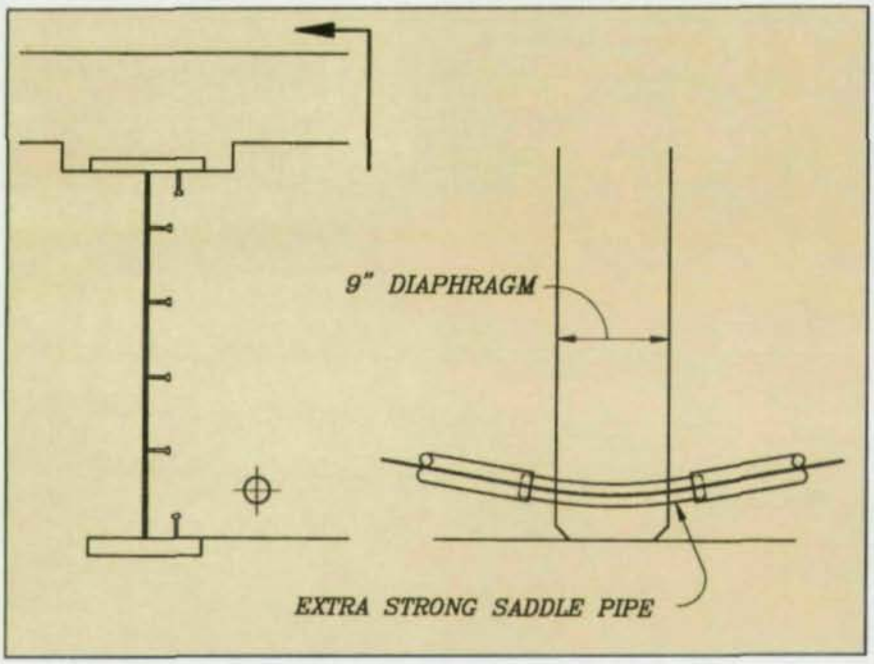
5. Composite design must be used for the entire structure even in negative moment areas. This implies providing compression in the deck over the piers. The advantage is that transverse deck cracking is eliminated due to greater stiffness and less live load deflection. Note, however, that using composite design means the analysis must also include creep and shrinkage, making the analysis slightly more complicated.

The use of the above criteria reduced the amount of structural steel required in the project by 25%, from 30 lbs. to 22 lbs. per sq. ft. of deck area. More importantly, superstructure construction costs were reduced from $342,600 for the original design to $267,250 for the post-tensioned steel design. This was $75,350 less than the concrete prestressed bulb T design, which





Redesigning a two-span bridge using a post-tensioned system reduced the cost of the superstructure from $342,600 to $267,250. The savings was achieved through various methods, including the use of a uniform cross-section, external post-tensioning tendons, composite design and a concrete diaphragm.



To optimize the system, the post tensioning anchors must be supported by the concrete diaphragms; they must not be attached to the steel girders.

cost $318,570.

The concept of post-tensioned steel structures is not new; it has been around since the early 1800s (England's Squire Wipple in 1837) and has been used more recently in Idaho for the Bonners Ferry Bridge by T.Y. Lin.

However, while post-tensioning is becoming widely accepted for rehabilitation projects or for the strengthening of existing structures, it still is not readily accepted for new construction—despite overwhelming evidence that it can provide a very cost efficient steel design.

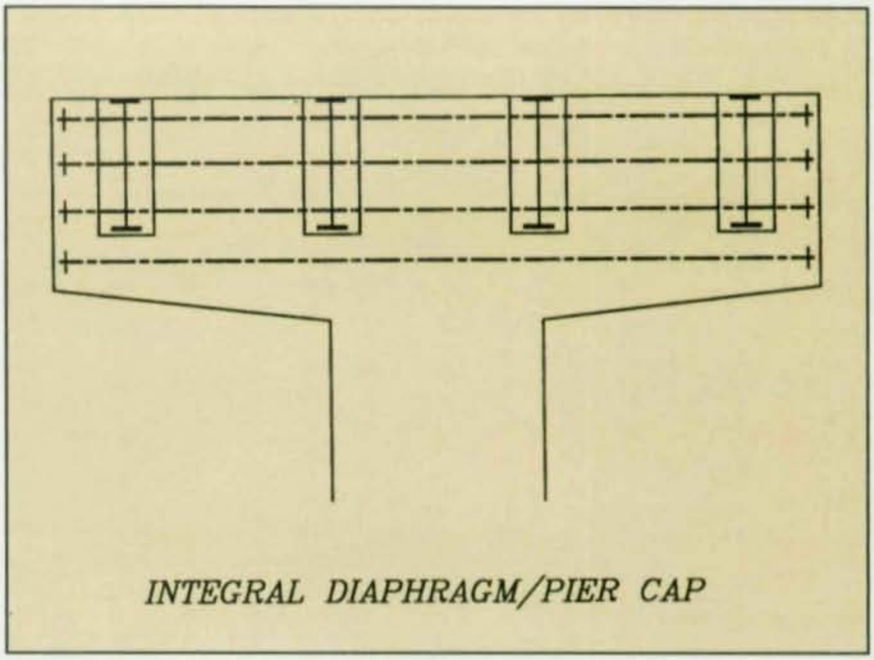
Concrete Diaphragms

Designing a cost effective superstructure is only half the battle. All too often efficiencies in the superstructure are negated by inefficiencies in the substructure. For example, in many alternate bid situations, the substructure for the steel alternate has the same number of piles and the same amount of reinforcing steel as the concrete alternate—despite the fact that the steel alternate is considerably lighter.

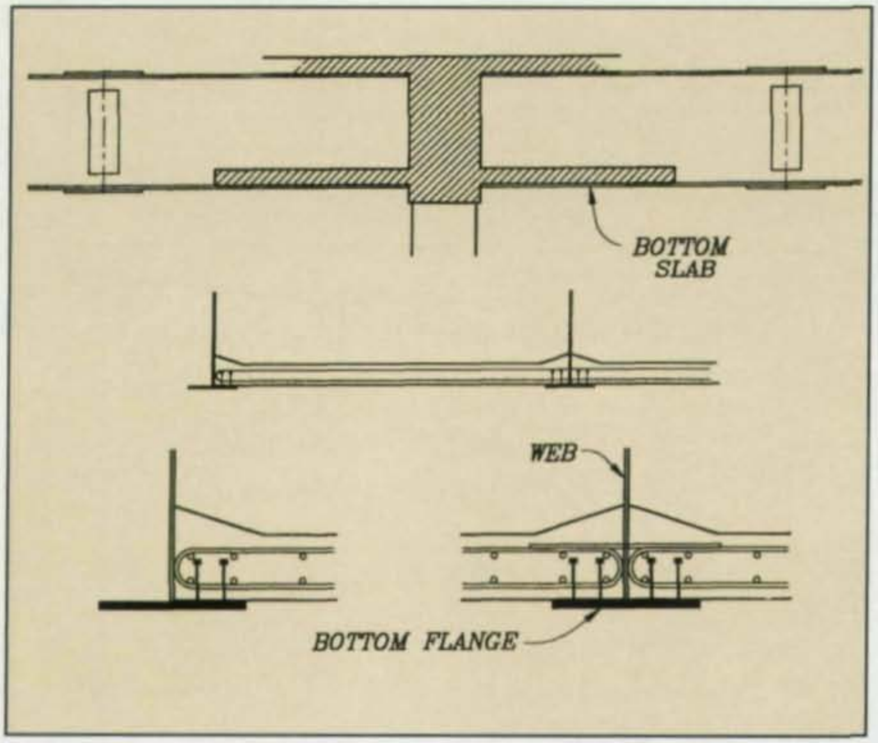
To fully realize the cost advantages of post-tensioned steel bridge design, it is important to use concrete diaphragms. These diaphragms provide the anchor locations and deviation points for the longitudinal post-tensioning and ideally will be used to create an integral connection with the substructure.

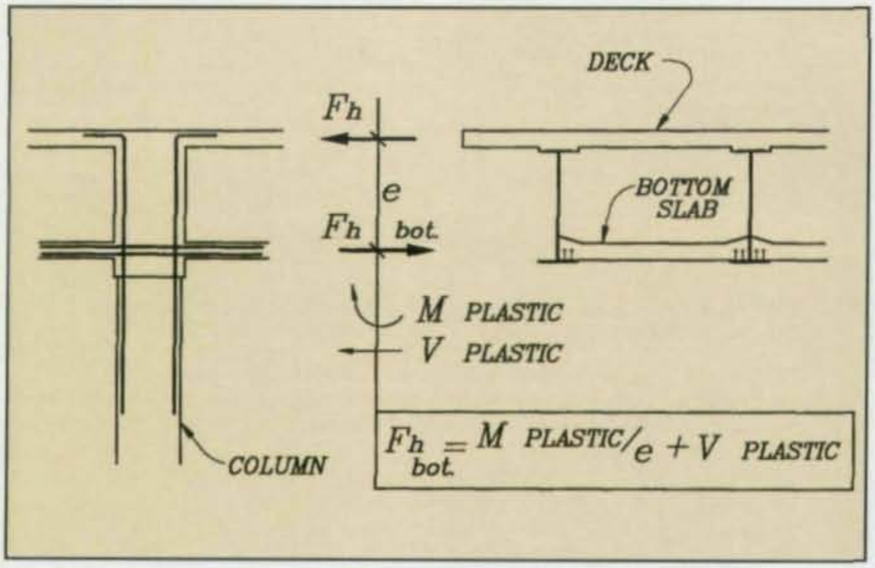
The advantages of this system include:

1. Eliminating bearings and expansion joints at the end bents.
2. Eliminating bearings at the piers.
3. Reducing bearing stiffness requirements and eliminating cross frames.
4. Reducing column and foot-



A fully integral connection elminates the bearings altogether and produces a simpler, more economic design.





Because there is the potential of the column force slipping along the face of the steel plate girder with an integral connection, we recommend casting a partial bottom slab, which, when properly anchored to the bottom flange with shear studs and reinforcing, would prevent any slip and allow the top of the column to fully develop its plastic moment capacity.

ing moments.

5. Developing plastic moment capacity of columns, which is beneficial in seismic zones.

However, even if an integral connection is not utilized, there are still many advantages utilizing concrete diaphragms:

1. Expensive crossframes and web stiffeners are replaced by relatively inexpensive cast-in-place concrete.

2. The concrete diaphragm also provides support for jacks in case of future bearing replacement.

3. If larger elastomeric bearings are used instead of pot bearings, a concrete pad provides ample support.

Note, though, that transversely post-tensioning the diaphragm and steel plate girder and making fully integral connections eliminates the need for bearings altogether.

The primary drawback to the use of concrete diaphragms is that false work is required to temporarily support the steel girders until the diaphragm is poured and post-tensioned. However, on a recent project in Indiana, the steel alternate using this method was successfully chosen over the concrete bulb T alternate.

Also, it is possible to create a design that eliminates the need for falsework. One method of doing so is to provide the lower portion of the pier cap/diaphragm (with slotted holes) with sufficient strength to support the dead load of the steel girders. After erection of the steel girders, the slotted holes are filled with concrete and transverse post-tensioning is applied. This scheme has been successfully used for concrete girders so there is every reason to expect it to be equally successful with steel designs.

Concrete integral diaphragms for steel structures have been used in the past, but primarily in cases with special conditions. Based on our experience, concrete diaphragms offer an economical alternative to conventional crossframing and bearing stiffeners.

Seismic Design

In seismic zones, where the lateral load can be as high as the total vertical load, the use of a steel superstructure, if properly designed and concrete diaphragms are used, can result in significant cost savings for the substructure.

If the masses of the superstructures of three different bridge types (box girder, bulb T girder and plate girder) are converted to an equivalent slab thickness, it becomes obvious that the plate girder shows a substantial weight reduction.

An examination of the substructure requirements for a conventional steel design (utilizing a pinned connection at the top of the column) compared to those of a box girder structure shows loads are about 48% less for the steel design. However, there are no substantial savings since the column rebar and footings are about the same, though there are fewer piles. This addition is primarily the result of the pinned connection at the top of the column. The use of a concrete diaphragm, which would create an integral connection, would change the picture dramatically: The rebar, footing size and pile requirements would be significantly reduced and would be parallel with the load reductions.

Because there is the potential of the column force slipping along the face of the steel plate girder with an integral connection, we recommend casting a partial bottom slab, which, when properly anchored to the bottom flange with shear studs and reinforcing, would prevent any slip and allow the top of the column to fully develop its plastic moment capacity.

Leo Spaans, P.E., is a partner with Janssen & Spaans Engineering in Indianapolis and is best known for his work designing concrete bridges.

STEEL GIRDER DESIGN: CAN IT BE SIMPLIFIED?

By Atorod Azizinamini, Steve Kathol, and Michael W. Beacham, P.E.

Pictured above is the completed bridge in the laboratory.

FIELD TESTING HAS SHOWN THAT AASHTO'S MANUAL FOR BRIDGE INSPECTION AND EVALUATION is, in many cases, conservative and does not accurately reflect the large reserve capacity of steel girder bridges. In addition, the Manual is not intended to address the role of such elements as diaphragms on the performance of steel bridges. However, such knowledge could be extremely useful in both retrofitting existing bridges and modifying design provisions of new bridges.

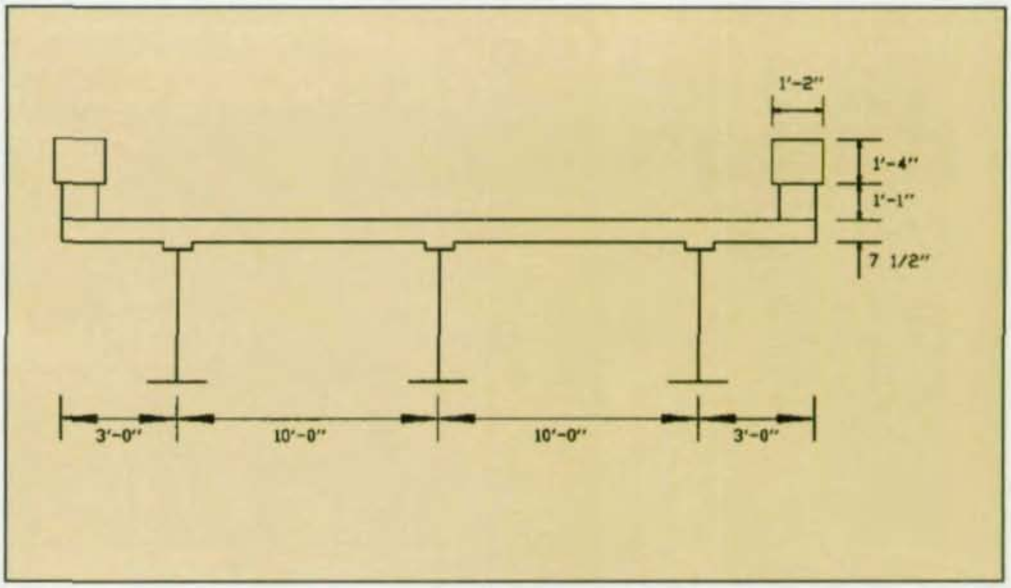
As part of an investigation to more closely evaluate and assess the contribution of different structural elements on the performance of steel girder bridges, a full scale steel plate girder bridge was constructed and several tests—including ultimate load tests and punching shear tests—were conducted at the University of Nebraska-Lincoln. Some of the unique characteristics of the bridge included using minimal reinforcement in the deck in accordance with empirical design provisions of AASHTO's upcoming LRFD manual in combination with relatively large girder spacing. In addition, the bridge was built so that the spacing and type of diaphragm could be varied.

The test bridge spans 70 ft. and is 26-ft. wide. The superstructure consists of three welded plate girders built compositely with a $7^1/_2$-in.-thick reinforced concrete deck. The girders, each 54-in. deep, are spaced 10-ft. on center and the reinforced concrete deck has a 3-ft. overhang. The concrete barrier structure is an open concrete bridge rail, with 11x11 in. posts spaced 8-ft. on center.

The construction sequence was identical to field practice, with assembly of the bridge components starting in July 1993 and the concrete deck being poured in September. The construction of rails and posts were completed on Sept. 21. Following the casting of the concrete deck, creep and shrinkage behavior of the bridge was monitored for a period of 110 days and the data recorded.

Following the creep and shrinkage tests, a total of 52 live load tests were conducted using 12 hydraulic rams capable of applying 400,000 pounds each. In each of the bridge's two lanes, six hydraulic rams were placed to represent the footprint of a single AASHTO HS20 truck, with the rams positioned to simulate either one truck in either the right- or left-hand lane or both lanes or a truck straddling the centerline.

One of the objectives of the research was to investigate the effect of different diaphragms and their spacing on performance of the steel girder bridge. Diaphragms are needed during construction; however, their contribution after construction is a point of debate. Two different types of diaphragms were used: K type, consisting of top and bottom horizontal T sections and two angles forming the diagonal members; and X type, consisting of two diagonal angles only. Testing was conducted at a load level corresponding to a truck weighing 2.5 times the AASHTO HS20 truck loads (180,000 lbs.). The spacing of the diaphragms was varied; the K diaphragms were spaced 22.4 ft. or 11.2 ft. while the X diaphragms were spaced 22.4 ft. During these tests, the response of the bridge was in the elastic region only, i.e., no permanent deformation



Shown at right is the cross section of the bridge. It was constructed full scale in the lab and spans 70 ft. with a 26-ft. width.

was observed after complete unloading.

Results of this phase of the investigation indicated that behavior of the bridge with X type diaphragms was almost identical to the case of K diaphragms. In some of the tests, all the diaphragms were removed and the behavior of the bridge was only slightly affected. The level of stresses developed at diaphragms was small. The same results also were obtained from a series of detailed finite element analyses carried out on this bridge and on bridges with different configurations.

Following the elastic load tests, the bridge was tested to collapse. During the ultimate load test, all diaphragms were removed except those at the supports. The bridge was designed for HS20 loads; however, it showed an ultimate capacity equivalent to approximately eight times AASHTO's HS20 truck loads in each lane. The behavior of the bride was linear, even at load levels corresponding to ultimate capacity as calculated in accordance with AASHTO's LRFD criteria. The bridge exhibited non-linear behavior only after the applied load reached an equivalent of approximately six HS20 trucks in each lane.

Following the ultimate load tests, a series of punching shear tests was performed. Punching shear capacity of the concrete deck varied between 122,000 and 156,000 lbs.

As part of the investigation, a user-friendly preprocessor that interfaces with the SAP90 finite element program from Computers and Structures, Inc., was developed. The preprocessor requires minimal information and in turn generates necessary data for complete three dimensional analysis of simple and continuous steel bridge structures. The package allows inclusion of the effect of the barrier structure if so desired. The results of the analyses match up well with the test data, both with respect to deflection and stresses. The input consists of very simple information such as span lengths, number of girders and material properties.

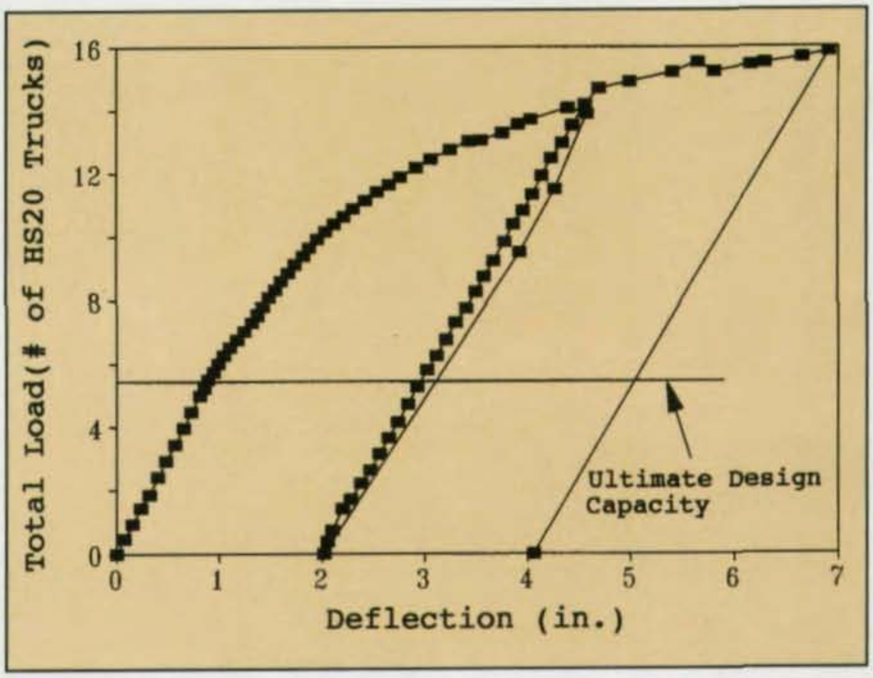
CONCLUSIONS

Although the project is still ongoing, results of the analytical and experimental investigations suggest the following conclusions:

1. For steel bridges with small skew, although diaphragms are needed during construction, their presence has little influence on the behavior of steel bridges after construction. Results indicate that after construction, diaphragms not only are unnecessary, but are to a degree harmful as they try to prevent the small tendency of the girders to separate during elastic ranges and as a consequence transfer restraining forces to beam webs, which have been shown to cause cracking.

After construction, the stiffness of the slab is sufficient to distribute the live load to adjacent girders. It could be argued that diaphragms are needed to provide redundancy in the bridge, i.e., diaphragms could be used to provide alternate load paths in the event of failure of such elements as the concrete deck. In this scenario, however, it is unlikely that diaphragms could provide such a function and bridge failure would be imminent anyway. This is especially important given the fact that most problems in steel bridges are caused by the presence of diaphragms.

Results of this research indicate that if it is desired to leave diaphragms in place, utilizing simpler forms of diaphragms such as the X type provides as good behavior as the more expensive K or other types. Another application of this conclusion could be in the retrofitting of old steel girder bridges. In cases where cracking in elements connecting diaphragms to the girder or girder web are observed, a viable solution could be removal of the diaphragms altogether and thereby avoiding



Shown is the load response of the bridge during ultimate testing.

costly repairs.

2. Results indicate that ultimate capacity of steel bridges with the slab designed based on empirical rule, even with wide girder spacing and no intermediate diaphragms, is several times more than that predicted by the AASHTO code. The use of empirical rule in the design of concrete slabs results in much smaller amounts of reinforcement in the deck. In addition to reducing construction time, this reduced steel reinforcement would also be beneficial in reducing corrosion.

3. A user-friendly three-dimensional analysis package has been developed that closely matches the test results. The input is simple, and the results are easily interpretable using the post-processor of the SAP90 computer program. Use of this type of analysis will eliminate the need for calculating distribution factor while very accurately reflecting behavior of the bridge. The analysis package could be used in cases where more a accurate behavior of the bridge is desired, such as during the retrofitting of a bridge. For more information on this processor, circle no. ?? on the reader service card near the back of this magazine.

Atorod Azizinamini is an assistant professor of civil engineering at the University of Nebraska-Lincoln. Steve Kathol is a former graduate student at the university and is currently a structural engineer with Schemmer Associates in Omaha. Michael W. Beacham, P.E., is research and development engineer with the Bridge Division of the Nebraska Department of Roads.

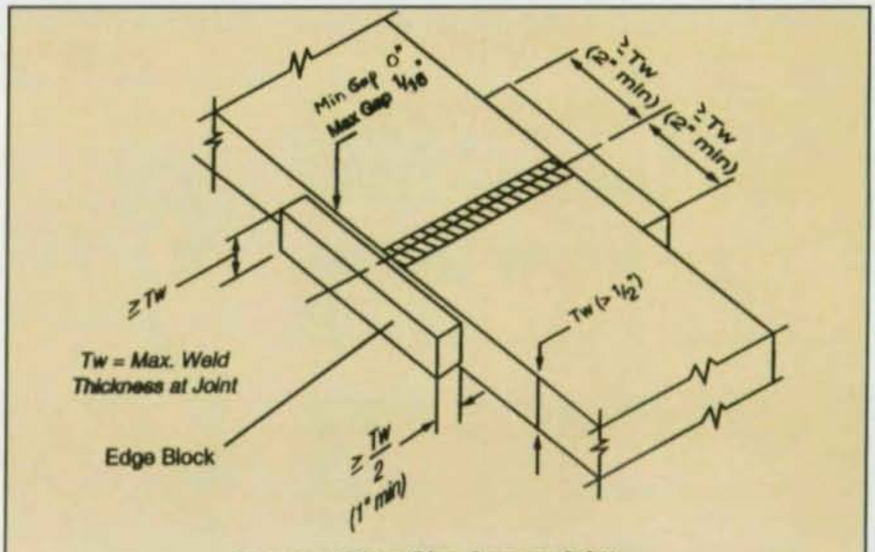
This project is sponsored by the Nebraska Department of Roads and valuable input was provided by Gale Barnhill, structural engineer, Lyman Freemon, bridge engineer, and Mo Jamshidi, assistant bridge engineer. During the course of the investigation, technical input was provided by Robert Nickerson, and Jim Luedke, Yerapalli Shekar and Bruce Keeler assisted in conducting the experimental and analytical studies. Additional support was provided by the Center for Infrastructure Research at the University of Nebraska-Lincoln.

EDGE BLOCKS FOR RADIOGRAPHIC TESTING

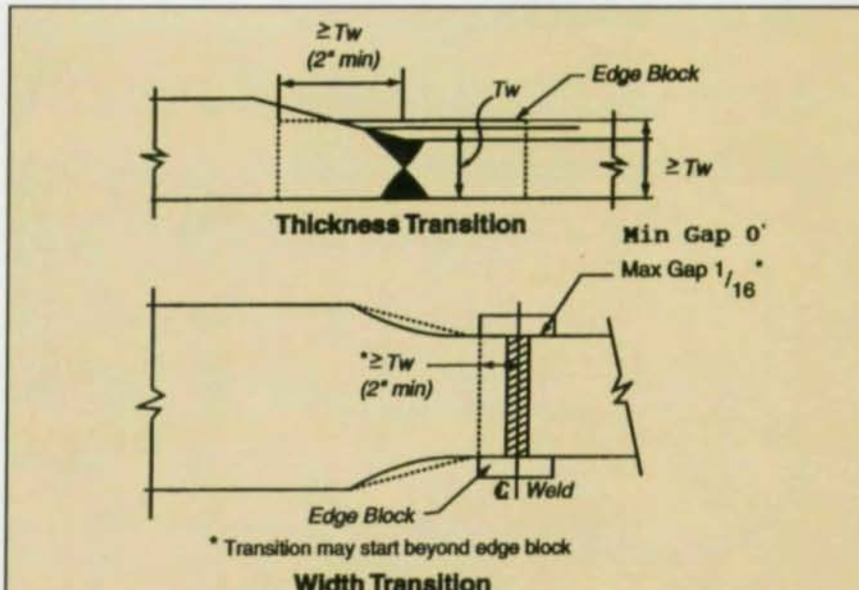
By Krishna K. Verma, Welding Engineer, FHWA

THE ANSI/AASHTO/AWS BRIDGE WELDING CODE COMMITTEE RECENTLY APPROVED a code revision that requires the use of edge blocks for radiographic testing of steel plates with a thickness greater than 12 mm (1/2 in.). This revision is not likely to appear in the Bridge Welding Code until 1996. A similar provision has already been adapted by the "ANSI/AWS D1.1 Code-Steel."

An Edge Block is simply a steel plate placed snugly against the end of the weld or plate edge to be radiographed. Previously, conventional radiographic testing did not mandate edge blocks. However, when edge blocks are used, there is an improvement in radiographic inspectability of the edge of the plate being tested (RT) adjacent to the block. The chief of the Federal Highway's Bridge Division, Stanley Gordon, has recently issued via a memorandum dated April 28, 1994, an advisory to all FHWA regional offices to encourage State DOTs to use edge blocks when performing radiographic testing of butt welds or when a plate edge is to be radiographed.



Constant Plate Size Across Joint



Thickness Transition

Width Transition

SUGGESTED SPECIFICATIONS

As approved by the AASHTO/AWS Joint Committee on Bridge Welding Code 1.5:

"Edge Block. Edge blocks shall be used when radiographing butt welds greater than 1/2 in. (12 mm) thickness. The edge blocks shall have a length sufficient to extend beyond each side of the weld centerline for a minimum distance equal to the weld thickness, but no less than 2 in. (51 mm), and shall have a thickness equal to or greater than the thickness of the weld. The minimum width of the edge blocks shall be equal to half the weld thickness, but not less than 1 in. (25 mm). The edge blocks shall be centered on the weld with a snug fit against the plate being radiographed, allowing no more than 1/16 in. (1.6 mm) gap. Edge blocks shall be made of radiographically clean steel and the surface shall have finish of ANSI 125 μin. (3 μm) or smoother."

(For more informatin, see "Application of Run-On/Run-Off Tabs and Edge Blocks for Steel Bridges," by Krishna K. Verma, presented at the NDT Conference in Atlantic City in February.)

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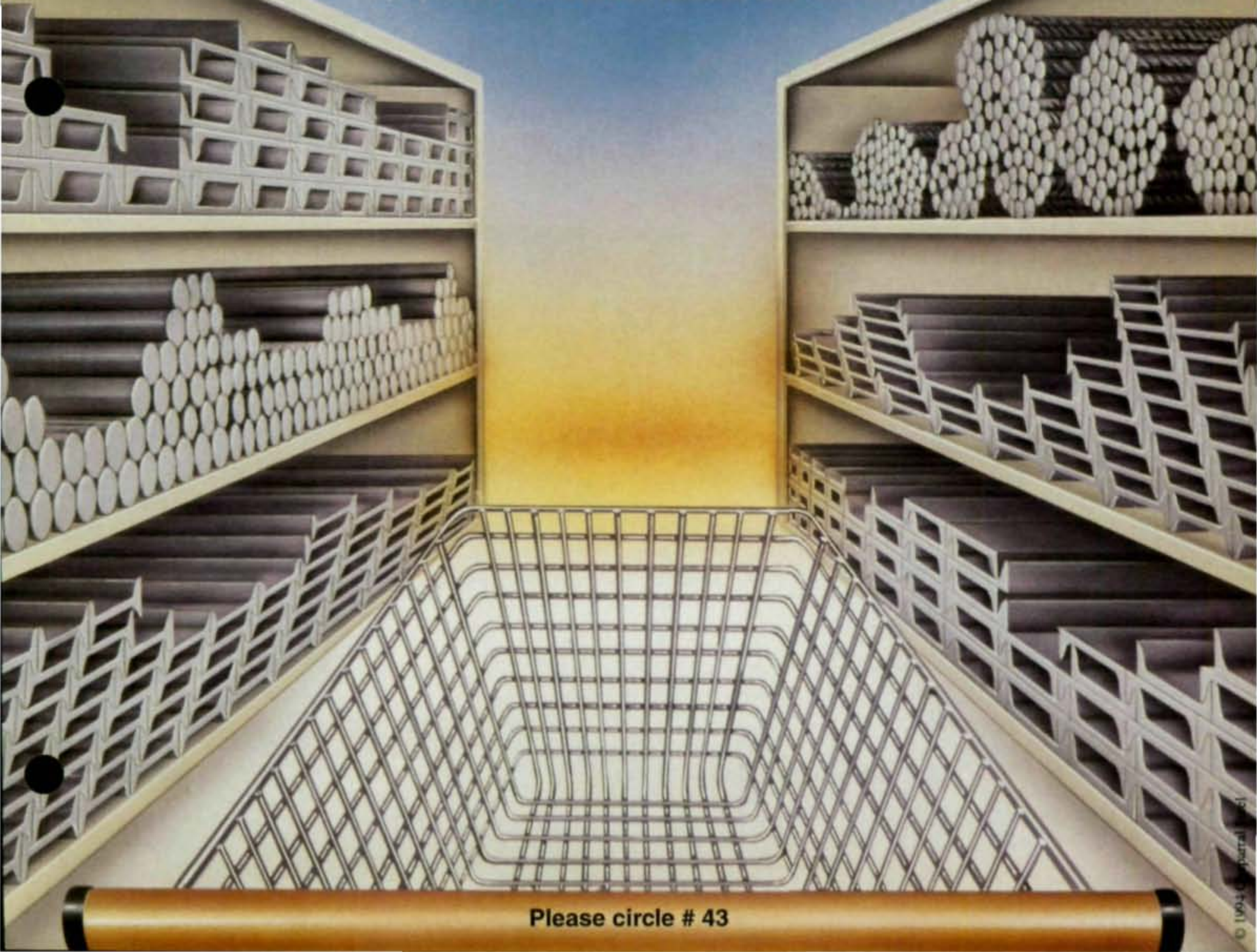

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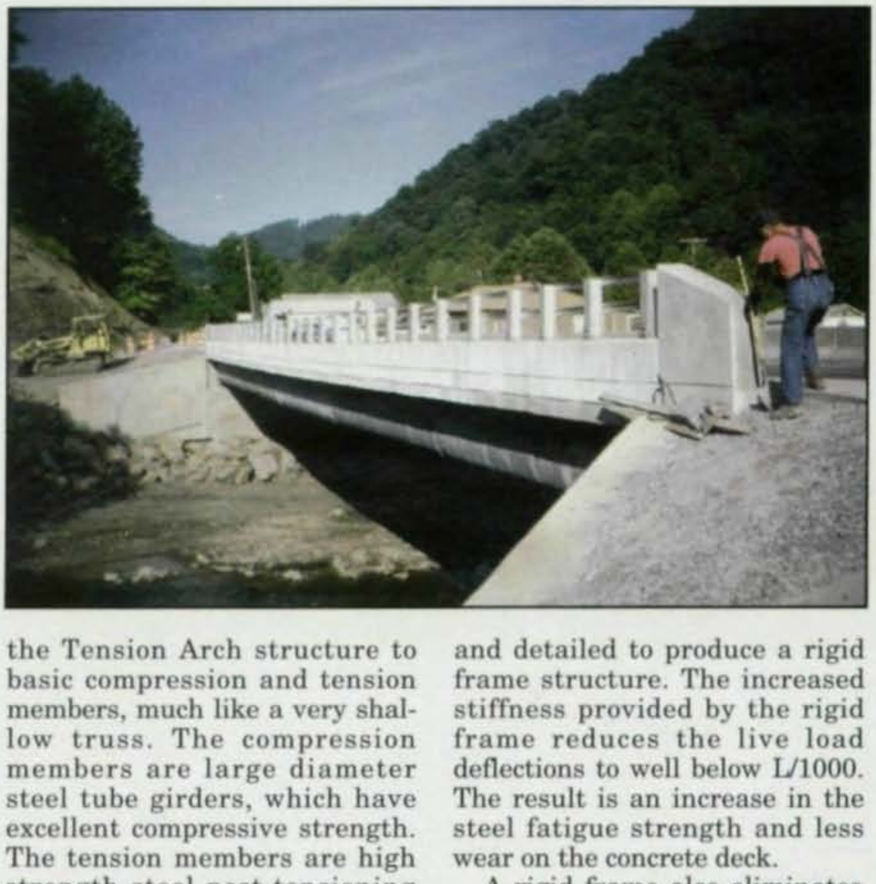
EXPLORING NEW BRIDGE DESIGNS

A new post-tensioned, segmental steel bridge design can reduce costs on replacement bridges

By David L. Weaver, P.E. and Samuel G. Bonasso, P.E.



WHEN A NEW BRIDGE OPENED EARLIER THIS YEAR IN THE TOWN OF WAR, WV, it was greeted with little fanfare. To its many users, it's indistinguishable from any other bridge over WV Route 16, deep in the Mid-Atlantic coal mining region.

But to bridge aficionados, the new bridge has the distinction of being only the second bridge constructed using a new Tension Arch structural system, a post-tensioned, segmental rigid frame bridge developed and patented by Samuel G. Bonasso.

DESIGN ADVANTAGES

With many bridges in need of replacement, the Tension Arch provides an alternate fabricated steel solution that is easy to construct and has minimal maintenance requirements.

The Tension Arch was conceived to be a manufactured product rather than a customized design. It uses common construction materials and techniques in an innovative configuration. With details that are adaptable to most any site, Tension Arch steel girder components could be mass produced similarly to precast concrete girders.

The Tension Arch system mimics the behavior of an arch while in a flat form. This is accomplished by post-tensioning the steel girders using a draped tendon profile.

Post-tensioning helps reduce the Tension Arch structure to basic compression and tension members, much like a very shallow truss. The compression members are large diameter steel tube girders, which have excellent compressive strength. The tension members are high strength steel post-tensioning strands.

Inducing compression in the tube girders increases fatigue strength and simplifies connection design.

The Tension Arch uses fabricated steel segments that are twenty to thirty feet long. Using small segments simplifies fabrication, shipping, and erection requirements. It also allows the steel to be hot dip galvanized in lieu of painting, using kettles that are typically available in the United States.

The steel superstructure and concrete abutments are designed and detailed to produce a rigid frame structure. The increased stiffness provided by the rigid frame reduces the live load deflections to well below L/1000. The result is an increase in the steel fatigue strength and less wear on the concrete deck.

A rigid frame also eliminates the need for expansion joints. In recent years, many bridge designers and owners have realized that eliminating expansion joints typically results in lower maintenance costs.

THE WAR BRIDGE

The bridge in War is the second Tension Arch bridge. The first was constructed in East Logansport, WV, in 1989 and has a clear span of 95 ft. The East Logansport bridge proved the system to be structurally sound, economically viable, easy and quick to construct, and to

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c. Staff Engineer/ Staff Architect/ Construction Manager
d. Other

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b. Civil engineer
c. Structural steel fabricator
d. Other fabricator
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have predictable stress and deflection characteristics.

With the success of the East Logansport bridge, in 1990 the West Virginia Department of Transportation (WVDOT) decided to build a second Tension Arch bridge. The bridge replaced a 140 ft. Pratt Truss bridge located on WV Route 16, a primary route through the town of War, in the state's southern coal fields.

The War bridge was designed for an HS-25 loading using the Allowable Stress Design provisions of the AASHTO Specifications and an ADT of 3800. A 26-ft. roadway width and a 5-ft. sidewalk accommodate two lanes of traffic and pedestrians. The bridge is straight and unskewed.

The Tension Arch design resulted in a single clear span of 141 ft. Four 48-in.-diameter galvanized steel tube girders consisting of five segments were used. Each girder was post-tensioned with 17 epoxy coated, seven wire strands located inside the girders. The post-tensioning strand was anchored at the back face of the two 4-ft.-wide concrete wall abutments to create the rigid frame. Fourteen precast concrete deck panels were connected to the tube girders to obtain composite action.

Construction Sequence

The War bridge used the following construction sequence, typical of any Tension Arch Bridge. Seven distinct stages are required

Stage 1: The abutments are constructed using conventional concrete forming and placement techniques. A hinge detail is provided at the base of the abutment wall to accommodate rotations caused by expansion and contraction.

A fabricated steel frame is cast in the top of the abutments and is used to align anchor bolts, post-tensioning strand conduits, and anchor plates. Use of a shop fabricated frame insures that these items will be properly

located in the field.

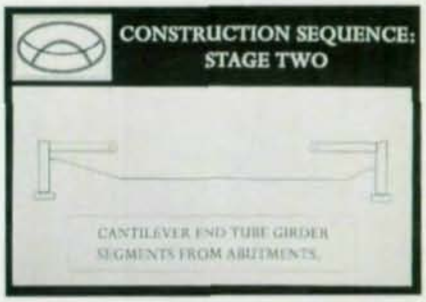
Stage 2: Two tube girder segments of a single girder line are attached to anchor bolts at the front face of each abutment, creating cantilevers at each end of the bridge.

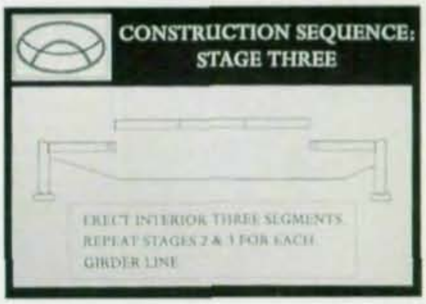
Stage 3: The remaining girder segments of the same girder line are bolted together at the site and then dropped into place between the two cantilevered segments. Stages 2 and 3 are then repeated for the remaining girder lines.

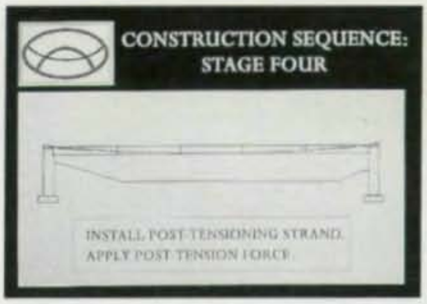
Simple single angle cross frames are installed to insure proper alignment of the girders and to maintain this alignment during the post-tensioning operation.

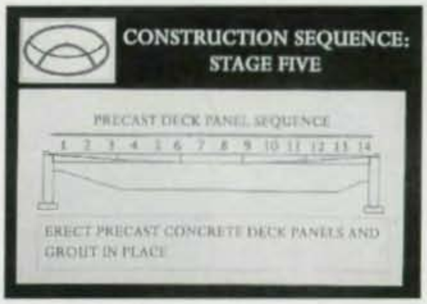
The tube girders are fabricated to be 6" shorter than the clear span. This 3" gap at each end of the girder line provides a very large tolerance that allows easy erection of the steel. After all steel is erected and the cross frames are installed, the two 3" gaps are filled with non-shrink grout. Therefore, the last component of the rigid frame goes in as a liquid.

Stage 4: Post-tensioning strand is installed and stressed. Each strand passes through an abutment and into a tube girder, where it crosses a hold-down point at each tube girder segment connection.









Stage 5: Precast concrete deck panels are erected and grouted in place. The deck panels are erected consecutively from one end of the bridge to the other, reducing the crane lift requirements.

Stage 6: The cast-in-place sidewalk and parapets are constructed using conventional techniques.

Stage 7: A waterproofing membrane is placed over the deck panels and an asphalt wearing surface is applied. The bridge can then be opened to traffic.

Construction of the War bridge was possible because the participants were willing to consider a new way of doing things.

Considering the uniqueness of the Tension Arch system, construction proceeded with little incident. Valuable input from WVDOT Personnel, the contractor and sub-contractors, and AISC, helped the success of the project.

The engineer was Alpha Associates, Inc. of Morgantown, WV, and the contractor was Bilco Construction Company, Inc. of South Charleston, WV. Post-tensioning strand and anchors were supplied by Florida Wire and Cable Company of Jacksonville, FL. Precast concrete deck panels were supplied

by Eastern Vault Company, Inc. of Princeton, WV.

The Tension Arch Bridge system was conceived to be a manufactured product. The details used in the segmental construction of Tension Arch Bridges could be mass produced to reduce cost.

The Tension Arch is a true rigid frame, combining steel tube girders and concrete abutments using post-tensioning to create a very stiff jointless bridge.

Finally, with hot-dip galvanized steel, epoxy coated post-tensioning strand, and with the stiff, jointless superstructure, the Tension Arch is very corrosion and fatigue resistant.

These factors combine to create an alternate steel bridge system that could result in lower installation and maintenance costs.

David L. Weaver, P.E. is a Senior Engineer with KCI Technologies, Inc. in Manassas, VA. Previously, he served as the Project Manager during construction of the War Bridge for Alpha Associates, Inc. in Morgantown, WV. Samuel G. Bonasso, P.E. is the President of Alpha Associates, Inc. in Morgantown, WV, and is the founder of the Tension Arch Structures Company, Inc.

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Bridge Expansion Joints & Bearings

Product: Seismic system
Company: **FIP Structural Systems**
Address: 38 Chatham Road
P.O. Box 604
Short Hills, NJ
07078
Phone: 201/376-8089
Fax: 201/376-7937

FIP offers bridge designers custom designed solutions to meet their most difficult seismic problems. FIP devices with custom response integrate multiple elements, each with a specific function, into a single system to yield the desired response. The basic building blocks of the system consist of: a conventional bearing, usually a pot type, to accommodate vertical loads and rotation; a lubricated PTFE/ stainless steel sliding surface to allow relatively unrestricted horizontal movement; and specially designed and manufactured austenitic steel dissipation pins that elastically resist horiziontal service loads and, when plastically deformed, limit the loads transferred to the substructure to a safe and predicatable level.

Circle no: 105

Product: Slide bearings
Company: **Voss Engineering**
Address: 6965 Hamlin Ave.
Lincolnwood, IL
606043

Voss Engineering offers a structural bearing pad design handbook. The handbook descirbes the performance characteristics of Sorbtex and Fiberlast for thicknesses 1" and greater. Also included in the publication is a description of the performance of Voss slide bearings. The handbook is the result of an extensive tesitng program and includes test results as well as design equations and tables. This data allows the design engineer, architect and detailer an opportunity to make design decisions based on material performance.

A bearing pad design computer program also is available. The program allows the user to design Sorbtex, Fiberlas and Voss slide bearings using an IBM PC or compatible computer.

Circle no.: 106

Product: Bridge Bearings
Company: **Merriman**
Address: 100 Industrial Park
Hingham, MA 02043
Phone: 617/749-5100
Fax: 617/749-3560

Lubrite self-lubricating bearings accommodate expansion, contraction and rotation of structural members—without maintenance or supplementary lubrication. The bearings also are unaffected by temperature extremes, immersion and/or corrosion. The company produces bearings for a wide variety of applications: flat expansion plates are designed to accommodate expansion and contraction in a single plane; radius plates are flat on one face and either concave or convex on the opposite face, with the radius plate accommodating the deflection or rotation of the structural member and the flat face providing for linear expansion and contraction; and spherical plates provide for rotation or deflection in any direction as well as normal expansion and contraction. All bridge bearings can be supplied in standard Lubrite or Lubrite F 100% teflon fiber mat, which offers a very low 0.03 coefficient of friction. The company offers a wide variety of completed assemblies, as well as design assistance.

Circle no.: 107

Product: Seismic Isolation
Company: **Dynamic Isolation Systems**
Address: 2855 Telegraph Ave.
Berkeley, CA 94705
Phone: 510/843-7233
Fax: 510/843-0366

A bridge is a dynamically loaded structure that must perform reliably throughout its lifetime when subjected to a variety of non-seismic short-term and long-term forces. DIS Force Control Bearings promote superior structural performance under these non-seismic conditions, while also preventing earthquake damage and downtime.

Circle no.: 108

Product:	Structural bearing
Company:	**Watson Bowman Acme**
Address:	95 Pineview Dr. Amherst, NY 14228
Phone:	716/691-7566
Fax:	716/691-9239

The Wabo-Fyfe Structural Bearing from Watson Bowman Acme is designed for heavily- trafficked bridge structures with movement capabilities from 3" to 30". Simply put, it works by dividing large movement into multiple 3" segments. Each 3" module has individual steel support beam mechanism, rigid welded connection and double-walled neoprene sealing element to protect against water and chloride intrusion. The counterforce control mechanism allows the modular system to move in harmony with the thermal expansion and contraction of the structure. This accordian- like design exhibits a long maintenance-free life and protects substructural steel from corrosion. The product features smooth riding surfaces, equidistance control and controlled folding movements.

Product:	Expansion Joint
Company:	**E-Poxy Industries**
Address:	14 West Shore St. Ravena, NY 12143
Phone:	518/756-6193
Fax:	518/756-3003

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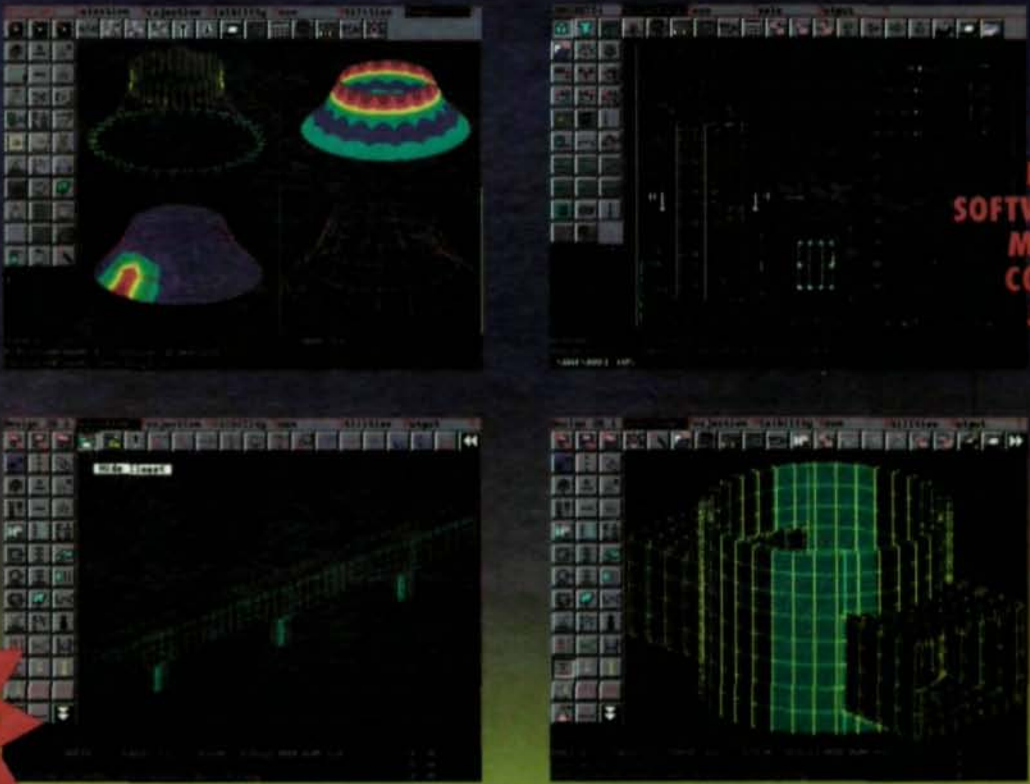

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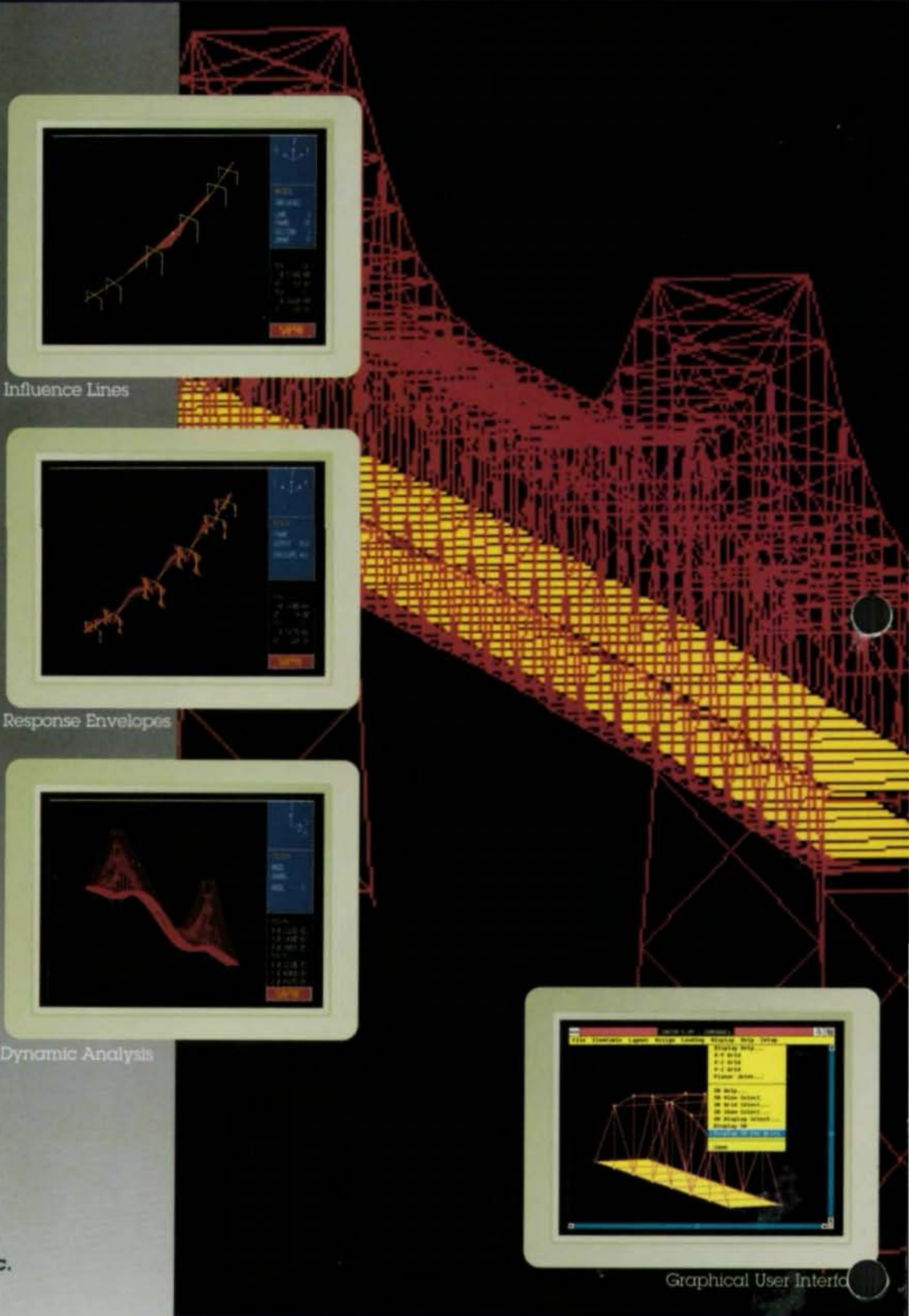

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