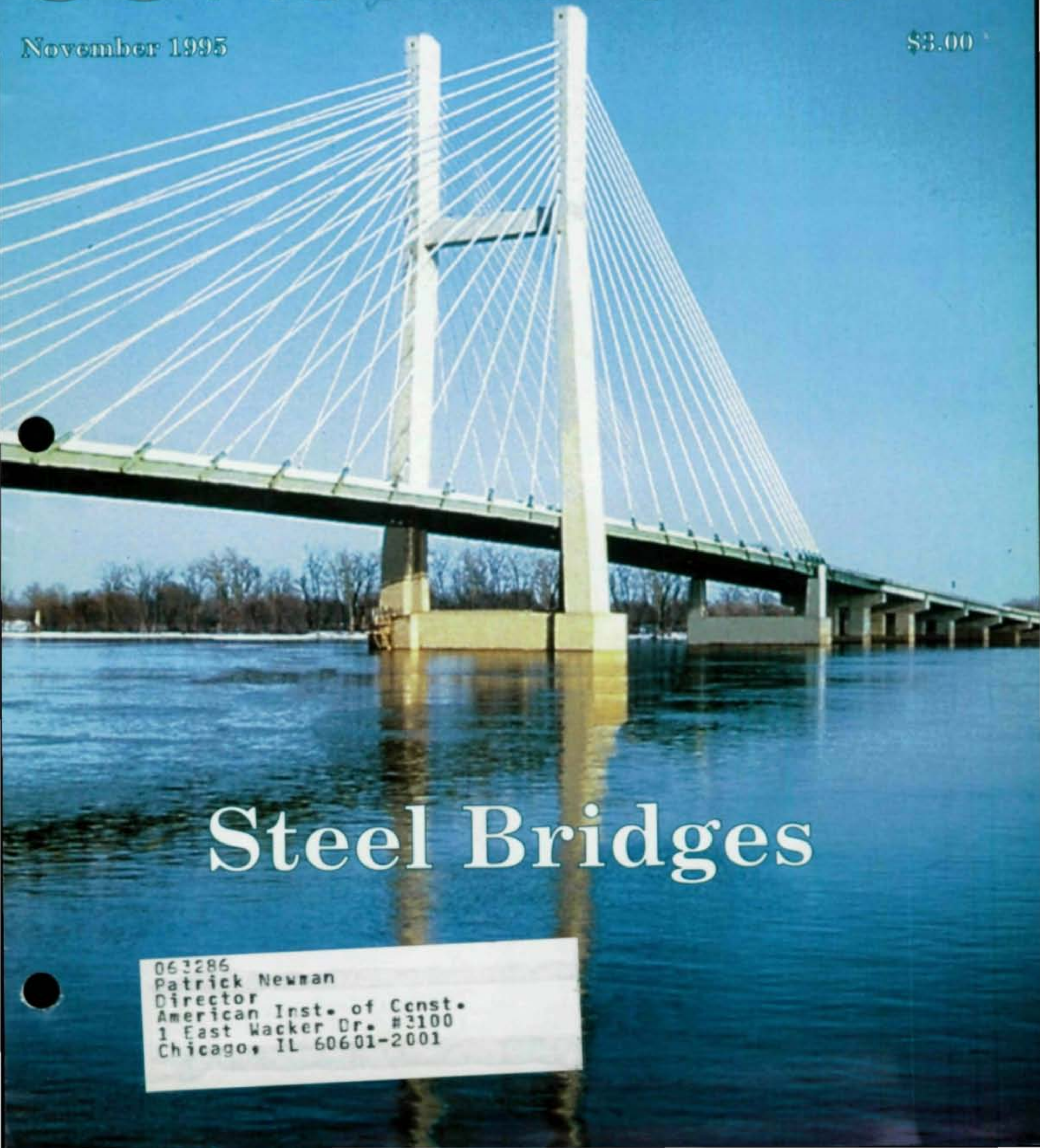
MODERN STEEL CONSTRUCTION

November 1995

$3.00



Steel Bridges

NCJ New Columbia Joist Company

How To Estimate Joist DEFLECTIONS

All joists and joist girders made by **NCJ** are furnished with standard camber. The amount of camber can be *approximated* by the equation: $y_c = L^2/2400$, inches, where L is the span in feet. The word *approximate* is used because the final camber can be influenced by handling during shipping and erection. Non-standard SJI *camber is an extra cost.*

Joist deflection under uniform load can be estimated by the simple beam formula, modified by 1.15 for trusses, $y_1 = 1.15 \times 0.013\ w\ L^4(1728)/EI_J$, where w is in pounds per foot, L is in feet, and E is in psi. I_J can then be found by working the equation backwards with known values from the **NCJ** joist tables.

The total downward movement is obtained by subtracting the camber from the calculated deflection.

Example: A 24K4 joist has a deflection limit (L/360) load of 150 plf on a span of 36'.

$$y_1 = \frac{36 \times 12}{360} = 1.2" \qquad I_J = \frac{1.15 \times 0.013(150)(36)^4(1728)}{29 \times 10^6 \times 1.2} = 187 \text{ inches}^4$$

Or, I_J could also be estimated by;

$$I_J = \frac{1.15 \times 0.85 A_T A_B d^2}{(A_T + A_B)}$$

where A_T is the top chord area, inches^2; A_B is the bottom chord area, inches^2; and d is the distance between the centers of gravity of the top and bottom chords, in inches.

The I_J could also be obtained (most accurately) by calling ☎ NCJ (717) 568-6761.

NICHOLAS J. BOURAS, INC.
475 Springfield Avenue, P.O. Box 662
Summit, NJ 07902-0662
(908) 277-1617

United Steel Deck, Inc.

3 joist design data sheet

Please circle # 33

ARBED

HISTAR® Grades 50 and 65.

WIDE FLANGE & TAILOR - MADE BEAMS

- High Strength, Low Carbon, Fine Grain, Weldable Without Preheating & Good Ductility.
- Reduces Material, Handling, Transportation, Erection & Welding Costs.
- Sectional properties are available on a free floppy disc (Lotus 1-2-3, Quattro Pro and ASCII versions for IBM, Excel for Macintosh) and the complete **ARBED** database is ready to use in the design softwares GTSTRUDL, ETABS, SAP90 AND STAAD-III.

TRUSS APPLICATIONS

- *Sports Arenas/Stadiums*
- *Airports (Hangars/Terminals)*
- *Convention Centers*
- *Gymnasiums*
- *Shopping Malls*
- *Casinos*

COLUMN APPLICATIONS

All of the above PLUS:

- *High-Rise Buildings*
- *Hospitals*
- *Heavy Industrial Plants*
- *Power Plants*

HISTAR® is a registered trade-mark of ARBED.

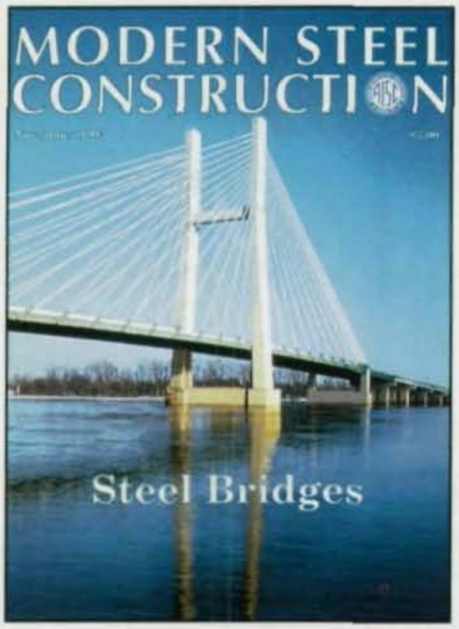
For complete information, availability, literature and floppy disc, contact one of our TradeARBED offices at the following locations:

- ❑ 825 Third Ave., New York, NY 10022. 1-800-272-3369, FAX (212) 355-2159.
- ❑ 60 E. Sir Francis Drake Blvd., Larkspur, CA 94939. (415) 925-0100, FAX (415) 461-1624/8257.
- ❑ 390 Brant Street, Suite 300, Burlington, Ontario, Canada L7R 4J4. (905) 634-1400, Fax (905) 634-3536.

Please circle # 35

MODERN STEEL CONSTRUCTION

Volume 35, Number 11 **November 1995**



Cable stayed bridges create a dramatic presence while also providing an economical solution. The story behind this Iowa bridge begins on page 18.

FEATURES

DEPARTMENTS

Modern Steel Construction (Volume 35, Number 11). ISSN 0026-8445. Published monthly by the American Institute of Steel Construction, Inc., (AISC), One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Advertising office: Facinelli Media Sales, O'Hare Lake Office Plaza, 2400 E. Devon Ave., Des Plaines, IL 60618 (708) 699-6049.

Subscription price:
Within the U.S.—single issues $3; 3 years $85.
Outside the U.S.—single issues $5; 1 year $36; 3 years $100.

Please fax any address changes to 312/670-5403 or to the address below.

Postmaster: Please send address changes to Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Second class postage paid at Chicago, IL and at additional mailing offices.

Reserve Strength

THE CHICAGO AREA LOST ONE OF MY FAVORITE LANDMARKS LAST MONTH. For many years, the Lindheimer Observatory stood guard on the lake shore on Northwestern University's campus. Unfortunately, it was a poorly maintained structure: its supporting steel frame was coated with a flaking lead-based paint and the inside was insulated with asbestos. Since the observatory no longer served a research function (there's a bit too much light pollution 10 miles north of downtown Chicago in Evanston), the university opted—despite the considerable chagrin of much of the surrounding community—not to spend the necessary $750,000 to renovate a building whose sole purpose would have been aesthetic.

Instead, the school hired a demolition company to perform what initially looked to be a fairly simple task. Given the structure's fairly isolated location, controlled explosives seemed like a logical way to bring down the once glorious structure. But despite its delicate appearance, steel can be a surprisingly strong material. So the demolition company set its explosives, and lo and behold, all they succeeded in doing was to bend some of the steel members.

Not to be denied, the workers next decided to hook chains up to the structure and simply pull it down. But again, the effort was to no avail. Instead of pulling down the observatory, all they did was create the Leaning Tower of Evanston.

Finally, they opted for the slow yet sure and disassembled the entire structure piece by piece.

As we watched this comedy of errors unfold night after night on the local news, my wife wondered why they were having so much difficulty. What immediately crossed my mind was the steel buildings that survived the Northridge Earthquake despite losing the integrity of many of their connections. Steel buildings have a remarkable reserve strength that isn't calculated in any design. The uncalculated strength comes from elements such as gravity framing, which actually provides some lateral strength.

Designers don't often consider the reserve strength of steel, but it's a key advantage of steel structures and it's one of the reasons that you have to work awfully hard to bring a steel structure down. **SM**

Editorial Staff
Scott Melnick,
Editor and Publisher
Patrick M. Newman, P.E.
Technical Director
Charlie Carter,
Senior Technical Advisor
Jacques Cattan,
Technical Advisor

Editorial Offices
Modern Steel Construction
One East Wacker Dr.,
Suite 3100
Chicago, IL 60601-2001
(312) 670-5407
Fax 312/670-5403

Advertising Sales
Facinelli Media Sales
2400 E. Devon Ave.,
Suite 267
Des Plaines, IL 60018
(708) 699-6049
Fax 708/699-8681

STEEL QUIZ

Steel Quiz, a new monthly feature in Modern Steel Construction, allows you to test your knowledge of steel design and construction. Unless otherwise noted, all answers can be found in the *LRFD Manual of Steel Construction*. To receive a free catalog of AISC publications, circle #10 on the reader service card in the back of this magazine.

1. Physically, the U.S. customary shape series and metric shape series are identical, True or False?

2. Cross-sectional dimensions and standard mill practice may be found in which of the following documents?
 a) ASTM A36
 b) ASTM A6/A6M
 c) AISC's Manual of Steel Construction
 d) a and c
 e) b and c

3. An installed bolt with the point of the bolt flush with the face of the nut is acceptable, True or False?

4. Which of the following methods for bolt installation is not recognized by the RCSC and AISC?
 a) standard torque
 b) turn-of-nut
 c) snug-tight
 d) calibrated wrench

5. In which of the following positions can weld metal be deposited at the fastest rate?
 a) horizontal
 b) vertical
 c) flat
 d) overhead

6. Which is more costly: a 1/4-in. fillet weld ten inches long or a 1/2-in. fillet weld 5 inches long?

7. If an extended end-plate moment connection is specified as slip-critical, the slip resistance of the bolts at the tension flange must be reduced for the tension present, True or False?

8. Which of the following factors is used to adjust for inelastic column behavior?
 a) K
 b) SRF
 c) m
 d) n
 e) m and n

9. In a partially composite beam, which of the following controls the flexural design?
 a) compression in the concrete
 b) tension in the steel
 c) compression in the steel
 d) shear strength of the shear stud connectors

10. Which of the following trusses does not utilize diagonal members?
 a) Pratt
 b) Fink
 c) Warren
 d) Vierendeel

Answers on PAGE 13

01244

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 800/644-2400.

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

Are there special requirements for the design of high-strength A325 or A490 bolts that are going to be in a high temperature area?

ASTM A325-89, paragraph 1.2 stated in part: "...where elevated temperature applications are involved, Type 1 bolts shall be specified by the purchaser."

However, this statement was removed from the 1993 update of the specification. This statement was all too encompassing , and not knowing the actual application for the bolts, it was determined that this statement should be removed. It is possible that the user could place these bolts under temperatures that may affect the physical properties of the bolts. An example being that they were not designed to be used in high-pressure, high-temperature service areas found in power plants.

ASTM A490 specifications do not address the use of these bolts in elevated temperature areas.

There are bolts manufactured for use in elevated temperature areas.

Not knowing the specific application or design requirements the bolts are being used for; look into the possibility of substituting these bolts with ASTM A193/A193M "Standard Specification for Alloy Steel and Stainless Steel Bolting Materials for High-Temperature Service" and ASTM A449 "Standard Specification for Quenched and Tempered Steel Bolts and Studs"

Tom Slovick
Industrial Steel
Mims, FL

If a W-shaped column is made up of three welded plates, how does one design the welds connecting the plates together?

In general it is best to weld along the full length of the web so one may ignore buckling of the individual elements, i.e. the web and flange. One must still consider buckling according to Table B5.1, Limiting Width-Thickness Ratios for Compression Elements, from the LRFD Manual of Steel Construction. Assuming there is no moment introduced into the column one should use the minimum weld size given in Tables J2.3 and J2.4 of the LRFD Manual.

If the column is subjected to bending stresses which may be due to eccentricities or a lateral load the weld must be designed to transfer the load between the web and the flange like one would do for a beam. The shear flow can be calculated using the formula:

$$\text{Shear Flow} = \frac{V_u Q}{I_x} \frac{kips}{in.}$$

The capacity of the weld is then calculated as:

Continuous Weld Design Strength = ϕR_{nw}

An example of this type of design can be found in Chapter 11 of Salmon and Johnson, Steel Structures. In most cases the minimum weld size will control.

James D. Palmer
Butler Manufacturing Company
Grandview, MO

For a continuous trolley beam with multiple spans and cantilevered ends what is the lateral unbraced length for the bottom flange?

One answer published in the July 1995 issue suggests that the lateral unbraced length is simply twice the cantilever distance. A more complete and exact solution to this problem is offered in a 3rd Quarter 1985 Engineering Journal paper by N. Stephen Tanner "Allowable Bending Stresses for Overhanging Monorails." By considering the ratio of the overhanging span to the adjacent interior span in developed equations, a less conservative answer may be computed.

Nestor Iwankiw
AISC, Inc.
Chicago, IL

When considering a point load on the standing leg of an angle, what provisions are there for determining the effective allowable member width?

The following two articles will give a practical approach including finite element study and the latter is a theoretical method for solving the problem of how to distribute a point load on a shelf angle.

Tide, R.H.R and Norbert V. Krogstad, "Economical Design of Shelf Angles," Proceedings, Symposium on Masonry: Design and Construction, Problems and Repair, Miami, FL, December 8, 1992, STP 1180, May 1993, American Society for Testing and Materials, Philadelphia, PA

Jaramillo, T.J., "Deflections and Moments Due to a Concentrated Load on a Cantilever Plate of Infinite Length." Journal of Applied Mechanics, March 1950, American Society of Mechanical Engineers, New York, NY

R.H.R. Tide
Wiss, Janney, Elstner Associates, Inc.
Northbrook, IL

Given a wall of sheet metal or plate subjected to fluid pressure and stiffened by same size parallel members spaced regularly, what section (or width) of the wall shall be used that contributes to the section of a stiffener? The stiffening member may be a flat bar, an angle, or a channel or any other section.

The section of wall that contributes to the section of a stiffener is defined by means of an effective width be which shall not exceed:

a) The geometric condition, that is the distance between center line of adjacent beams.

b) The shear lag condition, which may be estimated as $^1/_4$ of the effective beam span (length of positive moment area of the rib).

c) The stability condition of the plate between stiffeners (see "Specification for the Design of Cold Formed Steel Structural Members", AISI, 1986, B2.1).

With the effective width so defined a verification must be done of the resistance of welding between the sheet and reinforcement ("Coligon" stresses), and possible buckling due to weld spacing in the line of stress.

Miguel A. Dodes Traian
Buenos Aires, Argentina

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

Questions and responses will be printed in future editions of Steel Interchange. Also, if you have a question or problem that readers might help solve, send these to the Steel Interchange Editor.

Is there any criteria, except direct field measurements by drilling holes, to determine the percent loss of capacity in steel bridge members due to weather exposure for the purpose of rating the truck capacity of bridges.

Mike Alomari
Wayne County
Sterling Heights, MI

Where should a control joint be located in a composite large composite floor? Should it be located over the top of either the girders or the beams? What happens to the strength of the shear studs if the concrete cracks over the shear studs? Does this crack go all the way through the slab to the top of the steel beam?

Can any beam be cambered without heating? Is there a slenderness limit for the web to prevent buckling of the web while cold cambering?

Is it acceptable to either mechanically galvanize or hot dip galvanize high strength bolts? Are there different requirements for the installation depending on how the bolt is galvanized?

Join The Wasser Revolution!

Stop using 2-part epoxies, 2-part urethanes and inorganic zincs. After 40 years, it's time to move to a coating system that's far better. Wasser has revolutionized the industry. Join Us!

BRIDGES AND LEAD OVERCOAT

ASTORIA BRIDGE, OREGON COAST - Hundreds of bridge projects in environments like Oregon's coast, prove Wasser outperforms all other coatings.

WATER AND WASTEWATER

NORTH POLE ALASKA - Painting continues on these sludge beds, while standing in snow. Wasser outperforms epoxy coatings on these demanding projects.

UTILITIES, DAMS AND LOCKS

PRIEST RAPIDS DAM, COLUMBIA RIVER - Wasser has been chosen for their superior performance by agencies throughout the world.

PULP AND PAPER

CHIP CRANE, ROSEBURG LUMBER - Wasser Coatings allow perfect results in outdoor winter painting or indoor painting on paper machinery, for virtually every paper company. Imagine, immersion after only minutes, and superior performance.

1. ***Industry's BEST Corrosion Resistance***
2. ***Single Component.***
3. ***No Application Restrictions for Humidity, Dewpoint or Temperature (20°F)***
4. ***World's Largest Producer of Single Component Moisture Cure Urethanes.***
5. ***North America's Largest Manufacturer of Micaceous Iron Oxide Coatings.***

WASSER

HIGH·TECH COATINGS

FOR FREE VIDEOS

Call 800-MC-PAYNT

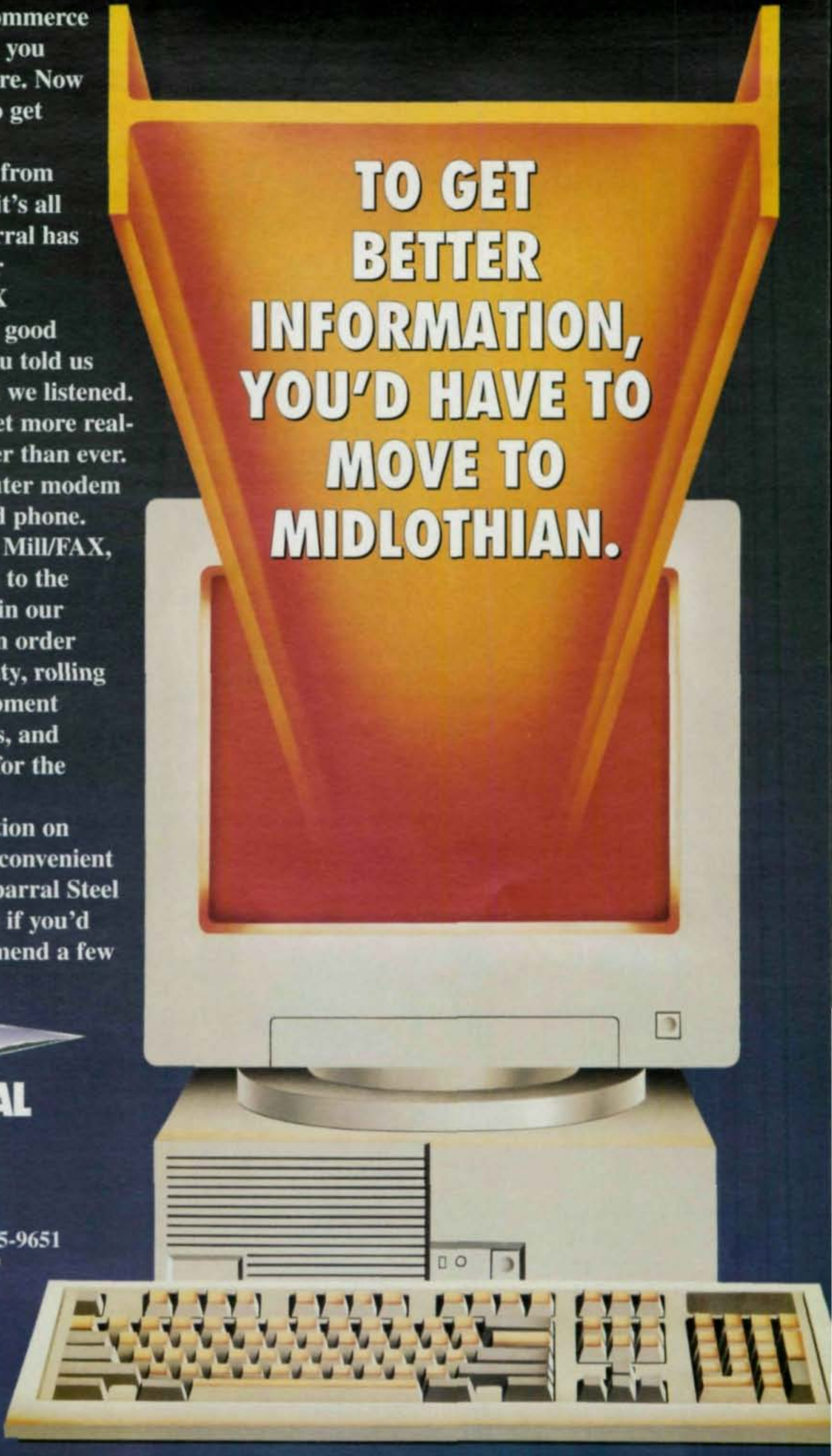
For Information About Wasser Circle Reader Response #50

Wasser, the Leader of the Coating Revolution!

Although the Midlothian Chamber of Commerce might disagree, you don't have to move here. Now there's a better way to get the most current and complete information from Chaparral Steel, and it's all thanks to you. Chaparral has recently upgraded our Mill/Net and Mill/FAX capabilities, making a good system even better. You told us what you wanted, and we listened. As a result, you can get more real-time information faster than ever. All it takes is a computer modem hook-up or a FAX and phone.

With Mill/Net and Mill/FAX, you have direct access to the most up-to-date facts in our system, including open order status, stock availability, rolling schedules, 10-day shipment history, rail shipments, and scheduled shipments for the day–all in an instant.

For more information on these free, even more convenient services, contact Chaparral Steel at 1-800-527-7979. Or if you'd prefer, we can recommend a few good Realtors.



CHAPARRAL STEEL

Chaparral Steel
300 Ward Road
Midlothian, Texas 76065-9651
Toll Free (800) 527-7979
U.S. and Canada
Local (214) 775-8241
Fax (214) 775-6120

Please circle # 43

STEEL QUIZ

The answers to this month's steel quiz are as follows:

1. True. Because our current shape series is one of the most efficient in the world, and because the inch-series dimensions are nominal (e.g., a W14 is not exactly 14 in. deep), the metric series is simply a soft conversion of it. Refer to AISC's *Metric Properties of Structural Shapes*.

2. e. ASTM A6/A6M is the standard which specifies cross-sectional dimensions and standard mill practice for rolled shapes. AISC's *Manual of Steel Construction* incorporates this information in Part 1.

3. True. RCSC Specification Commentary Section C2 defines full thread engagement as "having the end [point] of the bolt at least flush with the face of the nut."

4. a. Both RCSC and AISC discourage use of a standard uncalibrated torque value. Such an uncalibrated value may be too high and break well lubricated bolts or, more importantly, may be too low and result in undertensioned bolts if the thread lubrication is poor or the threads are dirty or corroded. Therefore, if torque is to be used, it must be calibrated according to RCSC Specification Section 8(d)(2).

5. c. Welding in the flat position allows the fastest deposition rate and therefore is the most economical welding position.

6. At first glance these welds seem to be of equal cost because they are of equivalent strength. However, because the volume of weld metal is proportional to the square of the weld size, the $^{1}/_{2}$-in. weld uses twice as much weld metal as the $^{1}/_{4}$-in. weld. Additionally, a $^{1}/_{2}$-in. weld will require multiple weld passes. In the end, the same strength will cost more than twice as much with the $^{1}/_{2}$-in. weld.

7. False. Because the tensile and compressive flange forces are equal, any loss of slip resistance adjacent to the tension flange of the beam is compensated for by an increase in slip resistance adjacent to the compression flange.

8. b. The stiffness reduction factor SRF is used to adjust G for inelastic behavior.

9. d. The flexural strength of partially composite beams is controlled by the shear strength of the shear stud connectors.

10. d. A Vierendeel truss utilizes chord and vertical members without diagonals. Therefore, unlike other ideal trusses, Vierendeel truss members must also transmit member forces due to bending.

Please circle # 4

NATIONAL STEEL CONSTRUCTION CONFERENCE EXPANDS SCOPE

SIX TRACKS WILL BE OFFERED AT NEXT YEAR'S NATIONAL STEEL CONSTRUCTION CONFERENCE. New in 1996 will be sessions on computers and erection. In addition, specialized tracks will be offered on: engineering—technical; engineering—management; fabrication; and construction management.

"There was a lot of enthusiasm generated last year when we first introduced specialized tracks," explained Franklin B. Davis, chairman of AISC's NSCC Committee. "The two new tracks are designed to continue and expand on that excitement.

"There has long been a feeling in the engineering consulting community that the erection of steel buildings has not received as much attention as it should. Adding a track on erection should help bring some key issues to the forefront. And adding a computer track is a natural for the show, given its wide attendance from all segments of the steel industry. The advances being made in software have been tremendous during the past few years. The NSCC is an ideal forum where the user and the producer can get together to interface for the benefit of all parties," Davis concluded.

The 1996 Conference will be held in Phoenix on March 27-29. Hot seminar topics include:

• "Selection of Bolted Joint Criteria" with Robert O. Disque, a former long-time AISC engineer and currently a consulting engineer with Besier Gibble Norden in Old Saybrook, CT, and Michael A. Gilmor of CISC and current chairman of the Research Council on Structural Connections;

• "Industrial Building Design" with Jules Van de Pas of Computerized Structural Design in Milwaukee, Duane Ellifrit of University of Florida, and Bob MacCrimman of Acres International, Niagara Falls, Ontario

• "Bar Coding—Fabrication Shop Connections" with Richard Bushnell of Quad II, Shalfont, PA

• Computer Program Usage (MathCAD and Beyond)

• Problem Connections

• "Architectural Fabrication: What is Involved" with David Sailing of Zalk Josephs, Stoughton, WI, and Jim McCrae of Nieder Hauser, Salt Lake City

• "How to evaluate Detailing Software"

• "Steel Erection Awareness"

In addition to more than 30 technical sessions, there will be an exhibit hall with nearly 70 product exhibitors. The sessions qualify for CEU credit and time also is available for extensive industry networking. Also, an extensive package spouse/family activities are scheduled.

As an added benefit, AISC will offer a Short Course on Bolting on Saturday, March 30, 1996. The course is separate from the National Steel Construction Conference and requires a separate registration.

To receive an information and registration packet, please call 800/787-0052 ext. 110.

1996 NATIONAL STEEL CONSTRUCTION CONFERENCE

MARCH 27-29, 1996

PHOENIX CONVENTION CENTER

PHOENIX, ARIZONA

CALL FOR PAPERS: NATIONAL STEEL BRIDGE SYMPOSIUM

THE NSBA NATIONAL STEEL BRIDGE SYMPOSIUM, to be held in Chicago in the Fall of 1996, brings together design and engineering professionals, FHWA and Department of Transportation Officials, fabricators, erectors and contractors to discuss and learn about state-of-the-art bridge design, fabrication and construction techniques. The Symposium includes workshops, lectures, technical sessions and the presentation of the AISC Prize Bridge Awards.

If you are interested in submitting a paper on bridge design, fabrication or erection, or on any aspect of an innovative bridge project (painting, welding, connections, bearings, etc.), send a short summary or abstract to: Fred Beckmann, AISC Director of Bridges, American Institute of Steel Construction, Inc., One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001(fax: 312/670-5403) by December 15, 1995.

Steel Joist Specifications

The Steel Joist Institute's 40th edition of "Specifications, Load Tables and Weight Tables" is now available. The 96-page catalog includes new stability requirements, specifications and load tables for the all-new KCS Joist, fire resistance ratings, metric units, and a one-page method for converting the load tables for use with Load and Resistance Factor Design (LRFD).

Copies of the catalog are available for $20 (or $30 outside the U.S.), including shipping and handling.

For more information, contact SJI, Suite A, 1205 48th Ave. N., Myrtle Beach, SC 29577; 803/449-0487 or **circle no. 54** on the readers service card in the back of this magazine.

Upcoming Conferences

- **ASCE Structures Congress**
  Chicago
  April 15-18, 1996
  (Includes more than 100 technical sessions and combines several specialized conferences: Council on Tall Buildings and Urban Habitat; Structural Stability Research Council; ASCE Committee on Analysis and Computation)
  Contact: ASCE at 212/705-7496
- **1996 FHWA-AASHTO National Metric Conference**
  Radisson MetroDome, Minneapolis, MN
  April 15-18, 1996
  Contact: Bob McPartlin, MN/DOT, 612/296-4337
- **Structural Steel—Developing Africa**
  Johannesburg, South Africa
  August 13-15, 1996
  Contact: SAISC, 7th Floor, Metal Industries House, P.O. Box 1338, Johannesburg 2000, South Africa; 011/838-1865; fax: 011/834-4301
- **Semi-Rigid Structural Connections**
  Istanbul, Turkey,
  September 25-27, 1996
  (Colloquium will include connections in steel, concrete, composite, timber and masonry; reports of the pan-European COST C! project; and links between research findings and their practical applications.)
  Contact: Reidar Bjorhovde, Department of Civil and Environmental Engineering, University of Pittsburgh, 937 Benedum Hall, Pittsburgh, PA 15261-2294;
  412/624-9876;
  e-mail: bjorh@civ.pitt.edu

Updated Bridge Welding Code

A newly updated "Bridge Welding Code (ANSI/AWS D1.5-95)" is now available from the American Welding Society. The specification, prepared in conjunction with AASHTO, describes the welding requirements for welded highway bridges made from carbon and low-alloy steels. The 220-page document costs $96 and includes a means to regulate welding in steel construction, a section of added rules for bridges and requirements for fabricating fracture critical structures. In addition, it contains information on workmanship, specific processes, qualification, inspection and statically loaded structures.

For more information, contact AWS Order Department, 550 NW LeJeune Road, Miami, FL 33126; 800/334-WELD or **circle no. 122** on the reader service card in the back of this magazine.

Bolting Tips

The Industrial Fasteners Institute offers a 15-page booklet on "Ensuring Assembly Integrity: Tips for Specifiers, Buyers and Installers of Mechanical Fasteners." The publication offers information on:

- specifying quality fasteners;
- qualifying vendors;
- reducing costs;
- recognizing quality;
- maintaining lot integrity and traceability;
- following safe, responsible maintenance practices; and
- finding help when you need it;

According to the booklet, "Despite efforts to legislate protection for the responsible purchaser/user of fasteners, there is still ample opportunity for confusion—if not outright fraud—in the marketplace. *There is really only one certainty: you, the buyer or user of fasteners, are ultimately responsible to ensure that the right fastener is specified and that the specified fastener is delivered and installed correctly.*" Among the many examples given in the booklet is that of the confusion between Grade 8 and Grade 8.2 bolts.

Copies of the booklet are available for $3.75 each, with discounts available for quantities of 26 or more.

Also available is the *Fastener Application Advisory* newsletter. The newsletter, aimed at those who specify or purchase mechanical fasteners, offers a wide variety of information, ranging from an FAA probe of bogus bolts to information on hydrogen embrittlement. Subscriptions to the are $89 for the first copy, with discounts for additional orders.

For more information, contact the Industrial Fasteners Institute, East Ohio Building, Suite 1105, 1717 East Ninth St., Cleveland, OH 044114-2879; 216/241-1482; fax: 216/241-5901.

The Decking Is Stacked In Your Favor.

All the types of premium steel decking you need, rolled to your specific job requirements. Delivered where you need it, when you need it. From six strategically positioned manufacturing/service locations throughout the nation:

- Beech Bottom, West Virginia
- Gary, Indiana
- Lenexa, Kansas
- Houston, Texas
- Fort Payne, Alabama
- Wilmington, North Carolina

Wheeling Corrugating is growing, and our expanded production capacity allows us to provide you with unsurpassed service, delivery scheduling and coordination. For more information on Wheeling's full line of steel decks and our network of service capabilities, give us a call today at the following sales offices: Wheeling, WV, 304-234-2300; Lenexa, KS, 913-888-4900; Houston, TX, 713-674-6737.

Wheeling Corrugating Company

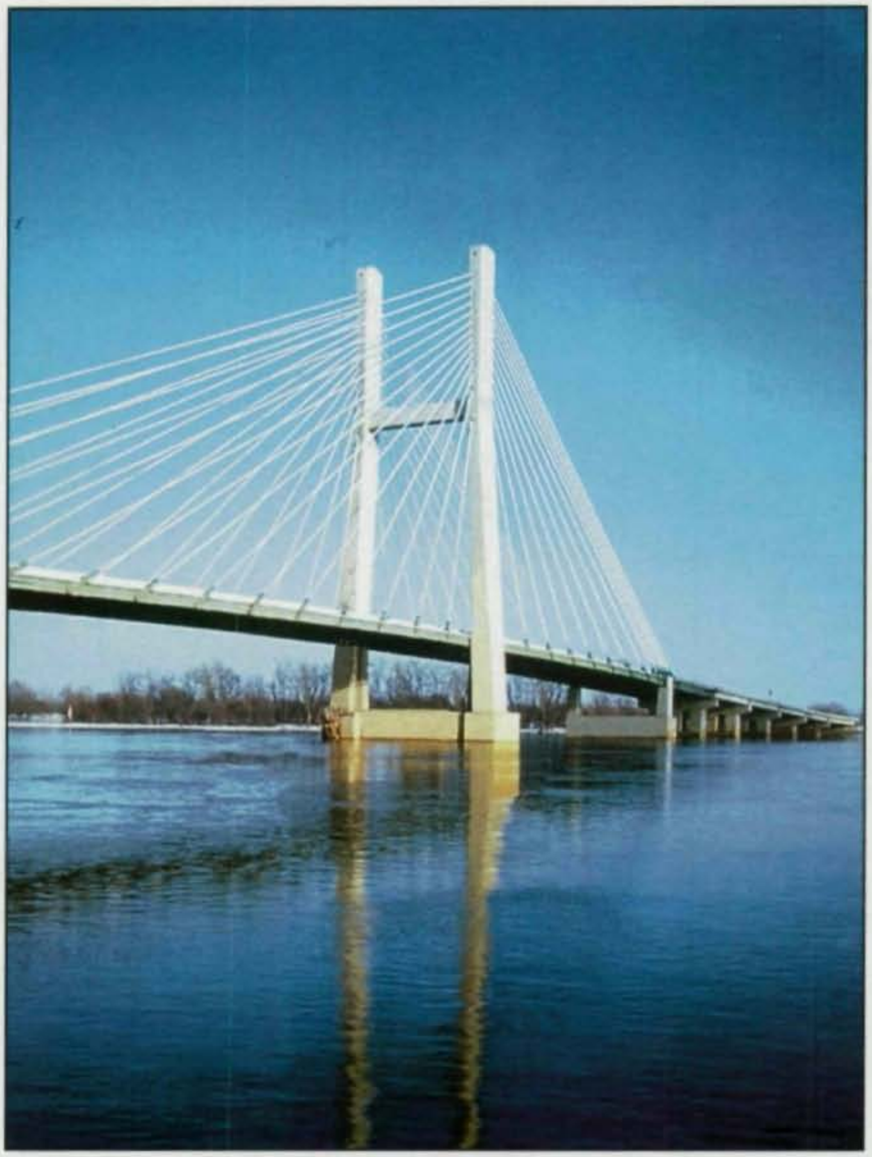
A DIVISION OF WHEELING-PITTSBURGH STEEL CORPORATION

Since 1890

Please circle # 99

A New Link Across the Mississippi River

A cable stayed bridge, with the cables arranged in a modified fan pattern, provided both an economical shallow deck structure and an aesthetic appearance



By Ernst H. Petzold, P.E.

BY THE MID-1980S—DESPITE A NUMBER OF EFFORTS OVER MANY YEARS AT REINFORCING AND MODIFIYING THE STRUCTURE—it was obvious that a replacement was required for a vintage 1917 Mississippi River crossing at Burlington, IA. In 1984, the Iowa and Illinois Departments of Transportation initiated a study by Sverdrup Corporation of St. Louis to investigate replacement alternatives for the MacArthur Bridge.

The design study developed into the preparation of final plans and specifications by Sverdrup for both steel and concrete alternatives for the cable stayed main spans and girder approaches and also included extensive modifications to the Burlington Interchange. The steel main span option was the alternative selected for construction.

Cable-Stayed Bridge Design

The bridge is asymmetric with a 660-ft main span and a 405-ft. side span. A third 180-ft. long span is also part of the cable stayed section. The cables are arranged in a modified fan pattern, which provides a compromise between the efficiency of the pure fan arrangement and the practical and aesthetic characteristics of a harp arrangement. To permit the use of a shallow deck structure, which is both economical and attractive, the system consists of a relatively large number of cable stays.

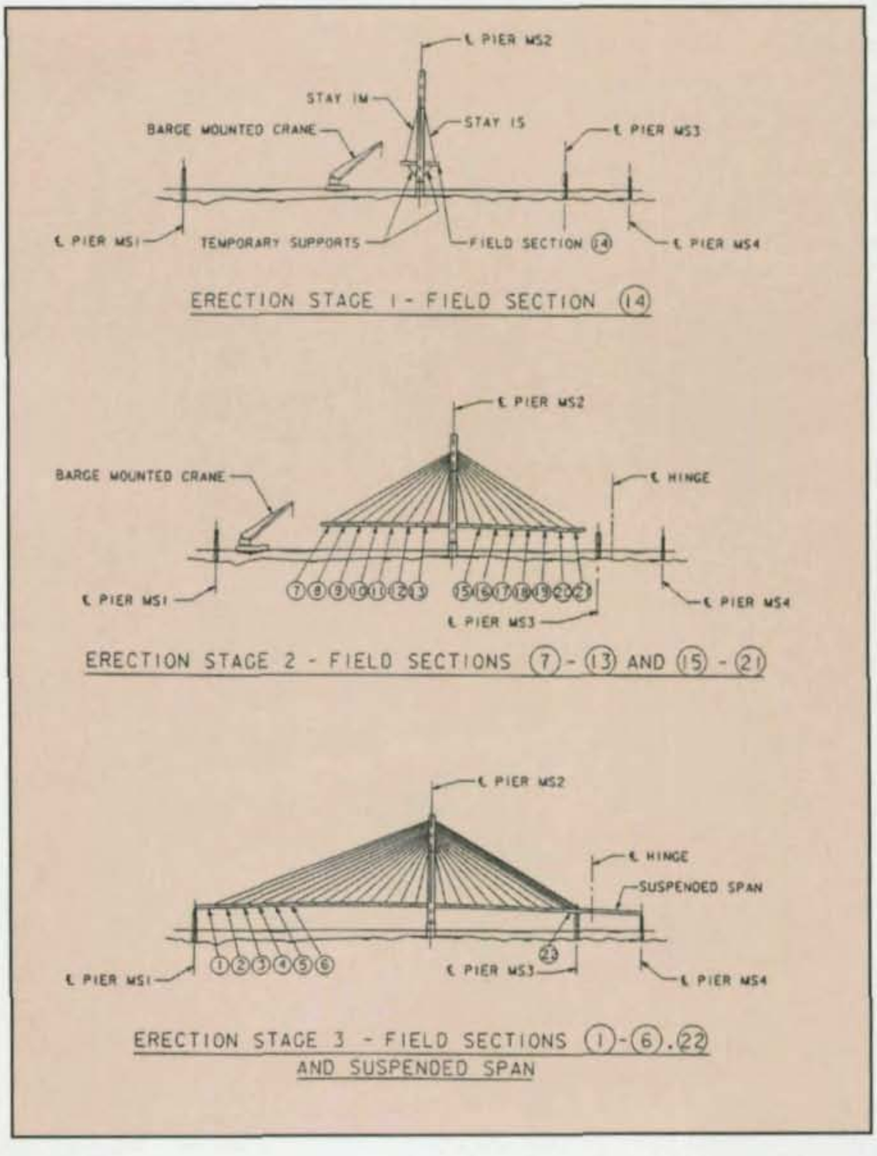
In the transverse direction the two essentially vertical planes of cables are supported by the 'H' shaped tower.

The deck of the cable stayed unit is extremely simple and straightforward. It consists primarily of steel edge girders at about 84-ft. centers, steel floorbeams at 15-ft. centers and precast concrete deck panels. This system is a composite one where the concrete deck participates with both the floorbeams (principally in resisting live load bending moments) and with the edge girders (in resisting mainly dead and live load axial thrusts). The concrete deck also functions as a diaphragm in resisting lateral loads. Longitudinal post-tensioning is used to provide sufficient capacity for the deck to span between floorbeams. The post-tensioning is done prior to effecting the composite connection with the edge girders to prevent the undesirable transfer of the imposed axial force to the steel girders.

A W16 strut under the median barrier is included in the deck system as a construction expedient. The precast deck panels are initially supported on leveling screws bearing on the edges of the floorbeam flanges. During placement of the panels it is possible, therefore, for the floorbeams to be subjected to significant torsional moments. The central strut resists this torsional effect by creating a couple between adjacent floorbeams.

The basic edge girder deck arrangement extends from Pier MS1 to the hinge near Pier MS3. The suspended span between this hinge and Pier MS4 is a conventional multiple steel girder layout with a composite cast-in-place deck slab. A box girder floorbeam at the hinge picks up the reactions from the multiple girder system and transfers them to the edge girders at this location.

The vertical cables are in the same plane as the edge girder webs. This arrangement eliminates the generation of primary



Cable end connections were assembled to the already erected edge girders and included a temporary deviator for use during installation of the cables (right).

During the cantilever erection fo the main span, the contractor provided additional stability by the use of temporary support bents in the side span (top).

torsional moments in the edge girders. The cables are dead-ended at the deck level; all jacking is done at the tower. The deck connection is made via multiple sets of steel plates which transfer loads from the girder web to the cable. The steel plates pass through blockouts in the concrete deck, around the top flange of the girder and are connected to the girder web using two W14 sections.

The tower is of reinforced concrete with post-tensioning being used in the cable stay anchorage area. The tower legs and strut are box sections with wall thicknesses of about 30 and 15 inches, respectively. In the transverse direction the tower acts as a moment resistant frame. The strut dimensions were selected to optimize this frame action. In the longitudinal direction the tower acts as a free standing cantilever with some elastic support offered by the cable stays. Since the tower end of the stays is the jacking end, the interior of the anchorage area was sized to permit the necessary jacking equipment to be placed and operated.

The cable stays consist of 0.6-in.-diameter epoxy coated prestressing strands contained in a grout -filled polyethylene (PE) pipe. The exterior of the pipe is wrapped with a light colored tape after grouting is completed. The stays are anchored in a matrix of epoxy resin, zinc dust and steel balls contained within a forged steel socket. To allow the steel balls to grip the metallic element of the strand directly, the epoxy coating is removed in the socket region. The epoxy resin/steel ball anchoring method was selected for this project for its favorable fatigue characteristics and due to the fact that it permits full prefabrication of the stay. Prefabrication of the stay can be done in an enclosed shop, thereby assuring maximum environmental protection and thorough inspection during fabrication.

Design Issues

The idealized cable stay system can be thought of as a combination of compression struts (the deck and tower) and tension ties (the stays). Maximum economy is achieved by minimizing the dead load bending moments in the compression elements of the system. The deck moments are minimized by balancing the deck dead weight against the vertical component of each cable. This balancing is accomplished by appropriate cable geometry which effectively reduces the cable support points along the deck to rigid supports (vertically) for dead load. The tower dead load moments are minimized by balancing the vertical loads left and right of the tower centerline. The 405-ft. side span does not totally balance the weight of the 660-ft. main span, however. The weight of the anchor pier (Pier MS3), which is developed by the use of appropriate hold-downs, and the reaction of the suspended span on the hinge account for the remainder of the necessary force.

Due to the importance of the cable lengths in achieving the desired force distribution in the system it was decided in the design stage that construction of the bridge would be by "geometry". In this system the theoretical cable fabrication length is calculated consistent with the desired final geometry and analysis assumptions. Nominal shims to be used with the as-fabricated cable provide the only adjustment to achieve this theoretical length. Prefabrication of the cable permits its construction to a finite length which is consistent with maintenance of overall geometry. The edge girder length and tower height are also important in establishing the system geometry and these elements are also constructed to predetermined dimensions. However, since these latter elements are not adjustable after construction, it is necessary to determine the precise position of both the deck and the tower cable anchorage work points prior to establishing the final cable shim thicknesses.

Deck Construction

While the foundation, substructure and tower construction provided their own unique set of challenges, it is the deck construction which is the most intricate and is described in some detail in this section.

The contract documents included a conceptual deck erection sequence. As designed the edge girder consists of a number of sections. These field sections correspond to the length of edge girder between adjacent field splices. These sections are generally 45 feet long and include the connection of one cable stay. Exceptions occur for section 14 (at Pier MS2, which includes two cable connections) and section 22 (at Pier MS3, which provides for the connection of five backstays).

Three primary erection stages were identified. Stage 1 consisted of the erection of the pier table, section 14, at Pier MS2. Stage 2 consisted of the balanced cantilevering of sections sufficient to reach Pier MS3. Stage 3 involved the completion of the major deck structure by erecting section 22 at Pier MS3, the suspended span structural steel and concrete deck and then completing the main span by free cantilevering to Pier MS1. This stage is, perhaps, the most intricate one since it involves the progressive inclusion of the mass of Pier MS3 and the suspended span to balance the increasing weight of the main span.

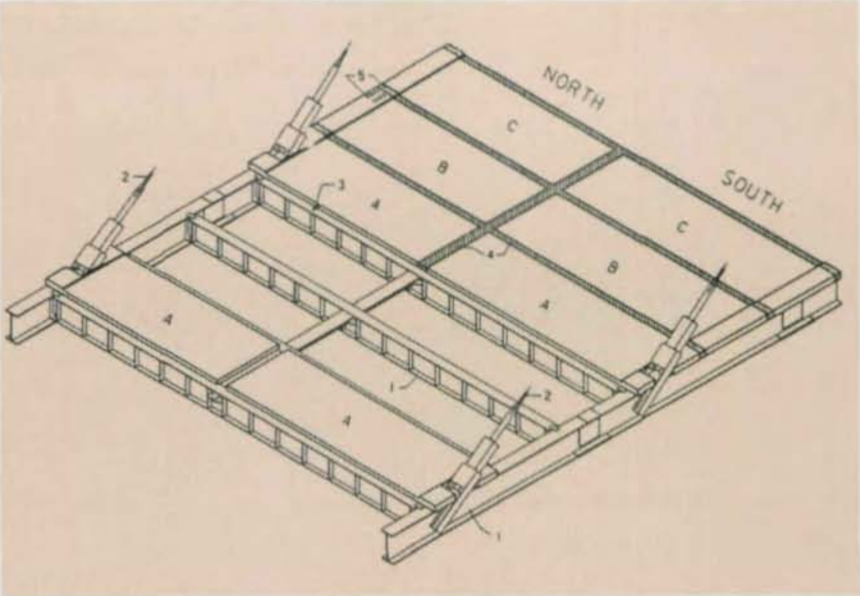
The proposed sequence for the typical free cantilevering of a section consisted of the following major steps (numbers in parentheses refer to items in the figure on the top of the opposite page):

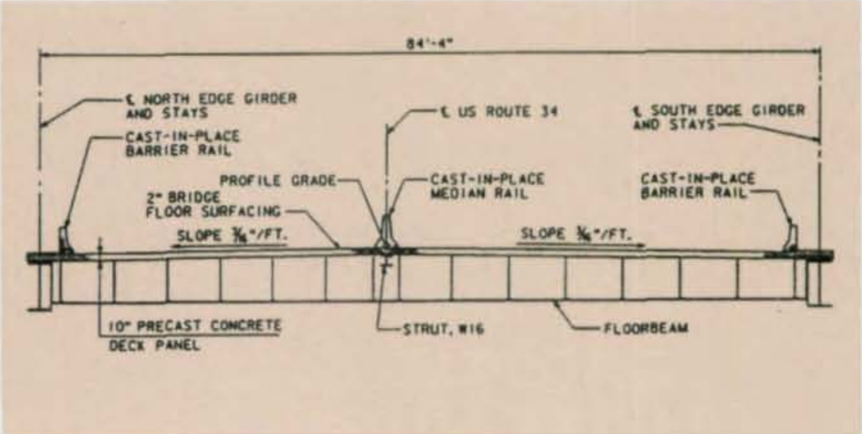
1. Erect edge girders and floorbeams (1), cantilevered from previously completed section.
2. Erect and partially stress cable stays (2).
3. Place north and south precast panels 'A', followed by panels 'B' and 'C'.
4. Use adjusting screws on panels to set panels to the proper elevation.
5. Place longitudinal post-tensioning bars (3) and couple to stressed bars in previously completed section.
6. Place longitudinal and three cast-in-place joints (4). Joints over the edge girders are not placed at this time.

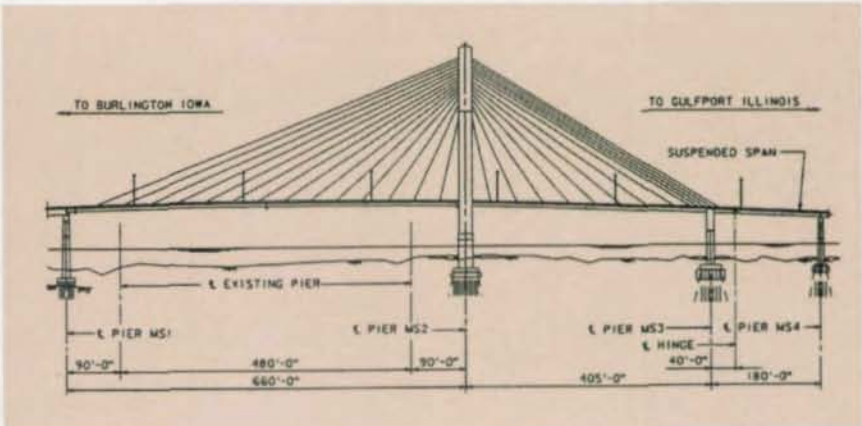
7. Stress the longitudinal post-tensioning bars.
8. Install shear studs on edge girders through stud pockets (5) provided in precast panels. Grout all pockets behind the stay cable just erected.
9. Stress the north and south cable stays to final length.

As noted, the sequences presented in the plans were conceptual and the contractor was required to verify the applicability of the given information to his desired method of construction. The design had been reviewed for general conformance with the concepts but, as is typical for U.S. practice, detailed design and step by step verification of the erection operations were the responsibility of the contractor.

With one exception the contractor elected to follow the general concept as presented. The aerodynamic studies for the bridge during the construction stages were not completed prior to the completion of final design. These studies did indicate a potential for the tower capacity in bending to be exceeded during Stage 2. Based on his analysis of these reports (all aerodynamic studies were furnished to bidders) the contractor decided to place falsework bents in the 405-ft. side span to provide the desired stability. The construction engineer did, of course, do a step by step analysis of the erection to predict both stresses and geometry at all construction stages. The construction engineer also confirmed that the proposed sequence would achieve the specified geometry and, additionally, predicted final member stresses.







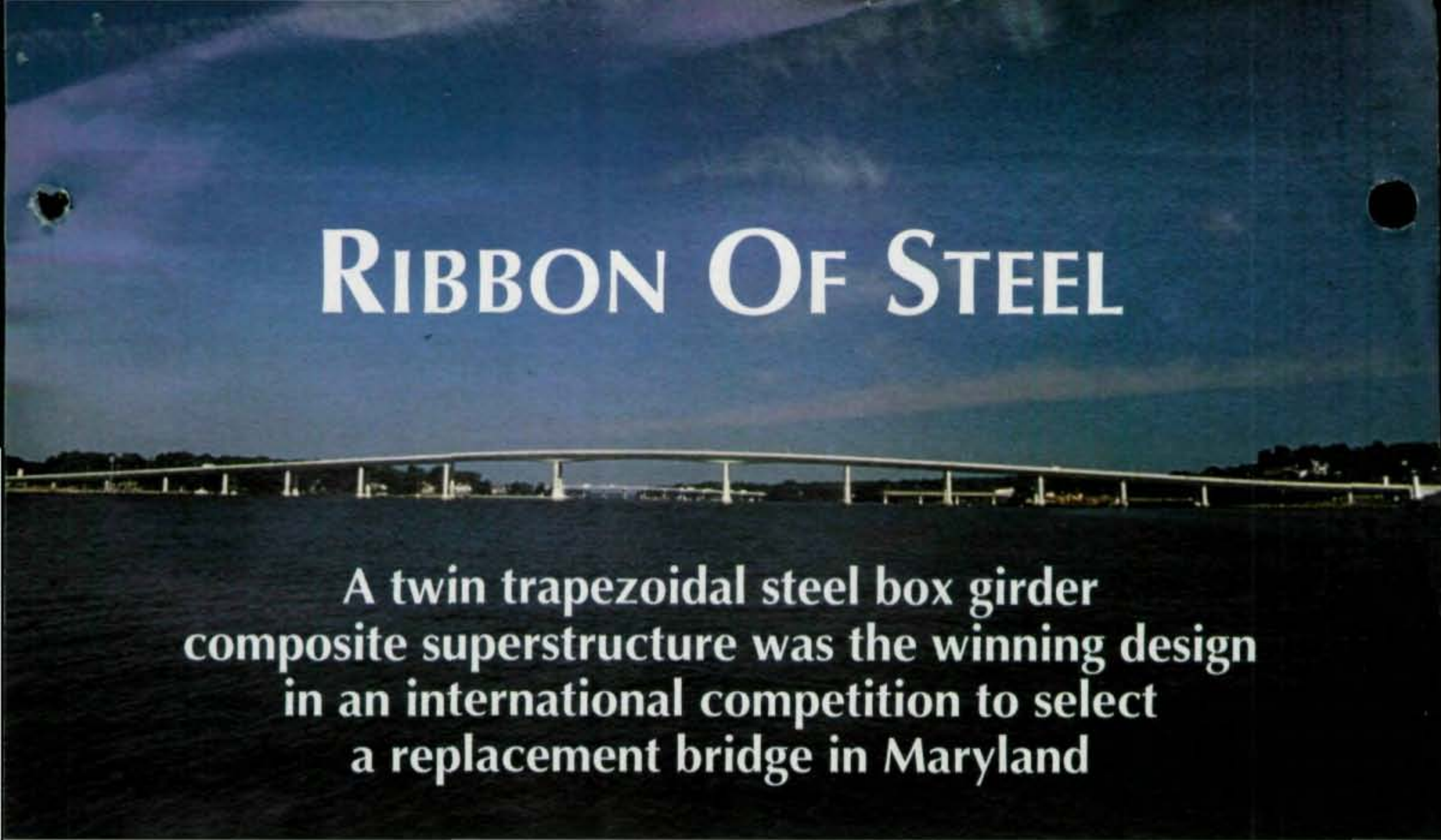
The steel erector chose to erect the steel members (two edge girders and three floorbeams) of a typical section as a complete unit. This had benefits for the schedule, worker safety and maintenance of desired geometry. These elements had been shipped from the fabricator in Des Moines to the erector's facility in Wickliffe, KY, where they were assembled on barges and ultimately returned up the Mississippi to the project site. Fabricator on the project was AISC-member Pitt-Des Moines, Inc. Other team members included: Leonhardt, Andra und Partner (consultant for the cable stayed spans); Wells Engineers (designers of the steel superstructure alternate for Iowa and Illinois approaches); Johnson Brothers (contractor for Illinois approach and substructure for the cable stayed unit); Edward Kraemer & Sons (contractor for superstructure for cable stayed unit, Iowa approach and inter-

change modifications); John F. Beasley (erector of cable stayed unit); Buckland & Taylor (construction engineering for cable stayed unit); VSL Corp. (Cable fabrication); and Raider Precast (concrete deck panel fabrication and fabrication of precast concrete girders for Illinois approach).

The cable stays were fabricated at a facility in Burlington just south of the bridge site. The completed cables were reeled onto spools and delivered to the bridge on barges. For the shorter cables near the tower the cables were uncoiled directly from the barges for installation. For the longer cables the spools were placed directly on the deck. For these cables the unreeling was accomplished through a coordinated operation using a crane to lift the cable end and a reel carrier to simultaneously uncoil the stay as it moved along the deck.

The cable dead end connection was not preassembled to the edge girders. Rather, these connections were erected with the cable stays. The dead end socket was bolted into the connection and the complete assembly was hauled out to the attachment point on the edge girder. A temporary cable deviator was also used at these locations.

Ernst H. Petzold, P.E. is Market Principal—Bridges for Sverdrup Civil, Inc., St. Louis, MO. He was Group Leader for the steel alternate design of the Great River Bridge and also served as Deputy Project Manager.



RIBBON OF STEEL

A twin trapezoidal steel box girder composite superstructure was the winning design in an international competition to select a replacement bridge in Maryland

By Thomas Jenkin, P.E.

AN IMPORTANT SITE, COMBINED WITH KEEN PUBLIC INTEREST, led the State of Maryland to conduct an international competition to select the design for a new bridge to replace an existing crossing of the Severn River at Annapolis, MD. The importance of the site, together with keen public interest in the project justified the unusual design competition. The new bridge, which opened in November 1994, features a twin trapezoidal steel box girder composite superstructure that is fully continuous over 17 spans as it follows a complex curved alignment for more than one-half mile. This "ribbon of steel" has won critical acclaim and the approval of the surrounding communities.

BRIDGE SITE

The bridge spans the Severn River, one of Maryland's most scenic rivers, near it's mouth at the Chesapeake Bay. The crossing serves as the eastern gateway to Annapolis, the capital of Maryland and home of the United States Naval Academy. Annapolis is noted for its fine colonial-era buildings including the Maryland State House which has an octagonal dome visible from the bridge. The narrowing of the Severn River at the bridge site has made this location a road crossing since colonial times. Ferry service originally prevailed until replaced in the nineteenth century by a timber trestle with a swing span. The trestle was replaced in 1924 with a double leaf bascule bridge and arch-shaped approaches. This handsome two lane bridge was a familiar landmark for the surrounding communities; however, automobile and river traffic both grew such that frequent bridge openings became the source of substantial traffic backups. The physical condition of the bridge had also deteriorated such that a major rehabilitation would be required for it's continued service. The Maryland State Highway Administration determined that replacement of the bridge with a fixed crossing was the best course of action to accommodate the future needs of both highway and marine traffic. The prominence and importance of the bridge site as well as the desire to have an outstanding design for the replacement structure provided the stimulus for a design competition.

DESIGN COMPETITION

The design competition was jointly sponsored by the Maryland State Highway Administration and the Maryland Governor's Office of Art and Culture. The competition was conducted in two stages. For the first stage, those interested in participating were invited to submit their qualifications and three to five examples of previous bridge designs. The 21 responses were received from engineering firms throughout the United States and from Western Europe. These submissions were reviewed by a selection committee, and six contestants were chosen to be finalists.

For the second stage of the competition, each finalist prepared a preliminary design including plans, cost estimates, an engineering report which included preliminary calculations, a narrative description of the design concept, a detailed cost estimate, and several renderings of the proposed struc-

Our Tru-Tension® bolt was designed with one very important advantage over other twist-off bolts: a hex head. So removing this bolt or checking the installation with a torque wrench is just as easy as installing it.

Using a quiet, hand-held power tool, our Tru-Tension system always provides the correct tension. It also eliminates operator error during assembly, the bolts come pre-assembled and you can even save money thanks to the quick, easy installation and lower costs for visual inspection.

But, install any button head fastener compared to our hex head fastener, and there's no turning back. Because it's almost impossible to remove a button head fastener, you may even need a welder to burn it off. That costs you time and money. If you'd installed our hex head bolt in the first place you could remove it with a simple, four dollar wrench.

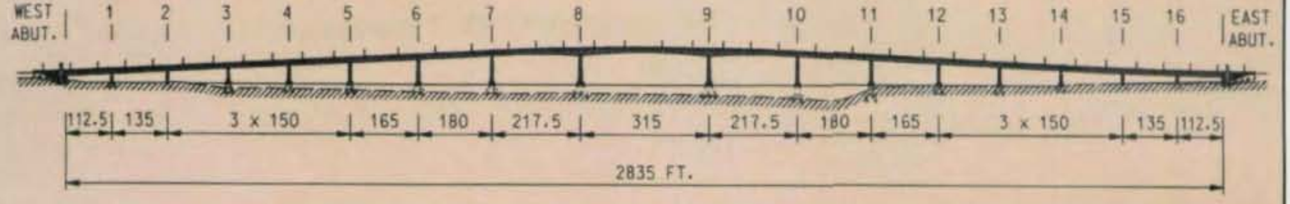
What if you have to verify the proper installation of a button head bolt? Again, it's almost impossible since you can't even get a wrench on the head to test or remove it. But you can verify installation of our hex head bolt with the turn of a wrist and a calibrated torque wrench.

Clearly, this is one bolt that won't let you screw up. Our Tru-Tension system is also fully traceable to our domestic sources, like all Nucor fasteners. They're fully tested and certified, including compliance with FHWA, DOT and AASHTO specifications for bridge construction.

So, forget about cutting bolts off with a torch if something goes wrong. Call us at 800/955-6826, FAX 219/ 337-5394 or write PO Box 6100, St. Joe, IN 46785 to find out more about our Tru-Tension system. And get a fastener that has a good head for business.

NUCOR FASTENER
A Division of Nucor Corporation

Please circle # 38



GENERAL ELEVATION

ture. Five of the six finalists submitted complete design packages in September of 1989. The identity of the designer of a particular submission was kept confidential and not shown on the submitted documents nor revealed to those reviewing and judging the entries.

Each entry was reviewed by two technical committees prior to presentation to the jury. A panel of bridge engineers critiqued the designs for technical merit while a separate group of experienced contractors provided independent construction cost estimates and assessed the constructability of the various proposals. In addition, a series of photo-montage renderings was created for the various entries to provide an objective visual comparison of the competing structures. The five finalists' entries included two concrete arch type bridges, one concrete cable-stayed bridge, and two bridges based on composite steel box girders for the superstructure. The 14 jurors provided a broad range of backgrounds and included practicing design professionals as well as representatives of state agencies, local governments and the surrounding communities. During three days of judging, the jury toured the site and received reports on the findings and opinions of the two technical committees. The jury carefully reviewed the entries and evaluated each relative to the technical, functional, aesthetic and economic goals which were established for the new bridge. The jury, by an "overwhelming majority" vote, then selected the entry submitted by Greiner, Inc., Timonium, MD, as winner of the competition. In addition to selecting the winner, the jury also provided comments on all of the designs as well as recommendations for further refinement of the winning proposal.

Greiner developed their entry in conjunction with Leonhardt, Andra, and Partner of Stuttgart, Germany. The two firms have collaborated on several major bridge projects in the United States and Asia. Greiner was subsequently engaged by the State Highway Administration to prepare preliminary and final designs and contract documents for construction of the new bridge based on their winning proposal.

BRIDGE LAYOUT

The bridge is aligned on large radius curves with spiral transitions. From the west abutment on the Annapolis side, the roadway first curves right and then reverses to a long central curve which connects with the eastern approach. The curved alignment was necessary to maintain traffic during construction and avoid conflicting with an electrical substation located on the western bank of the river. Allowing pedestrians and drivers to view the structure from the approaches, the continuous, large radius curves enhance the visual flow and interesting geometry of the bridge. Spiral transitions, necessary for future construction of a light rail transit line, further smooth the graceful sweep of the structure.

The profile of the bridge allows 75 ft. of vertical clearance over the 140-ft. navigation channel, and the higher level of the approaches and longer spans effectively open the river and allow recreational sailboats new access.

At its eastern end, the bridge widens for a signalized intersection on the approach roadway and crosses the entrance to a park area. Beyond the intersection, the approach rises to a scenic overlook which has long afforded a spectacular view of the river, the Naval Academy, and Annapolis.

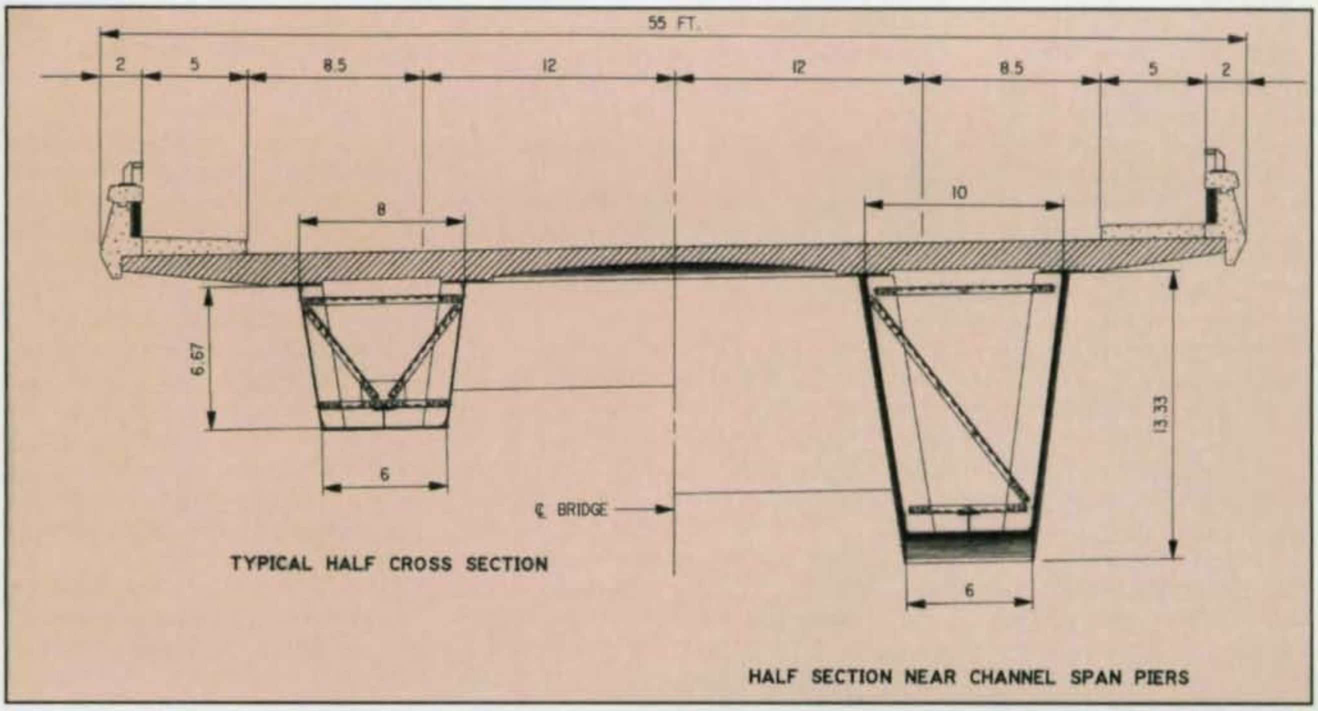
SUPERSTRUCTURE

The superstructure of the bridge is continuous over all seventeen of the structure's spans with a total length of 2,835 ft. between abutment bearings. The span arrangement is symmetrical with respect to the center of the 31- ft.-long navigation span. The approach span lengths vary with the structure's height so that the span-to-height ratio is kept reasonably uniform.

The composite superstructure has a reinforced concrete deck carried on two widely-spaced trapezoidal-shape steel box girders. The deck width is 55-ft. and furnishes two 12-ft. roadways with shoulders, sidewalks, and parapets. Slab thickness varies with a straight taper on the 9-ft.-6-in. cantilever and a parabolic variation over the 20-ft. span between the girders. On the eastern bridge end the deck width widens to 64-ft. As the bridge width increases, each girder stays parallel with the adjacent slab edge keeping a uniform overhang while the span between girders increases to 29-ft. This increased deck span is structurally accommodated by a general thickening of the slab between the boxes.

The bridge parapets include a six-course brick band on their roadway faces, connecting with the colonial-era structures of the city. The upper facia surfaces of the parapets are inclined to increase their brightness in sunlight, and an offset cap creates a shadow line. Low level decorative lighting fixtures at relatively close spacings illuminate the roadway at night and form a string of lights across the sweep of the bridge. Aesthetic under-bridge lights also illuminate the inner surfaces of the piers and box girders and gives the structure a warm glow of reflected light from the river's surface.

Trapezoidal steel box girders were chosen as the main structural elements since this form best satisfied the functional and aesthetic objectives of the design. The lack of exposed bracing and stiffening gives the bridge underside an uncluttered appearance, while the proportions of the boxes strongly express their support of the bridge deck. For most of the structure's length, the girder depth is uniform with sloping webs providing 6-ft.-8-in. depth between the flanges. At the center span the web depth is doubled to 13-ft.-4-in. over the channel piers to carry the longer

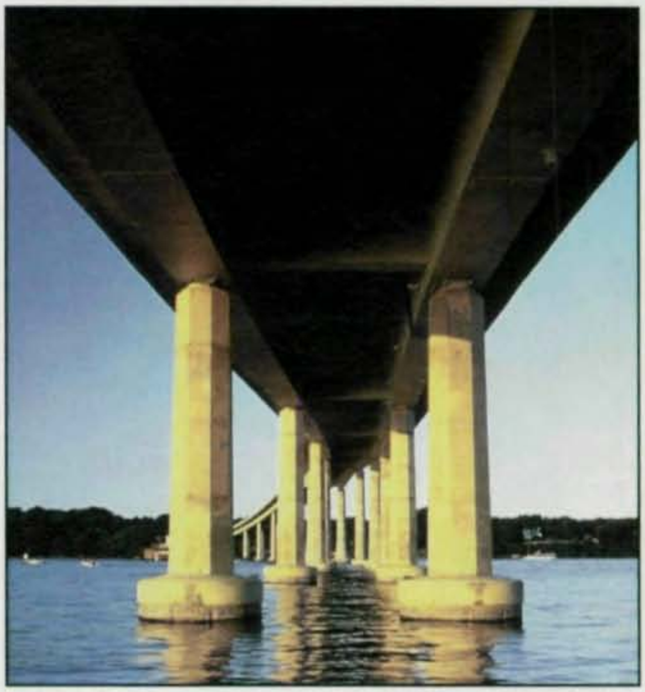


span. The girder's profile in this region has a parabolic variation.

As the girder depth increases, the spacing of the webs at the lower flange and the inclinations of the webs have been kept constant. The spacing of the top flanges thus reflects the parabolic web shape, gradually increasing from 8-ft. to 10-ft. over the main piers. The web planes slightly fold over internal diaphragms at the piers again reflecting the plane changes of the lower flange as it crosses the pier. This shaping of the webs is subtly revealed by a gently rising shadow line from the reduced cantilever overhang. The unusual web geometry gives a well-ordered enlargement of the box girder's proportions in recognition of the increased structural demands.

At each pier, the two main boxes are connected by short transverse composite boxes. These smaller connecting members have vertical webs set at three-quaters of the depth of the main girders. These members provide torsional support since each main girder is placed on a single bearing.

At the abutments, the transverse diaphragms are enlarged and the steel sections completely closed by full width top flanges. These stiff members equalize the end rotations of the two main girders and give bending continuity to the girder ends through a torsional link. By coupling the two girder ends and using a relatively short span length, the redundancy of the end spans is greatly improved.



The box girders were fabricated from ASTM A709 Grade 50 material. The lower flange is stiffened against buckling with a single T-shaped stiffener at its center. Cross frames, positioned near the ends of each girder field section and at intermediate spacings up to 21-ft.-6-in., stiffen the boxes against distortion. At the piers, the girders are stiffened by two internal plate diaphragms aligned with the webs of the transverse connecting boxes. Angles connected directly to the top flanges form an upper lateral bracing system. This bracing closes the box for torsional loads during construction of the deck slab. Temporary cross frames located between the girders near field splices also stabilized the boxes as the deck was built and were later removed.

The superstructure reactions are all carried by multi-rotational bearings with stainless steel spherical segments fixed to the top of the substructure. Longitudinal restraint of the bridge is through the two fixed channel piers. Expansion and contraction of the superstructure between these fixed connections is accommodated by flexing the channel piers. Bearings for the approach piers and above the abutments have horizontal sliding surfaces, transversely restrained, but free to move longitudinally along the local bridge tangent. This bearing layout allows bridge movements without developing significant lateral bearing forces because of the large radii used for the bridge alignment and the relatively flexible lateral elastic support from the columns. The arrangement of the bearings maintains vertical force alignment on the column as the boxes move position. Detailing of the internal stiffening of the box carries the variable load position with a grillage of stiffeners connected with the girder webs and the two transverse internal diaphragms. Modular expansion joints provide 18-inches of movement capacity at each abutment.

SUBSTRUCTURE

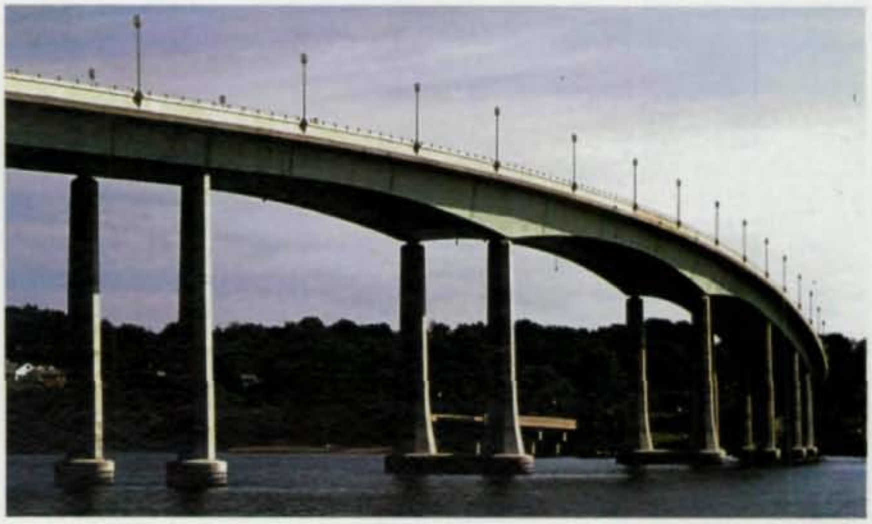
Each pier has two octagonal-shaped tapered columns directly supporting the girders. Transverse structural interconnection at the top of the columns is maintained through the bearings and superstructure. The column bases are supported above the water's surface by concrete pedestals which extend to the pier's foundation at the river bottom. For the main channel piers, the columns have a pronounced lower flair accenting the main span and its arched shape, as well as reflecting the importance of these columns as the fixing points for the structure. The concrete pedestals are granite-faced through the water's surface to enhance durability and compliment the granite used for nearby structures of the Naval Academy. The last two spans at each bridge end are over land, and pedestals are omitted.

Granite-facing is also used at the abutments which have concrete pilasters similar to the pier columns. Two imposing Georgian-style pylons, built of white concrete, flank the roadway at the abutments and create an entrance for the bridge. Bronze medallions with the Maryland state seal are mounted on the pylons. The wingwalls and parapets flair out to terminal posts with a similar style and color as the pylons.

CONSTRUCTION

Bids for construction of the bridge were received in

November, 1991. The project was awarded to AISC-member Cianbro Corporation of Pittsfield, ME, which offered the best price from a field of 13 contractors. Cianbro's bid of $33.4 million included construction of the new bridge and approach roadways, as well as removal of most of the old bridge and conversion of a more recent portion of the old bridge into a recreational fishing pier. The bid price for the structural steel was $8.94 million.



Shop drawings were prepared by AISC Associate-Member Tensor Engineering, Indian Harbour Beach, FL; bearings and expansion joints were supplied by AISC-member D.S. Brown, Inc. of North Baltimore, OH.

Following construction of the piers and abutments, Cianbro erected the 60- to 90-ft. long box girder field sections starting with the three spans in the center of the bridge. Temporary frames attached to the pier columns stabilized the first girder sections as they were centered over the columns. Each 217-ft.-6-in. approach span girder was then completed as two sections were first spliced together and then lifted and connected with the sections over the piers. Once a span was in place, additional field sections were added to extend each box cantilever in the center span. The center sections of the main span were then lifted and connected with one cantilever while supported by temporary hangers from the other cantilever. To close the last connections in the center span,

Cianbro first placed a temporaryload of 115 tons on the nearer approach span, elevating the cantilever and reducing the slope discontinuity between the two sides. The contractor then simultaneously pulled the girder sections together using hydraulic jacks and temporary brackets attached to the web plates. This jacking closed a gap of approximately $1\frac{1}{4}$-in. and aligned the top flange connections, which were then fastened together.

Next, cranes lifted the cantilever sides of the joints until the remaining gaps in the bottom flanges closed and these connections were bolted. The jacks and brackets were then removed and the web splices completed. The cranes releasesd the girders, and the temporary hangers and the temporary loading were subsequently removed. Closing operations resulted in inward deflections of the channel pier columns, which would later be restored to vertical with construction of the deck.

Following main span closure, Cianbro erected the balance of the approaches over the river by lifting pre-spliced sections extending the girders by complete spans with each lift. Over land, steel erection proceeded using smaller equipment and false work towers. The superstructure was completed by sequenced placement of the deck working from the center of the main span towards the bridge ends

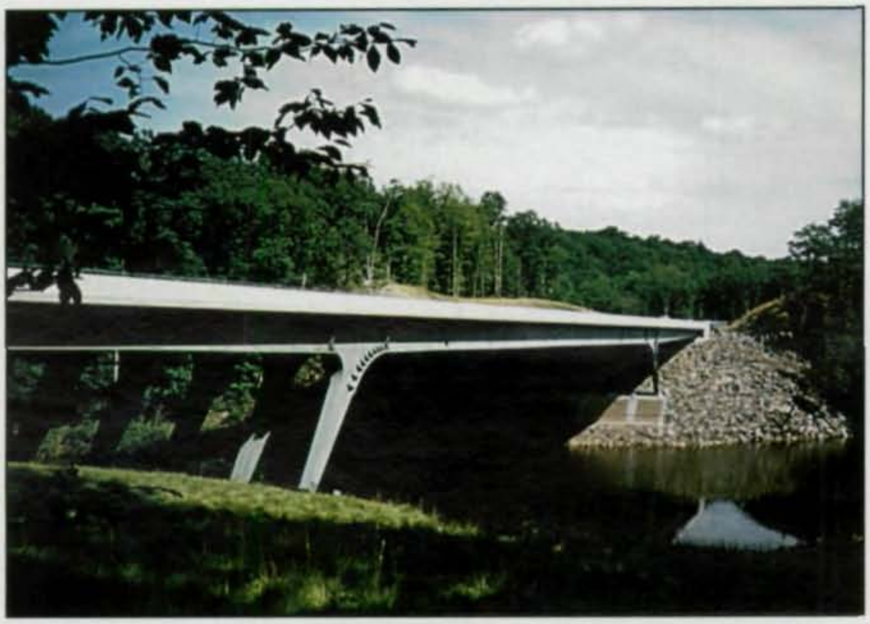
At the west end of the bridge, the two land piers and three spans they supported were constructed in stages to maintain traffic. Temporary supports at the piers and abutment provided torsional support for one box as its share of the deck was built and opened to two lanes of traffic. After removing conflicting portions of the old bridge, the pier foundations were widened, the missing columns constructed, and the second box completed. This staged construction demonstrated the techniques proposed for future redecking.

The new bridge was officially opened in November, 1994, and named the United States Naval Academy Bridge in honor of the venerable institution which has stood for 150 years beside the crossing on the western bank of the Severn River.

Thomas Jenkins is vice president and chief bridge engineer with Greiner, Inc., Timonium, MD.

SLOPING LEGS KEEP BRIDGE PIERS ABOVE WATER

Since the Occoquan Reservoir is a primary source of drinking water, it was essential for quick approval that a new bridge's piers be kept out of the water



By John D. Anderson, P.E & David A. Charters, Jr., P.E

LOCATED APPROXIMATELY 25 MILES SOUTHWEST OF WASHINGTON, DC, the population of Prince William County (PWC), VA, has tripled to 235,000 people in the past 20 years. Unfortunately, though, prior to this year there were only two east-west connector roads in the county and both were winding two-lane roads, with alignments following old colonial routes. Traffic on these roads slowed to a crawl for three hours each day during the morning and evening peak rush hour periods, and they became very dangerous. There have been as many as nine fatalities along one 7-mile stretch of road during a single 10-month period.

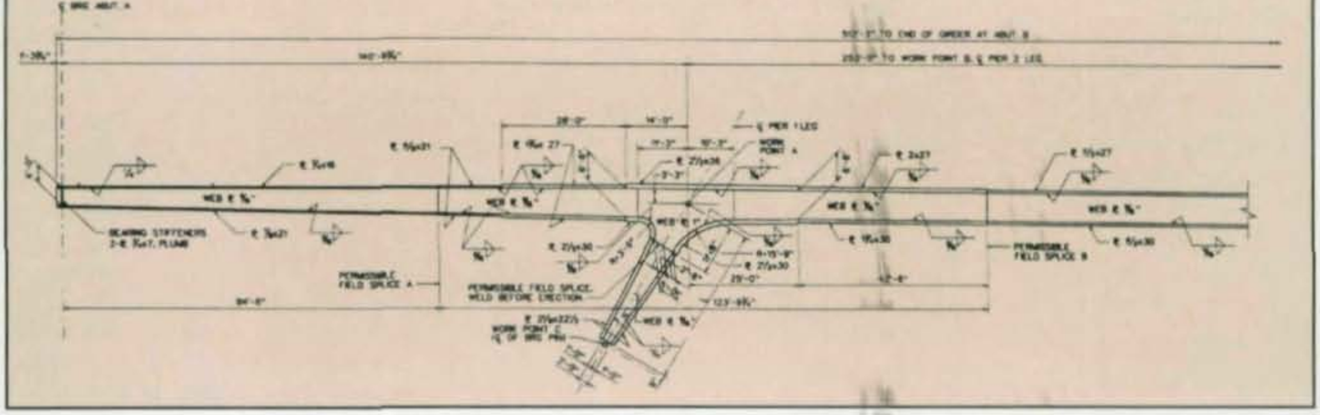
The Prince William Parkway (PWP) was conceived by PWC to connect 1-95 and U.S. Route 1 in Woodbridge to 166 in Manassas, a distance of approximately 18 miles. The PWP would help relieve traffic congestion on Davis Ford Road.

PWC retained Parsons Brinckerhoff (PB) to design an 8.5-mile section of the PWP on a new alignment through undeveloped, wooded terrain, and to administer its construction. The Prince William Parkway Bridge over the Occoquan Reservoir in Manassas, VA, is approximately at the center of this section.

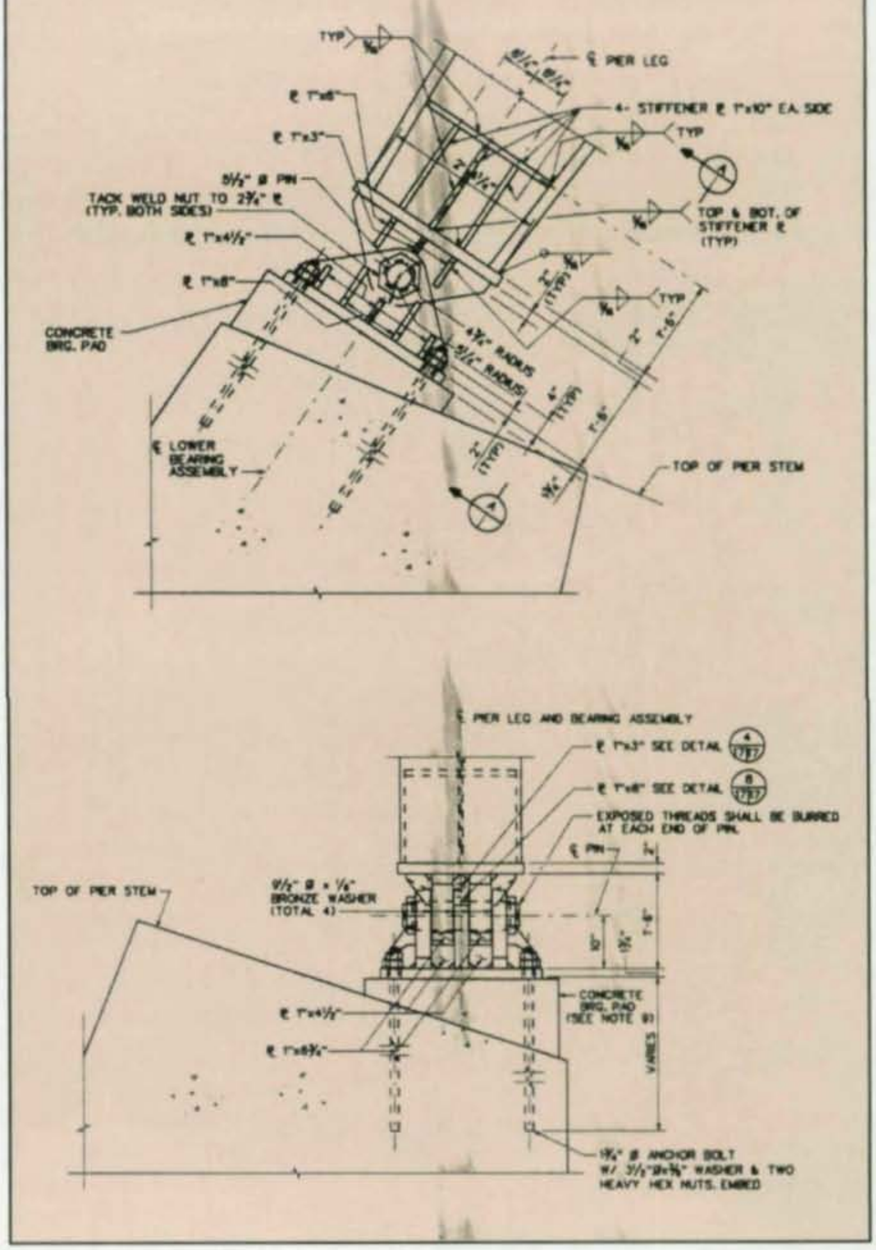
BRIDGE DESIGN

The Occoquan Reservoir is a source of drinking water for

Partial Steel Frame Elevation



Bearing Details At Piers
(Section thru pier along centerline of frame)



Fairfax County. The design objective for the bridge given the specifics of the project site was to provide a long center span, thereby keeping the piers out of the reservoir except during periods of very heavy flooding. The sloping steel pier leg configuration implemented by project designers was very efficient in clearing the waterway and, at the same time, limiting the length of the center span. It was critical to avoid building a pier in the reservoir, because obtaining the necessary permits would have significantly delayed the project.

Opened to traffic on December 8, 1994, the river crossing actually comprises twin bridges, one serving eastbound and the other westbound traffic, separated by an expansion joint. Each bridge is a 513-ft.-long, three-span continuous, steel rigid-frame structure, with a 253-ft.-long center span and side spans of 142 ft., 6 in. and 117 ft., 6 in. Transverse girder spacing is 9-ft., 2-in. The deck is about 50 ft. above the mean water level and it crosses the reservoir at a skew angle of 40 degrees.

The bridge substructure consists of cast-in-place reinforced concrete drilled shafts under the west abutment and pier, and direct rock-bearing shallow foundations for the east abutment and pier. The drilled shafts are 42 inches in diameter and extend as far as 41 ft. below the bottom of the footings.

The two massive concrete piers are battered and skewed and have a "fractured fin" architectural treatment on the side facing the river. They are 150 ft. long, 11 ft. wide at the base of the wall, and 25 ft. high from the bottom of the footing to the bearings.

The superstructure includes 13 three-span continuous steel frames. Each frame consists of the two sloping steel pier legs field-welded to the steel plate girders. The depth of the plate girders varies from 4 ft.-2 in. at the abutments to a maximum of 6 ft.-11 in. over the pier legs). Expansion joints were located at the abutments.

The cast-in-place concrete deck is approximately 115-ft. wide, constructed to accommodate six lanes of divided traffic with 10-ft. shoulders and a fenced bike path adjacent to the westbound lanes.

The bridge was designed to AASHTO's Standard Specifications, including VDOT's modifications as appropriate. VDOT's Road and Bridge Standards were also used.

Substructure

Due to a high degree of differential weathering, fracturing, and mud seams in the rock subsurface, the abutment and pier on the west side of the reservoir were founded on 42-in.-diameter drilled shafts. The more competent rock on the east side of the reservoir permitted the use of shallow foundations for that pier and abutment.

A hydraulic analysis was performed to confirm foundation depths to ensure that the bridge would not be susceptible to scour during a flood. This analysis was used to determine the gradation and limits of the 48-in.-thick riprap layer installed around the piers.

The top of the rock at the west pier slows downward from the south end of the pier to its north end. The top of rock was as much as 4 feet below the bottom of the footing for the northernmost 50 feet of the 150-ft.-long pier. To confirm the safety of the pier, PB performed a structural analysis of the pier, modelling the upper 4 ft. of the northernmost drilled shafts as if they were exposed during a severe flood.

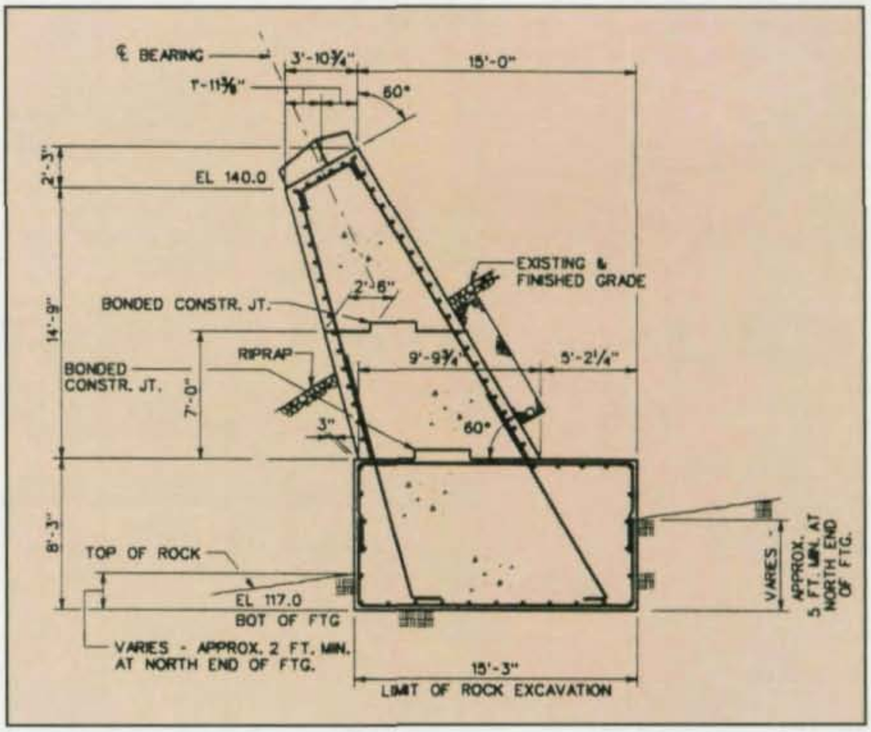
Special precautions were taken to guard against heat of hydration problems, notably shrinkage cracks, at the massive concrete piers. Provisions were added to the specifications to establish design criteria for the concrete mix.

The size of the concrete pours for the pier stems was limited and the contractor was not permitted to use high early strength cement, calcium chloride, or accelerating admixtures. An attempt was made to limit those items that could add heat to the concrete mix. The contractor was required to submit documentation showing that the concrete design mix and placement methods were in accordance with the special provisions and referenced ACE documents. The contractor chose to use cold waterline and granulated iron blast-furnace slag (to replace half of the cement content) in the mix to satisfy project specifications.

All of these efforts to eliminate excessive shrinkage cracks due to uncontrolled heat of hydration were quite successful. PB's field inspectors remarked at the noticeable lack of these commonplace shrinkage cracks.

Special pin bearings were designed for the pier locations to allow the sloping steel pier legs to rotate as required. The 5.5-in.-diameter pins were made of stainless steel in accordance with ASTM AS76, UNS Designation S28200, with a yield strength of 60,000 psi. To protect the pin bearings from deterioration from the reservoir, the top of the pier concrete was set above the 100-year flood level.

During a routine ultrasonic inspection of the pins by VDOT, the inspector noticed some unusual reflections showing up on the ultrasonic equipment. After further investigation, project engineers concluded that the configuration of the vertical bearing plates with respect to the pin caused the reflections, not a Jack in the pin. VDOT concurred with PB's assessment. To document that there is no structural problem in the pins, a baseline ultrasonic test was performed on all pins in May 1995 and will be repeated next year for comparison.



SUPERSTRUCTURE

The material for the rigid-frame steel members is ASTM A709, Grade 50. A finite element computer analysis was performed at the intersection of the main girders with the top of the sloping steel pier legs to confirm the transfer of load through this structure intersection. The bridge utilized 1,700 tons of steel (58 lbs./sq. ft. of deck).

Project specifications required the steel to be painted with a zinc rich paint system. As an alternative to the conventional three-coat system, VDOT per-

mitted the use of a shop-coated single coat paint system, which avoided the costs of field painting over the reservoir. All faying surfaces, as well as the top flange of the girder, were painted and galvanized bolts were used. The shop application of the paint was carefully monitored by inspectors from PB, VDOT, the fabricator, and the paint manufacturer. The paint was Carbo Zinc 11 HS by Carboline.

Careful erection procedures are required when single-coat paint systems are used. These procedures help minimize the amount of field touch-up work that is required after erection, which improves the overall aesthetics of the bridge.

Environmental Issues

Aside from the special precautions to avoid field painting over the river, the following restrictions were included in the contract documents to protect the *reservoirs natural environment:*

- The Nationwide Permit from the Corps of Engineers restricted any fill placed in the reservoir to a maximum of 200 cubic yards.
- No piers were allowed to be constructed in the waterway at its normal elevation.
- Booms were placed in the river to restrict the spread of any fuel spilling from the construction equipment.
- The downspouts from the deck drainage inlets were not permitted to discharge directly into the reservoir;

aside from the deicing salt present in the runoff, an attempt was made to keep any fuel spill on the bridge deck from entering the reservoir.

CONSTRUCTION ISSUES

Shirley Contracting Corporation (SCC) won the bridge contract with a bid of $5.2 million, well below the engineer's estimate of $6.7 million. The very competitive construction environment at the time of the bidding period contributed to the contractor's low bid.

AISC-member Atlas Machine & Iron Works fabricated the structural steel. The geometry of the sloping legs and the curved web haunch sections at the top of the pier legs was very complex. No two frame sections were alike since all 26 pier bearing pins were at the same elevation. Adjustments were made in the structural steel to provide a cross slope in the deck surface.

McKinney Drilling Company constructed the drilled shafts. They used mud augers and rock bits to drill the shafts, with casings to prevent material from falling off the walls of the shafts.

Williams Steel erected the structural steel. Iron workers used 21,000 high-strength bolts to connect 65 separate sections of the plate girders. Five girder sections form each of the 13 rigid steel frames. In each frame, two girder sections have field-welded pier legs attached.

After pier leg attachment, the 26 bridge frame sections supported by the piers were erected. The rocky bank on the east side of the river was too steep to use to transport girders or a crane down to the pier. Williams built a frame using wide-flange steel members and attached it to a tri-axle dolly to support the pier/leg girder sections as they were moved down the west slope to their barge-mounted crane. They used two "tugger" winches to move the barge across the reservoir, pulling on cables anchored at four points on the shore, two on each bank. Before lifting the girder sections above the piers, the barge was anchored by dropping two Spuds" through well holes at opposite corners of the barge. Their weight drove them into the mud beneath the barge, preventing lateral movement.

With the leg and the bearing attached, the heaviest girder section stands 33 ft.-high, is 124 ft.-long, and weighs 85,000 pounds. The middle section of five sections that form each frame is the longest of the girder sections, measuring 135 ft.-long. Williams' largest crane was a 230-ton Manitowoc, rigged with a 140-foot boom and a 30-foot jib. They Walked" it on a sectional barge to erect many of the heaviest girder sections. The pin bearings at the piers were attached to the pier legs at the fabrication shop and remained attached during field-welding and erection.

Field welding was used to attach the sloping steel pier legs to the plate girders. This connection method was chosen instead of bolted splice connections due to the proximity of the curved flange plates to the splice location and the improved aesthetics from using field welds.

In the fabrication shop, the webs and flanges of the adjoining members were beveled to prepare them for welding. The fabricator carefully aligned the pier legs and the girders at the proper orientation in the shop. For field alignment purposes, several punch marks were placed on the webs of the pier legs and girders. One punch mark was placed at the structure's work point above the pier leg. Additional punch marks were placed on each side of the intended field weld location. The fabricator then measured and recorded the distances from the web punches to the center of the bearing pin. Upon arrival in the field, the welding crew re-established the proper alignment of the pier legs and girders using the punch marks and measurements furnished by the fabricator.

The contractor performed the field welding by setting the pier legs and girder webs horizontally on steel support stands. All welding was performed in accordance with BDOT specifications and the AASHTO/AWS Bridge Welding Code. Welders worked on the flanges first, alternating passes on the top and bottom flanges to minimize creep due to temperature changes. The flanges were welded in a vertical direction (upwards) using a flux cored welding process with a Lincoln E71-T1 electrode, 0.045-in.-diameter wire and carbon dioxide gas shielding.

The contractor began using a flux core process to weld the webs also, but found this to be too time consuming due to the length of the welds, so switched to a submerged arc process. Welding in the horizontal position with a steel back-up bar, the welders used a Lincoln L61 electrode and $^{3}/_{32}$-in.-diameter wire. Full penetration of the weld was achieved from one side of the web. After welding the top of each web, the welders removed the back-up bars and ground off the weld reinforcements. All welds were ground flush and 100% radiograph inspected to check for voids and imperfections.

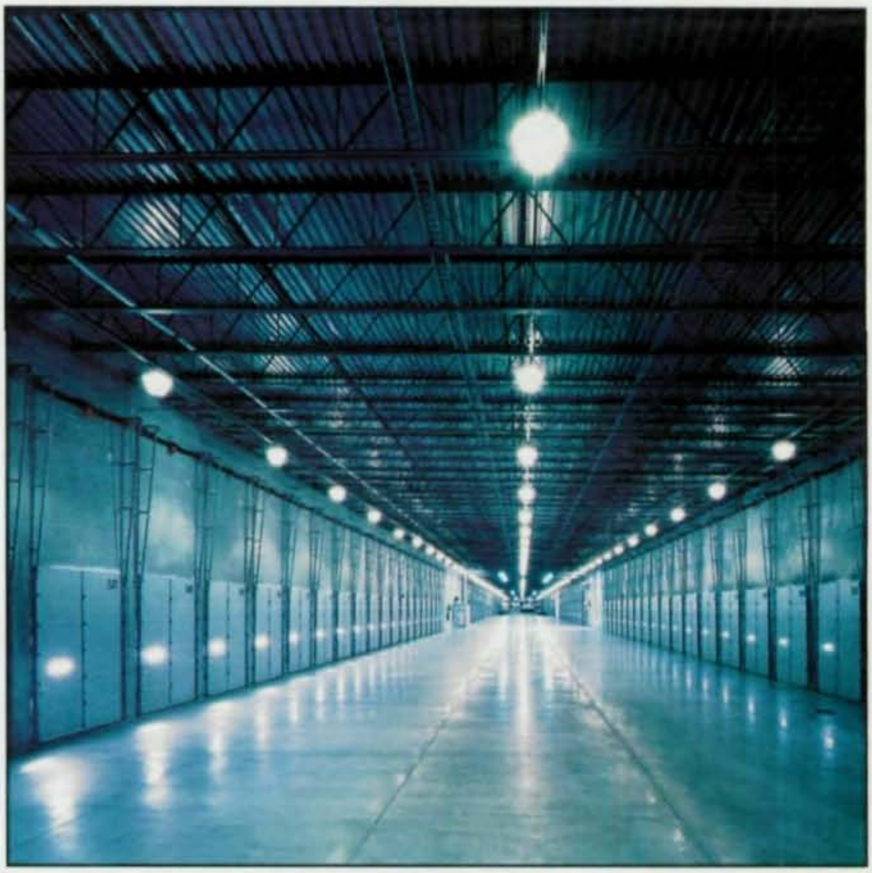
John D. Anderson, P.E., is a resident engineer with Parsons Brinckerhoff Construction Services, Inc., and David A. Charters, Jr., P.E., is a project structural engineer with Parsons Brinckerhoff Quade & Douglas, Inc.

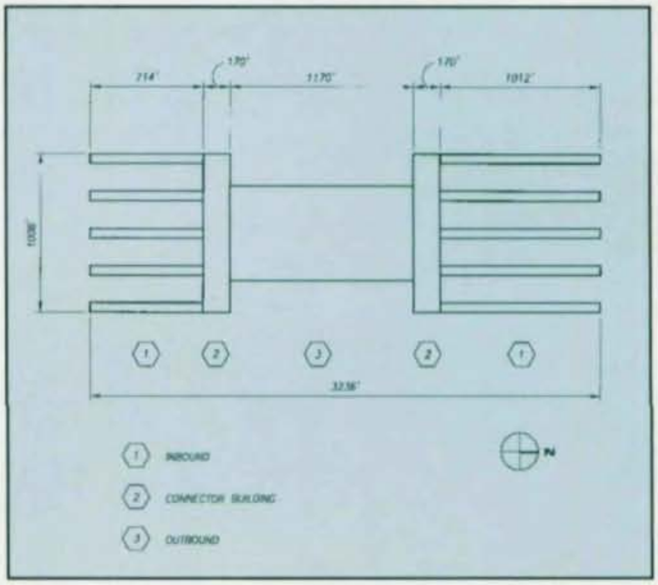
 Please circle # 23

the outbound modules. Each of the connector elements measures 170-ft. by 1,008-ft., with a clear height requirement varying from 30-ft. along the inbound side to a minimum of 21 ft. at the transition to each outbound module.

The outbound modules are long narrow structures allowing dock doors at 12-ft. on center along both long sides. The areas between the parallel fingers are used for vehicle maneuvering. All 10 outbound modules are 60-ft. wide; the five north fingers are 1,012-ft. long, while the five south fingers are 714-ft. long. The interior of each is column-free with a minimum interior clear height of 21 ft.

In addition to the main building, there are several auxiliary facilities, such as truck wash buildings, a truck maintenance shop and guard houses.





The 1.9-million-sq.-ft. Chicago Area Consolidation Hub houses a sortation facility that will be the principal hub for all of UPS's east-west traffic.

STRUCTURAL DESIGN

The design of the structure is of interest both for its process and its product. The process began with UPS's design of the external flow of vehicles (inbound and outbound) and the internal flow of packages (conveying and sorting). In plan, the building shape is that of a block I with narrow finger structures extending from the top and bottom of the block I. Using wide flange beam terminology, the "web" of the I contains the inbound receiving dock doors and sorting equipment, while the "flanges" contain conveyors connecting the inbound areas and outbound areas. Lastly, the outbound modules contain conveyors leading to the outbound dock doors. Each of these three functional areas was studied individually and the structural systems used are unique to each area.

The building's shape also was heavily influenced by the internal flows. The layout and vertical arrangement of chutes and conveyors largely dictated the locations of columns and their possible size. The structure contains 866 columns, including wind columns. UPS's equipment needs dictated that the east-west grid lines jogged 10 ft. and 16 ft., respectively, on the east and west sides. Only the first two columns aligned as pairs on each side. The center section of the building was shifted. Also, the bay modules in both the north-south and east-west grids changed to suit the equipment layout.

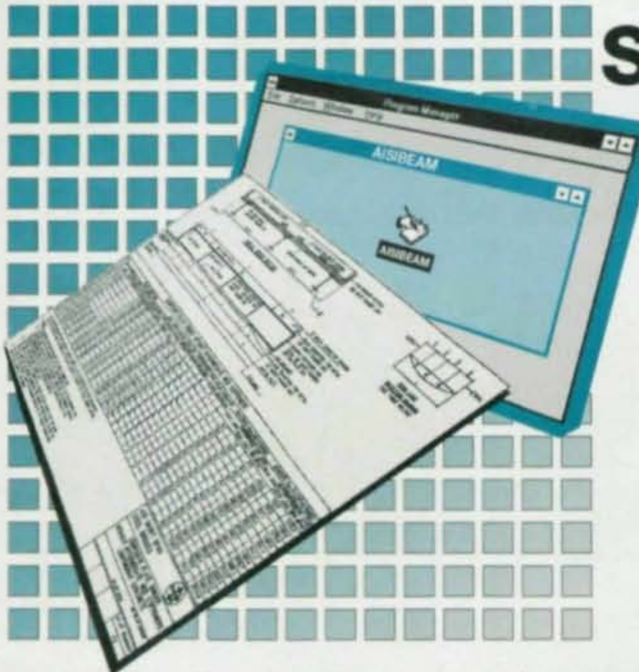
After the process layout was established, UPS began a series of studies to determine a narrow range of feasible structural systems and enclosure systems. Of foremost importance for the structural system were vertical and horizontal clearances—both of which were previously established by UPS's needs. These studies were done by UPS's engi-

WITH VULCRAFT ON DECK, THIS.

The Ballpark at Arlington, home of the Texas Rangers, built with Vulcraft composite deck.

261

New Plans and Software for Short-Span Steel Bridges



With AISI's new comprehensive short-span steel bridge package designers can minimize design time and project cost. Quickly explore design and detail options for:

- Welded plate girders with unstiffened and partially stiffened webs
- Rolled beams with and without cover plates for composite and non-composite construction
- Normal and lightweight concrete deck
- Elastomeric bearings
- Jointless and conventional abutment information
- Crash tested Jersey barrier
- Tables of reactions, moments and deflections
- Table of quantities for each design (structural steel, rebar and concrete)
- AASHTO's Load Factor Design
- HS25 Live Load
- Over 150 11" x 17" design and detail sheets
- Much more!!!

This two-volume Short-Span Steel Bridge Package contains more than 1,100 predesigned steel superstructures with details, plus user friendly, PC compatible software, that will bring you quick, efficient solutions to many simple-span non-skewed bridges.

The plans tabulate the design of bridges that are 20 feet to 120 feet long in five foot increments for 24, 28, 34, 40 and 44 foot roadway widths. The software provides back-up calculations to the pre-designs and allows the engineer to custom design, code check or rate the bridge.

The package was developed by Dr. S. Taavoni, PE, of Kennedy Porter & Associates for the American Iron and Steel Institute in collaboration with AISC Marketing, Inc. Sponsors of the package include Bethlehem Steel Corporation, Inland Steel Industries, Lukens Steel Co., and U. S. Steel.

Please send me:

❑ Information about the AISI Short-Span Steel Bridge Package.

❑ A copy of the AISI Short-Span Steel Bridge Package (payment enclosed).

Ship Short-Span Bridge Package to:

Name ______________________

Company ______________________

Address ______________________

City ____________ State ______ ZIP ______

Phone ____________ Fax ____________

FAX: 202-463-6573 **or**

MAIL: American Iron and Steel Institute
1101 17th St., NW
Washington, D. C. 20036

TEL: 202-452-7140

Steel the recycled material

<u>Order Form</u>

Short-Span Bridge Package___@$150 = $ ____

Additional shipping and handling will be added for orders outside the U. S.

❑ Check (payable to AISI) ❑ Visa ❑ Mastercard

Cardholder Name ______________________

Card Number ______________________

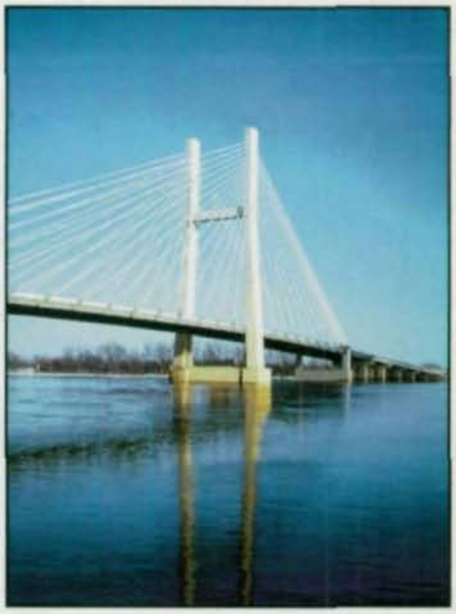
Expiration Date ______________________

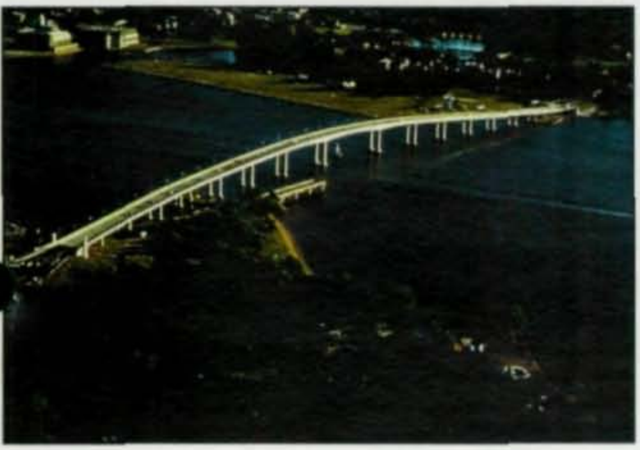
Signature ______________________

Please circle # 15

Call For Papers

NATIONAL STEEL BRIDGE SYMPOSIUM

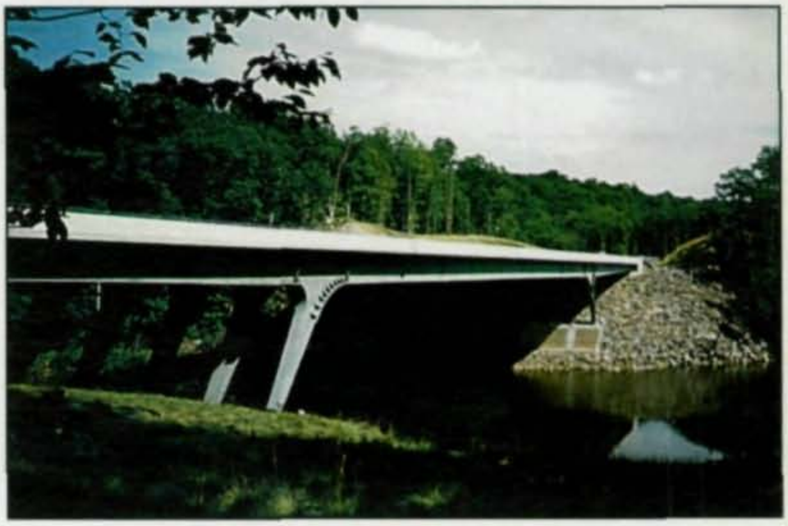




Wanted: Innovative Steel Bridges

If you have recently designed, built, fabricated, erected or painted an innovative steel bridge—or done anything innovative relative to steel bridges—and wish to present a paper at the Fifth National Steel Bridge Symposium, send a short summary or abstract to Fred Beckmann, AISC Director of Bridges, American Institute of Steel Construction, Inc., One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001(fax: 312/670-5403) by December 15, 1995.

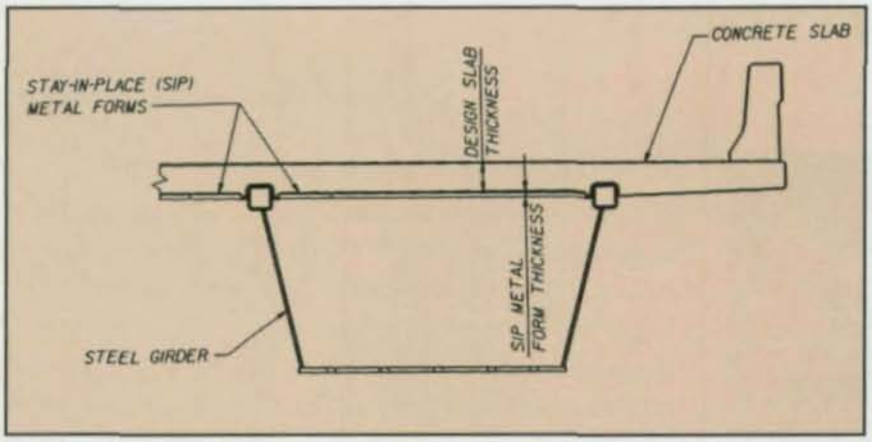
The **NSBA National Steel Bridge Symposium**, to be held in Chicago in the Fall of 1996, brings together design and engineering professionals, FHWA and Department of Transportation Officials, fabricators, erectors and contractors to discuss and learn about state-of-the-art bridge design, fabrication and construction techniques. The Symposium includes workshops, lectures, technical sessions and the presentation of the AISC Prize Bridge Awards.



Setting Standards For Over 70 Years

THE PRINCIPAL OF PRESTRESSING IS CLEAR-CUT: PRIOR TO THE APPLICATION OF LOAD, A SET OF STRESSES IS INDUCED OF SUCH DEGREE THAT THE TOTAL OF THESE STRESSES AND THE STRESSES PRODUCED BY THE DIRECT DEAD AND LIVE LOADS WILL NOT EXCEED CERTAIN LIMIT STATES

— *USS BRIDGE REPORT, JUNE 1985*



POST-TENSIONED BOX GIRDERS

The use of post-tensioned steel box girders on three bridges in Florida is expected to improve aesthetics without increasing costs while also enhancing long-term performance characteristics

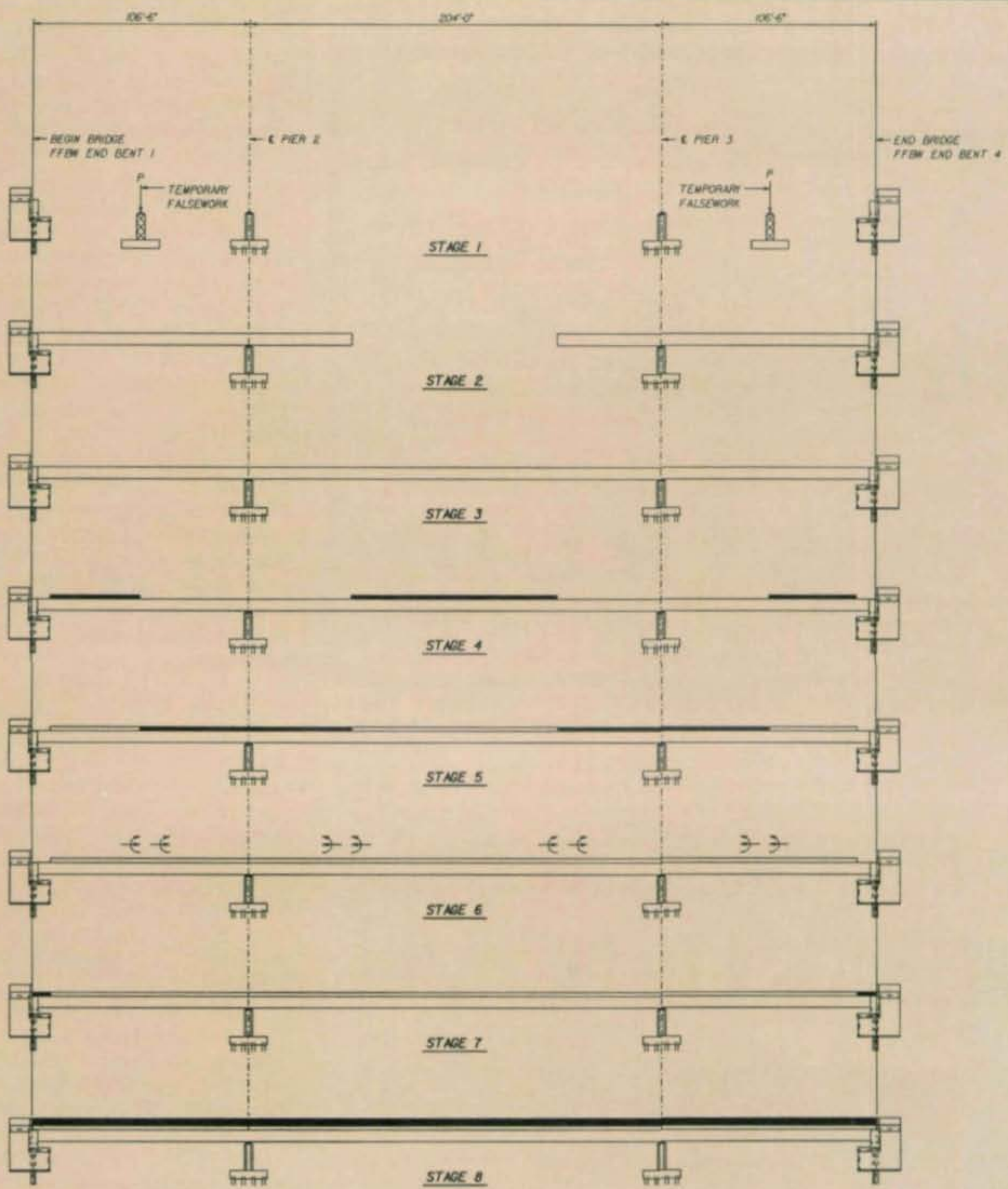
By Robert B. Anderson

WHILE PRESTRESSING IS OFTEN USED ON CONCRETE STRUCTURES, ITS ADVANTAGES ARE RARELY EXPLOITED ON STEEL STRUCTURES even though the technical advantages of prestressing for steel structures—better performance at a lower cost—have been discussed in U.S. and Canadian literature since the mid-1950s. Unfortunately, the complexity of designing prestressed steel structures has undoubtedly stymied its use.

Now, however, Greiner, Inc., under the direction of the FHWA and the Florida DOT, is designing three innovative bridges utilizing post-tensioned steel box girders. The box girders, thought to be the first of their type, incorporate two-stage longitudinal post-tensioning of the steel box and the steel/concrete composite section. The forerunner to this scheme was the Bonner's Ferry Bridge in Idaho (see sidebar) that incorporated post-tensioning of plate girders.

The two-stage prestressing technique is to be used for three identical bridges designed as part of the improvements to State Road No. 55/U.S. 19 in Clearwater, FL. The highway is being upgraded from general lane service to limited access in a highly congested urban setting and the bridges form an integral part of an "urban" interchange.

The superstructure spans separate turnarounds for the frontage roads and the crossroad intersects beneath the main span. The bridges are three-span continuous with side spans of 106-ft.-6-in. and center span lengths of 204-ft. Six girders spaced 19-ft.-5-in. on center are used to support the roadway, which measures 117-ft.-1in. wide. Three piers, trapezoidal in shape and located within a median area, each carry two girders at the intermediate supports. The bridge length is defined by the end bents. These structural elements are integrated with a proprietary retaining wall system that is configured to form



Stage 1: Construct bridge superstructure including end bents 1 & 4, piers 2 & 3, and retaining walls adjacent to end bent caps. Erect temporary falsework.

Stage 2: After superstructure concrete has obtained a design strength of 3400 psi, erect girder side spans and cantilever portion of main span, also erect end diaphragms, temporary bracing between the girders and cross frames diaphragms at the piers. Dismantle temporary falsework.

Stage 3: Erect center of main span.

Stage 4: Cast deck placement 1A, 1B & 2 including anchorage blisters.

Stage 5: Cast deck placement 3A & 3B including anchorage blisters.

Stage 6: After slab concrete has reached a strength of 4000 psi, remove all temporary cross frames, stress and grout second stage post-tensioning.

Stage 7: Cast final deck placement at bridge end. Cast anchorage head covers.

Stage 8: Place barriers.

the approach embankment. Final design plans have been completed for the three bridges and have been submitted to the FDOT with contract letting expected sometime in 1996.

The selection of post-tensioning for the steel box was based primarily on cost with some consideration of bridge aesthetics. As with all prestressing schemes, the prestressing concept is economically feasible only if the cost reduction for the main section is more than the additional costs of the prestressing material and its installation. To establish the economic feasibility of the prestressing scheme, 12 bridge alternatives were evaluated, including various arrangements of steel plate girders, steel box girders, AASHTO beams, and cast-in-place concrete box girders, along with the post-tensioned box girder option. The post-tensioned box girder scheme was approximately 8% more expensive than the least costly conventional plate girder alternative.

Post-tensioning offered a cost savings of roughly 5% when compared to the conventional steel box girder. None of the concrete alternatives were competitive for this application. The post-tensioned steel box made economical sense, though, not only because it offered savings in steel weight, but also because it decreased structural depth, which in turn lowered the approach roadways and retaining walls and lessened right-of-way acquisition costs. All but the right-of-way costs were considered in the preceding cost studies. The post-tensioned steel box was recommended since it proved to be nearly the same overall cost as other schemes and offered improved aesthetics when compared with plate girders.

Because the concept could be applied to three identical bridges, the development cost that might be required for the application of a "new" technology could be shared by multiple structures. In this regard, the plans were submitted to two fabricators and AISC Marketing, Inc. for their review during design development. Each suggested ways to improve the design.

PRESTRESSED/NON-PRESTRESSED

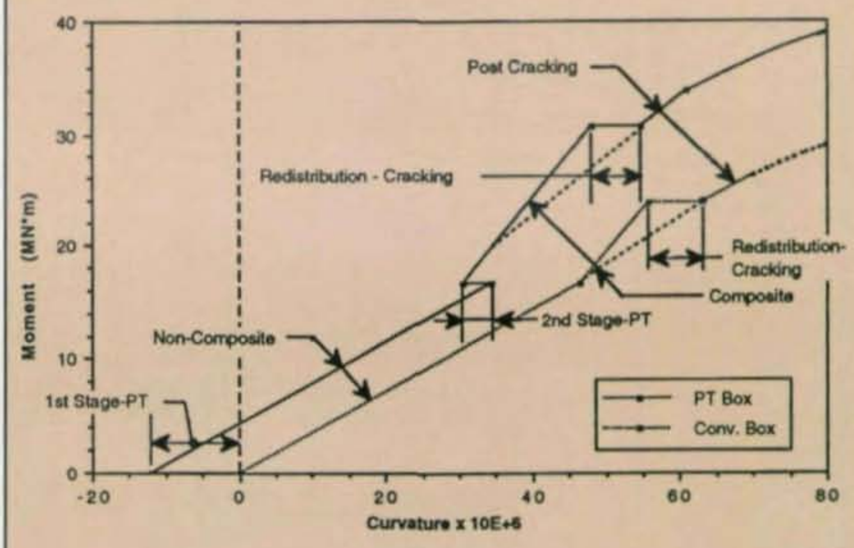
A conventional, non-prestressed design was used in the positive moment regions of the box girder. In the negative moment regions, the girder top flange was replaced by a A500-Grade C tube section with a yield strength of 50 ksi. Running through each tube are 25 1/2-in.-diameter prestressing strands. After application of post-tensioning, the tubes are grouted.

The plates conform to ASTM 572-Gr. 50. The second stage post-tensioning was supplied by eight tendons per girder, each with seven $^1/_2$-in.-diameter strands. All post-tensioning strands are low relaxation type with a tensile stress of 270 ksi, conforming to ASTM 416-85. The composite slab is constructed of 5,000 psi cylinder strength concrete. The web depth of 56-in. gives a resulting span-to-depth ratio of 28. (The span is measured between dead load points of contraflexure without superimposed prestressing loads.)

The bridges are designed to meet AASHTO Specifications (14th edition-1989 and Interim Specifications through 1991) using load factor design with a design live load of HS20 truck loading.

DESIGN CONSIDERATIONS

The construction sequence was set up to minimize the tensile stress in the slab. While it was desirable to engage the composite section as early as possible in the construction sequence to relieve the steel stress, the drawback is that more post-tensioning must be added to overcome slab tension. The most feasible alternative was to make the slab composite over the pier only for superimposed dead loads and



Comparison Of Moment Curvature Behavior

live loads.

Early in the project it was recognized that creep and shrinkage of the concrete slab over time would be a critical element in the design of the post-tensioned steel box. A preliminary study was done using an age-adjusted concrete modulus for a simplified composite section. The goal of this study was to develop an understanding of the load transfer mechanisms that occur with creep and shrinkage. As concrete creeps or shrinks over time, the change in concrete strain will result in both axial force and resulting moment, due to eccentricity of the axial force being redistributed within the section. Under a sustained load, the steel stresses will generally increase while the concrete stresses will reduce with time. The conclusion of this study was that it is appropriate to use a varying modular ratio to account for creep and shrinkage effects when designing post-tensioned steel girders.

Consequently, two separate service level analysis and stress evaluations were undertaken: one at time equal "one" when the bridge is opened to traffic; another at time equal "infinity". Additionally, where cracking of the slab was shown at the ultimate level, the section properties in the high negative moment regions were reduced to use only the bare steel box and the mild steel and post-tensioned reinforcing of the slab.

The cracked section reduced the negative moments at the intermediate support and increased the midspan positive moments. The most important aspect of this methodology, the use of separate analyses, is that the behavior of the girder is bracketed for all time periods of levels of loading.

MOMENT-CURVATURE ANALYSIS

A moment-curvature study of the post-tensioned box also was undertaken. Here, the advantages of the post-tensioned scheme can be clearly seen. First stage post-tensioning imparts negative curvature to the post-tensioned section. As moment is applied, the variation in curvature for the post-tensioned box follows the same slope as the conventional girder. The second stage post-tensioning again imparts negative curvature to the post-tensioned steel box and there is an increase in stiffness due to the composite action of the concrete slab.

The post-tensioned and the conventional alternative follow the same slope until slab cracking. The level of moment to crack the slab, however, is approximately 5,000 kip-ft. higher for the post-tensioned box than the conventional box. At cracking, both the post-tensioned and conventional boxes see a redistribution of the slab stress to the steel section. The slope of the moment-curvature diagram after cracking is the same for both alternatives and the offset between the two curves, while in the elastic range, is the curvature produced by the first and second stage post-tensioning.

The results of the moment-curvature analysis show two benefits of the post-tensioned steel box. First, for the same amount of material and some effort, the flexural capacity of the box is improved by the post-tensioning. Second, the negative curvature imparted by the post-tensioning reduces the curvature of the box for a given applied moment and hence the net deflection of the box under loading will be less. A comparison of deflections showed the post-tensioned steel bridge is deflected 2.7 in. at midspan under dead load plus live load. This is equivalent to L/907. The identical conventional box girder showed a deflection of 3.73 in. or L/656. This would exceed the limit of L/800 per AASHTO and the section size would have to be increased.

First-Stage Post-Tensioning

As stated previously, the first stage post-tensioning was applied to the girder top flange at the negative moment regions over the piers. An 8x8x5/8 structural tube encloses the post-tensioning strands and was adapted for connection to the conventional flange with welded tabs. The tabs were then bolted to the conventional flange using a modified splice. To avoid overstressing the girder, special instructions were given regarding the application of the first stage post-tensioning.

First, prior to application of the first stage, top lateral bracing and temporary bracing for the tube section are required to be in place. Single end stressing, from either tendon end, with two jacks per girder, was instructed.The two tendons are stressed sumultaneously wiht a 5% maximum differential pressure between jacks.

The plans included camber diagrams and ordinates for first and second stage post tensioning, as well as dead loads, superimposed dead loads and creep and shrinkage effects. The plans also stated the amount of elastic shortening that would be expected due to the first-stage post-tensioning. To negate any uncertainties with the deflections produced by the first-stage post-tensioning, it was the FDOT's preference to require that all the first-stage post-tensioning be done in the steel fabrication shop. This also allows shop fitting of the steel girders before shipment to the jobsite to avoid fit-up problems in the field.

Second-Stage Post-Tensioning

The second-stage post-tensioning tendons were staggered over the piers for efficiency. The post-tensioning force was introduced at blisters located on the underside of the slab. In lieu of blisters, the design considered the use of block-outs, with either plate anchorages and small jacks or coupled tendons and jacks with curved stressing noses. The major impetus for contemplating block-outs was to avoid the complication of stick forming beneath the blisters. With block-outs, the entire length of the girder could use stay-in-place forms, which had two advantages. First, the forming would be simplified. And second, the roadways beneath the bridge would have to be closed for only a very short time during construction. The disadvantage of the block-outs, though, is that a

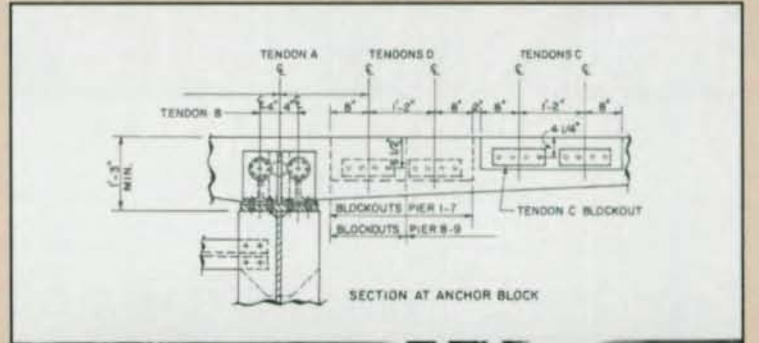
Bonners Ferry Bridge

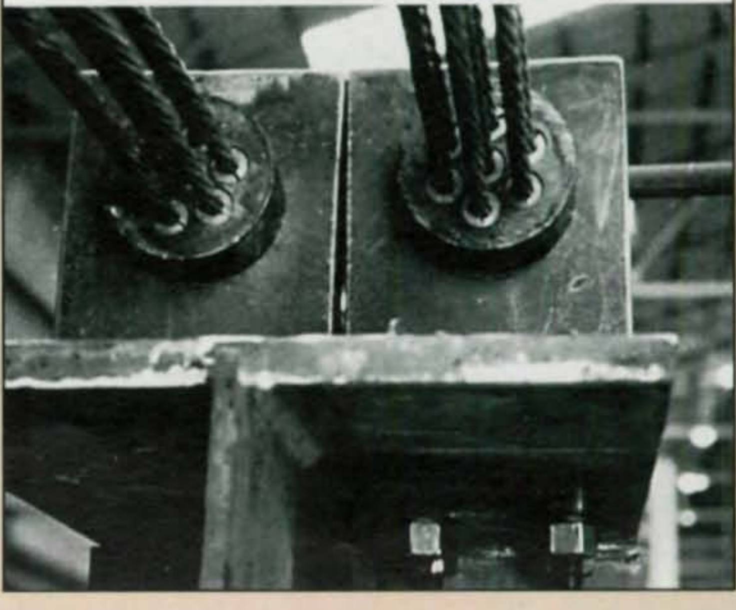
In 1980, the Idaho Transportation Department and the FHWA hired T.Y. Lin International to design a steel alternate to a prestressed concrete design for the replacement of a bridgeover the Kootenai River in Bonners Ferry, ID. The innovative—and cost saving—result was a prestressed steel girder bridge.

The bridge features longitidinal cable stressing of the steel girders and post-tensioning of the steel/concrete composite section in the negative bending moment regions over the piers. The stressing was applied in two stages, in order to limit the stresses during construction as well as in the service condition. The first stage of stressing produced a positive bending moment on the steel girder before the concrete deck was poured. This was done to control dead load tension stress induced in the relatively thin top flange. The second stage had as its purpose production of sufficient longitudinal compressive stress in the concrete deck to eliminate tensile stresses induced on the composite steel/concrete section by the AASHTO HS25 service loading. Since, thereby, the deck is always caused to be in compression, and is attached to the top flange by shear connectors placed throughout the full length of the bridge, full composite action is achieved not only for regions of positive moment, but also for the negative moment regions over the piers.

Another major design feature is the transverse prestressing of the concrete deck. This is accomplished by using strands placed in ducts and anchored at the edge of the concrete deck. For the portion over the piers, the concrete deck is prestressed in two directions, insuring a crack-free surface in this often troublesome region. One consequence of the transverse prestressing is that spacing between the steel plate girders can be increased to reduce the number of girders required for the 70-ft.-wide deck. The wide spacing of 18-ft. between girders results in only four girders being required for the full length of th bridge. Transverse prestressing also allowed a long overhang of the deck cantilevering beyond the outside girders, which meant the pile caps could be kept could be kept narrow and substructure costs could be reduced.

(Information condensed from USS Bridge Report, June 1985)





substantial portion of the concrete deck would require stressing pockets and subsequent patching of the pockets. Also, corrosion of post-tensioning anchor systems has been cause for concern. Based on FDOT experience, these patches—unless extraordinary measures are used—generally leak and invariably fail to control the ingress of corrosion to the post-tensioning system. Therefore, to assure maximum long-term performance, blisters were used in the final design.

Terminating the tendon at the blister complicated the slab design, however. Specifically, additional slab steel was required to resist bursting forces, deviation forces, tie-back forces and local slab moments produced at the anchorage. Influence surfaces were used to estimate the slab's two-way actions.

Creep And Shrinkage

The consideration of long-term losses for prestressed steel structures poses a unique problem. To account for creep and shrinkage losses for the first-stage post-tensioning, only the stress change at the girder top flange from time equal one to time equal infinity needs to be tabulated. For the post-tensioned box application, the long-term loss for the first-stage tensioning was only 1.5 ksi, or 1.3% of the jacking stress after elastic shortening and friction losses.

Time dependent losses for the second-stage post-tensioning were evaluated by the noniterative step-by-step method and long-term losses due to creep and shrinkage of the concrete were found to be 4.6%.

To avoid cycling through the design to account for concrete creep and shrinkage losses, it is important to predict the long-term losses for post-tensioning as accurately as possible. The general results of parametric studies show that an assumed long-term loss due to creep and shrinkage of concrete of 8 per-

MODERN STEEL CONSTRUCTION
Reader Service Card

Please type or print only

name

company name

address

city　　state　　zip code

(　　)　　(　　)

business telephone　　fax number

For free information on advertisers and other companies mentioned in this issue, circle the appropriate number below:

1	2	3	4	5	6	7	8	9	10	11	12
13	14	15	16	17	18	19	20	21	22	23	24
25	26	27	28	29	30	31	32	33	34	35	36
37	38	39	40	41	42	43	44	45	46	47	48
49	50	51	52	53	54	55	56	57	58	59	60
61	62	63	64	65	66	67	68	69	70	71	72
73	74	75	76	77	78	79	80	81	82	83	84
85	86	87	88	89	90	91	92	93	94	95	96
97	98	99	100	101	102	103	104	105	106	107	108
109	110	111	112	113	114	115	116	117	118	119	120
121	122	123	124	125	126	127	128	129	130	131	132

☐ Check this box to renew or start a **FREE*** subscription to *Modern Steel Construction* *This offer good only for practicing professionals within the U.S.

signature

1. Type of work
Please circle **all** that apply
a. Bridges
b. Office Buildings
c. Hospitals
d. Retail
e. Industrial
f. Schools
g. Multifamily
h. Institutional
i. Low-Rise
j. High-Rise

2. Your position
Please circle **ONLY** one
a. Owner/President
b. Chief Engineer/ Chief Architect/ Department Head
c. Staff Engineer/ Staff Architect/ Construction Manager
d. Other

3. Type of business
Please circle **ONLY** one
a. Structural engineer
b. Civil engineer
c. Structural steel fabricator
d. Other fabricator
e. Educator/libary
f. Architect
g. Building owner/developer
h. Steel mill
i. Erector
j. Steel product manufacturer
k. GC/CM
l. Student
m. Other

4. Reason for Inquiry
Please circle **ONLY** one
a. Information file
b. Immediate or Future Purchase

NOVEMBER

Use these cards to request information from manufacturers and other companies mentioned in this issue. Simply circle the numbers referenced in the advertisement or article, fill out the rest of the information requested on the card, and either mail it to: Modern Steel Construction, Creative Data Center, 650 South Clark St., Chicago, IL 60605-9960 or fax it to: **312/922-3165.**

Please type or print only

name

company name

address

city　　state　　zip code

(　　)　　(　　)

business telephone　　fax number

For free information on advertisers and other companies mentioned in this issue, circle the appropriate number below:

1	2	3	4	5	6	7	8	9	10	11	12
13	14	15	16	17	18	19	20	21	22	23	24
25	26	27	28	29	30	31	32	33	34	35	36
37	38	39	40	41	42	43	44	45	46	47	48
49	50	51	52	53	54	55	56	57	58	59	60
61	62	63	64	65	66	67	68	69	70	71	72
73	74	75	76	77	78	79	80	81	82	83	84
85	86	87	88	89	90	91	92	93	94	95	96
97	98	99	100	101	102	103	104	105	106	107	108
109	110	111	112	113	114	115	116	117	118	119	120
121	122	123	124	125	126	127	128	129	130	131	132

☐ Check this box to renew or start a **FREE*** subscription to *Modern Steel Construction* *This offer good only for practicing professionals within the U.S.

signature

1. Type of work
Please circle **all** that apply
a. Bridges
b. Office Buildings
c. Hospitals
d. Retail
e. Industrial
f. Schools
g. Multifamily
h. Institutional
i. Low-Rise
j. High-Rise

2. Your position
Please circle **ONLY** one
a. Owner/President
b. Chief Engineer/ Chief Architect/ Department Head
c. Staff Engineer/ Staff Architect/ Construction Manager
d. Other

3. Type of business
Please circle **ONLY** one
a. Structural engineer
b. Civil engineer
c. Structural steel fabricator
d. Other fabricator
e. Educator/libary
f. Architect
g. Building owner/developer
h. Steel mill
i. Erector
j. Steel product manufacturer
k. GC/CM
l. Student
m. Other

4. Reason for Inquiry
Please circle **ONLY** one
a. Information file
b. Immediate or Future Purchase

NOVEMBER

NO POSTAGE
NECESSARY
IF MAILED
IN THE
UNITED STATES

BUSINESS REPLY MAIL

FIRST CLASS MAIL PERMIT NO. 12522 CHICAGO, IL

POSTAGE WILL BE PAID BY ADDRESSEE

MODERN STEEL CONSTRUCTION
c/o CREATIVE DATA SERVICES
650 S CLARK ST
CHICAGO, IL 60605-9960

Use these cards to request information from manufacturers and other companies mentioned in this issue. Simply circle the numbers referenced in the advertisement or article, fill out the rest of the information requested on the card, and either mail it to: Modern Steel Construction, Creative Data Center, 650 South Clark St., Chicago, IL 60605-9960 or fax it to: **312/922-3165.**

NO POSTAGE
NECESSARY
IF MAILED
IN THE
UNITED STATES

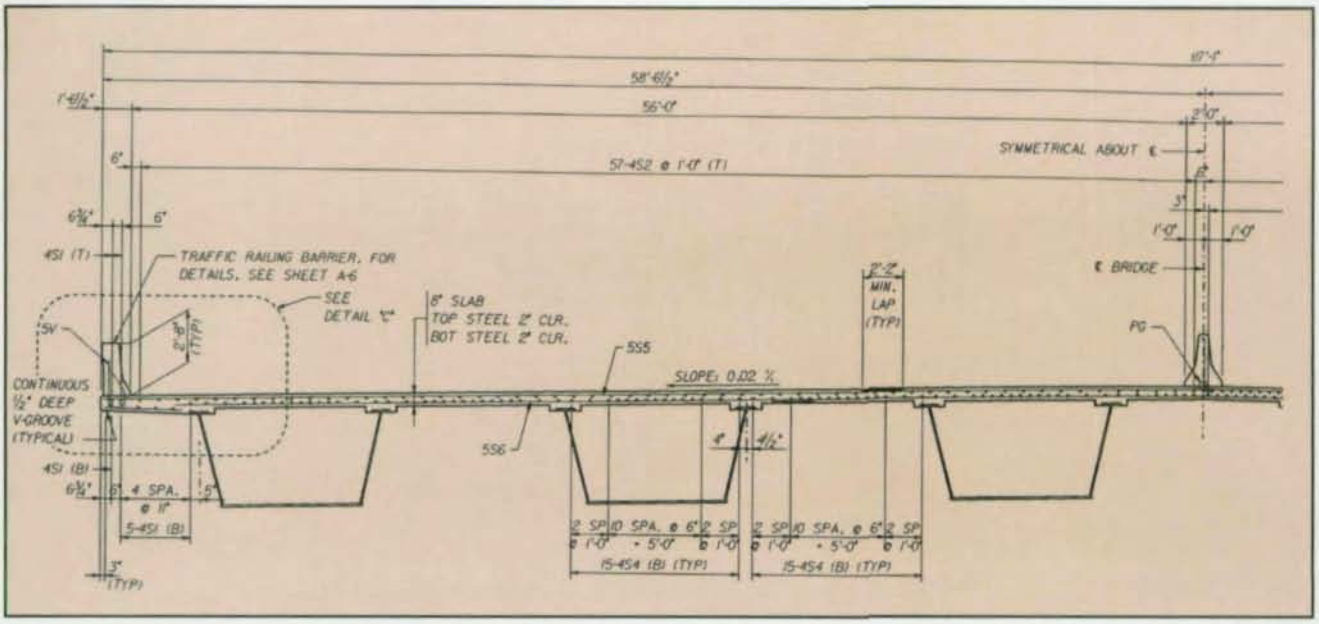
BUSINESS REPLY MAIL

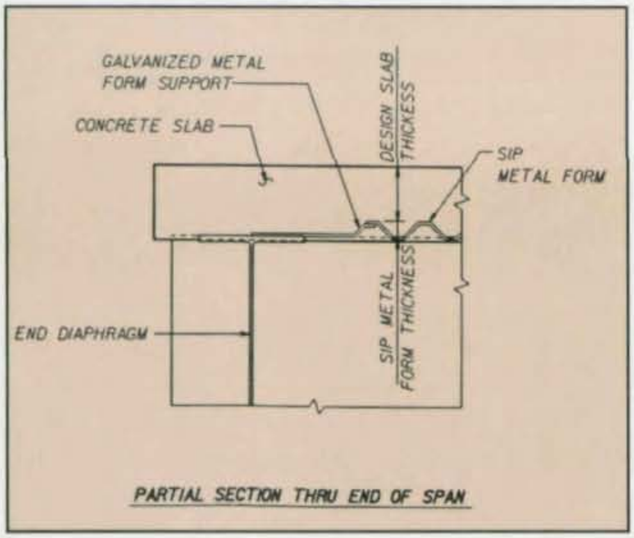
FIRST CLASS MAIL PERMIT NO. 12522 CHICAGO, IL

POSTAGE WILL BE PAID BY ADDRESSEE

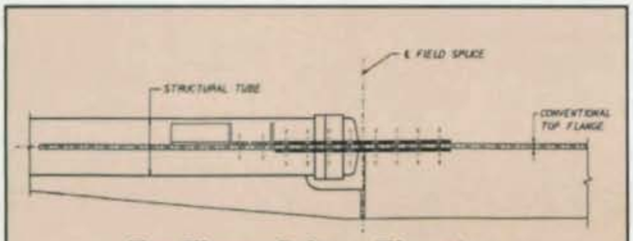
MODERN STEEL CONSTRUCTION
c/o CREATIVE DATA SERVICES
650 S CLARK ST
CHICAGO, IL 60605-9960

MODERN STEEL CONSTRUCTION
Reader Service Card





PARTIAL SECTION THRU END OF SPAN



Top Flange Splice—Elevation

cent is reasonable for most configurations. Additionally, when starting the design, an estimate for the final prestressing stress of 0.6 times the tendon tensile stress based on an initial jacking stress of 0.8 times the tensile stress should be suitable.

The post-tensioned steel box shows improved fatigue performance, primarily because the composite action reduced the fatigue stress ranges. At the negative moment region, the top flange stress range was reduced by a factor of 2 when compared to the conventional alternative. The lower stress range at the negative moment region results in a one category improvement in AASHTO fatigue rating. However, Charpy V-notch testing still is not eliminated.

The box girders generally require top flange lateral bracing to resist the horizontal component of the shear in the inclined webs. They also prevent compression instability of the top flange before the concrete deck is in place. Special attention must be given to the bracing layout to miss both the splice and blister locations. One special feature required was temporary bracing to support the girder section at each free end before it was assembled with adjacent girder sections. After the bare beam at the negative moment region was spliced with the rest of the bridge during construction, permanent bracing between the field sections was assembled and the temporary bracing was removed.

While prestressing steel bridges results in a more complex design, it also provides overall savings from reduced cost for the bridge structure, approach embankments, retaining walls and right-of-way acquisition. Also, the prestressing shares some of the load, which improves ductility, and enhances a structures long-term performance by reducing fatigue stress and corrosion of the slab reinforcing.

Robert B. Anderson, P.E., is a project engineer at Greiner, Inc., in Tampa, FL. Greiner has more than 40 offices nationwide, as well as in Hong Kong and Malaysia. The company specializes in transporation engineering for highway bridges and airport projects.

Please circle # 21

SUCCESSFUL RESURFACING FOR AN ORTHOTROPIC STEEL DECK BRIDGE

By Vellore S. Gopalaratnam and Arthur M. Dinitz

The Poplar Street Bridge in St. Louis, one of approximately 30 orthotropic steel-plate deck bridges in the world, carries approximately 130,000 vehicles—including about 15,000 large trucks—across the Mississippi River each day. The five-span bridge is 2,165-ft. long and consists of two independent bridges supported on a set of common piers. Each bridge carries four lanes of traffic and is supported by two box girders with girder depths ranging from 16 ft. to 25 ft. The deck thickness typically is $^{9}/_{16}$ in. and is stiffened by closed trapezoidal stringers or ribs. The $^{5}/_{16}$-in. thick stiffeners on 13-in. centers are 11-in. deep and run along the length of the bridge. Load from the deck also is transferred to the box girders by transverse floor beams spaced on 15-ft. centers. Transverse frames brace the box girders at 60-ft. centers coinciding with every fourth floor beam location, providing the bridge additional torsional rigidity.

When the bridge was constructed in 1967, the steel deck was covered with a wearing surface consisting of two layers of epoxy tack coat and $1^{1}/_{2}$-in. of rubberized asphalt concrete wearing surface. Stone chips were embedded in the second layer of epoxy as an anchor for the rubberized asphalt layer. This system performed well until 1983, when it was completely removed and replaced with a second, identical surface. Unfortunately, this time the system only lasted three years. In 1986, the eastbound lanes were replaced with a proprietary system that included a fiberglass reinforcing mat in the asphalt wearing surface. Despite the reinforcing mat, unacceptable amounts of rutting and shoving necessitated the placement of a new wearing system in 1992.

After testing six systems, a Transpo T-48 epoxy concrete was chosen. A total of 20 working days was used by the contractor, Pace Construction Co. of St. Louis, to place the 226,000-sq.-ft. of approximately $^{1}/_{2}$-in. thick wearing surface.

This was the first use of the system on large steel decks subjected to a combination of severe environmental and heavy traffic conditions. As a result, the system was heavily monitored by the Missouri Highway and Transportation Department (MHTD), which awarded the University of Missouri a five-year contract for long-term inspection and testing of the wearing surface. The yearly inspection includes resistivity tests to qualitatively monitor cracking in the wearing surface, pull-out tests to establish the adhesion strength in tension between the wearing surface and the deck-plate, chain-drag/acoustic tests to detect delamination of the wearing surface from the deck-plate, observations to record wearing surface thickness, aggregate loss and other signs of deterioration.

Two visible cracks, one at each end of the southmost-eastbound lane (in the transition zone where the overlay thickness changes from $2^{1}/_{2}$-in. at the finger-type expansion device to $^{1}/_{2}$ in. on the bridge) appeared after one year of service and were repaired using Transpo T-48 epoxy primer. No further crack growth or deterioration of the wearing surface in the vicinity of the cracks were observed even after two additional years. While the cause of these cracks has not been conclusively established, differential thermal expansion is a probable culprit.

After approximately three years of service, the Transpo T-48 epoxy concrete wearing surface system has performed very well, with no additional visible cracking, loss of skid resistance, excessive wear or any other failure to date.

For more information on the Transpo System, **please circle no. 30**.

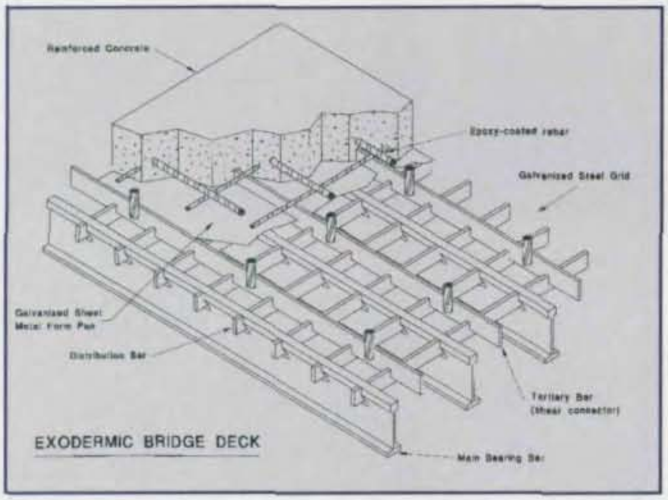
Vellore S. Gopalaratnam is an associate professor of civil engineering at the University of Missouri and Arthur M. Dinitz is president of Transpo Industries.

EXODERMIC BRIDGE DECKS

An exodermic bridge deck is a 3.5-in. to 5-in. reinforced concrete slab on top of, and composite with, an unfilled steel grid. This efficient deck design permits significant weight savings compared to a standard reinforced concrete deck while providing the same or better stiffness and strength. The modular nature of the deck permits rapid erection, even during very short (overnight) work periods.

An exodermic deck is comprised of two main components: a fabricated steel grid similar to those that have been used for concrete-filled grid bridge decks for many years, and a reinforced concrete component, which is typically 3.5-in. to 5-in. thick, with rebar selected to provide negative moment capacity. The concrete component is made composite with the steel grid by the embedment of shear connecting elements known as "tertiary bars".

In place on a structure, the concrete handles compressive forces from traffic, while the steel grid han-



EXODERMIC BRIDGE DECK

dles tensile forces.

One key benefit of an exodermic deck is its concrete riding surface. In fact the concrete component is often designed to the same specifications as the top half of a standard concrete slab, and can be installed and maintained using the same tools and techniques as standard bridge decks.

Exodermic decks are attached to the superstructure of the bridge by the use of headed shear studs, and the placement of concrete full depth through the grid over the supporting beams. No field welding or bolting of steel grid panels are required.

Because the design is modular, exodermic decks are ideal where deck construction or replacement must be accomplished rapidly. The concrete component of an exodermic deck can be pre-cast or cast-in-place. Pre-cast exodermic decks are used where traffic conditions require rapid deck replacement, generally during a nighttime work window. Such work permits the structure to be fully open to traffic during the day. Pre-cast exodermic projects include the Driscoll Bridge on the Garden State Parkway, the Pitman Creek Bridge (US 27) a 700-ft. deck truss in Somerset, Kentucky, emergency repairs on the Tappan Zee Bridge over the Hudson River, and the Troy-Menands (Route 378) Bridge over the Hudson River in Albany, NY.

Where traffic permits, an exodermic deck can be cast-in-place. In these cases, the steel grid acts as a rapidly-placeable, stay-in-place form. Significant time savings are realized over conventional forming, and the benefits of a lightweight deck realized at the same time. Examples include the Mohawk River Bridge in St. Johnsville, NY, the Calumet Street Bridge in Philadelphia, the Popolopen Creek (Route 9W) Bridge in Highland Falls, NY, and the Clover Run Bridge in St. George, WV.

In addition to the re-decking of existing structures, Exodermic deck is also used on new construction. Its light weight permits the entire bridge to be lighter in weight, and thus less expensive. Projects of this type include an interim viaduct that will be carrying 1700-ft. of I93 southbound in Boston during construction of the Central Artery project, and two new bridges in Ithaca, NY.

The Exodermic Bridge Deck Institute (914-722-2448) is an information source for exodermic design and construction, providing printed and computer-based design aids, suggested specifications, informational materials, and technical assistance to bridge engineers, owners, and contractors.

For more information, **circle no. 69**.

OTHER DECKING PRODUCTS:

United Steel Deck, in affiliation with Nicholas J. Bouras, Inc., offer a wide variety of decking solutions. A wide variety of special finishes combine with roll forming and bending capabilities to provide solutions to he most unique decking demands.

For more information, contact Nicholas J. Bouras, Inc., P.O. Box 662, 475 Springfield Ave., Summitt, NJ 07902-0662; 908/277-1617 or **circle no. 33.**

Wheeling Corrugating Co. offers all types of premium steel decking, rolled to specific job requirements. The company has six strategically located manufacturing/service locations to fully serve the fabricated structural steel industry.

For more information, call 304/234-2300 or **circle no. 99.**

The Steel Deck Institute has released a new Design Manual that includes all the latest information for deck design and use, including metric specifications, metric load tables, short form specifications, code of recommended standard practice, and design examples.

For more information, contact: Steel Deck Institute, P.O. Box 9506, Canton, OH 44711 (216) 493-7886 or **circle no. 77.**

Epic Metals Corp. offers a wide range of decking solutions, including its MAXSPAN BRIDGE DECK FORM, an entirely new concept in the design of permanent metal deck forms for bridge deck slabs.

For more information, contact: Robert Paul, Product Engineer, Epic Metals Corp., Eleven Talbot Ave., Rankin, PA 15104 (412) 351-3913 or **circle no. 25.**

Heavy Duty Grip Strut bridge inspection walkways are suspended beneath bridge deck to enable close inspection of load-carrying members. The well-made catwalks span 24' openings with minimal deflection, which reduces the need and expense of extra supports. Also, gravel, mud, snow and ice fall through large diamond-shaped openings. Choices include 9, 10, or 11 gauge grating with serrated or non-serrated steel, and widths up to 36" with 5" integral toeboards, which eliminate extra welding.

For more information, contact: GS Metals Corp., R.R. 4, Box 7, Pinckneyville, IL 62274 (800) 851-9341 or (618) 357-5353 inside Illinois. or **circle no. 80.**

Grid Reinforced Concrete Deck manufacturers off products with deck thicknesses ranging from 3-in. to 10-in. withe weights as low as 45 lb./sq. ft. Modular grid panels also play a vital role in the complex traffic staging often required on rehabilitation projects.

For more information, contact: BGFMA at 412/547-2660 or **circle no. 21.**

268

STEEL MARKETPLACE

STEEL MARKETPLACE

Advertisers' Index