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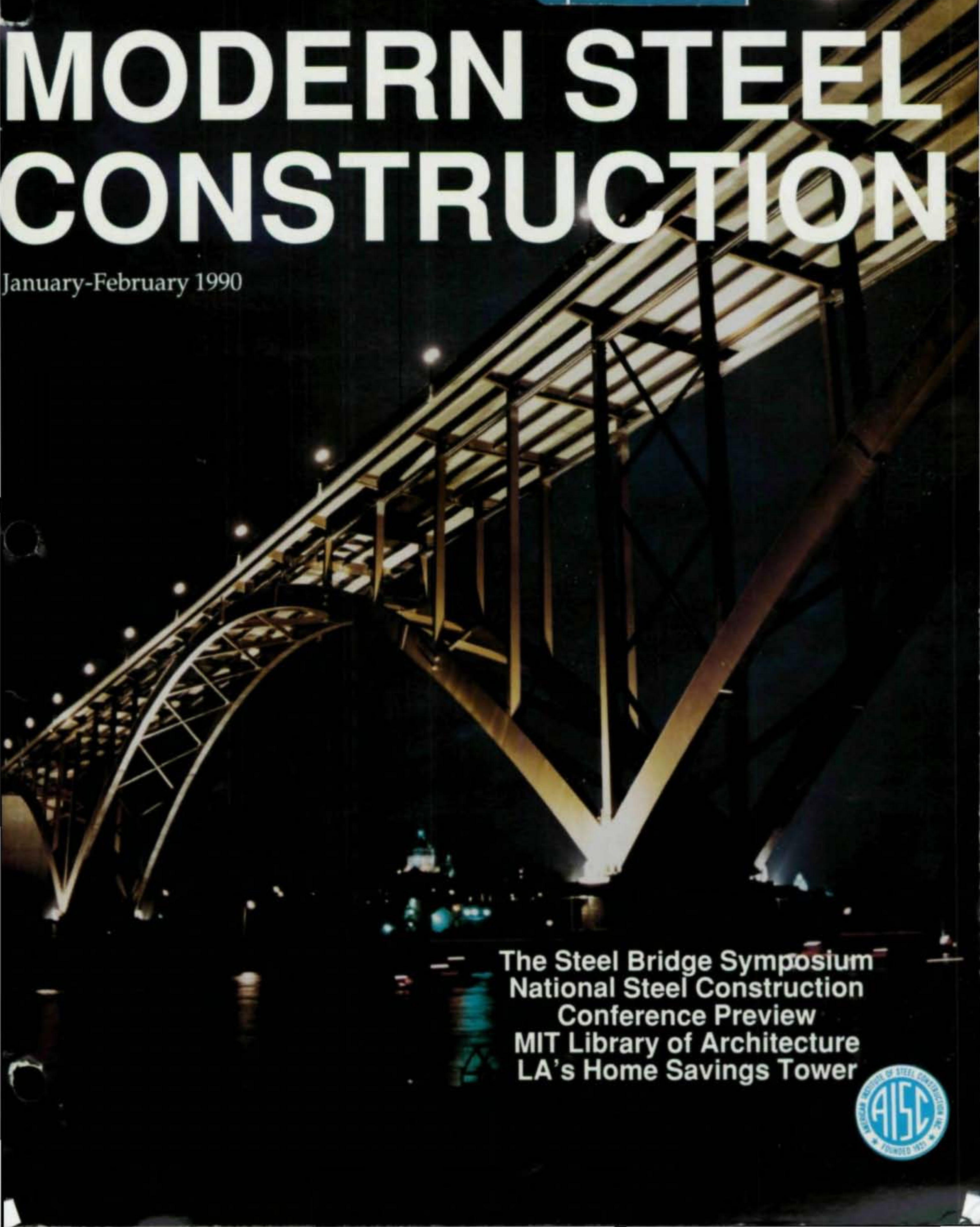
MODERN STEEL CONSTRUCTION

January-February 1990



The Steel Bridge Symposium
National Steel Construction Conference Preview
MIT Library of Architecture
LA's Home Savings Tower

UNITED STEEL DECK, INC. DECK DESIGN DATA SHEET No. 13

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THE TABLE REPRESENTS NORMAL INVENTORIES; HOWEVER ANY FINISH ON ANY PRODUCT MAY BE AVAILABLE ON SPECIAL ORDER.

NOTES — ROMAN NUMERALS IN THE TABLE CORRESPOND TO NUMERALS IN NOTES.

I. A. CHECK U.L. FIRE RESISTANCE DIRECTORY FOR FINISH REQUIREMENTS. GALVANIZED DECK SHOULD BE USED ON *ROOF* CONSTRUCTION WITH SPRAYED FIRE RESISTIVE MATERIALS. (SFRM).
B. GALVANIZED DECK IS RECOMMENDED FOR HIGH HUMIDITY AREAS.
C. GALVANIZED ROOF DECK IS RECOMMENDED FOR ROOF CONSTRUCTIONS WITH INSULATION BOARDS THAT ARE FASTENED TO THE DECK WITH PIERCING FASTENERS.
D. USD RECOMMENDS THE USE OF GALVANIZED MATERIALS FOR MOST EXPOSURES.
E. GALVANIZED STEEL IS COVERED BY ASTM A446; GALVANIZING IS COVERED BY ASTM A525; G60 AND G90 ARE COATING WEIGHTS.

II. A. "PHOS/PTD." MEANS THE FLOOR DECK IS ONLY PAINTED ON THE EXPOSED SIDE—THE CONCRETE SIDE SHOULD DEVELOP TIGHT RUST BEFORE THE CONCRETE IS POURED.
B. USE ONLY FOR INTERIOR APPLICATIONS—I.E. OFFICES OR HOTELS.
C. CHECK U.L. FIRE RESISTANCE DIRECTORY—SEE NOTE I.A.
D. "PHOS./PTD." IS APPLIED TO ASTM A611 STEEL.

III. A. "PRIME PAINTED" MEANS A PRIMER COAT OF PAINT IS APPLIED OVER CLEAN BARE STEEL. THE PRIMER PAINT IS FORMULATED TO HAVE "TOOTH" TO HOLD SUBSEQUENT APPLICATIONS OF FINISH PAINT BUT IT IS NOT INTENDED TO PROVIDE EXTENSIVE WEATHER PROTECTION; IT IS FREQUENTLY LEFT EXPOSED IN WAREHOUSES AND MANUFACTURING PLANTS, AND WHEN USED WITH SUSPENDED CEILINGS.
B. USE FOR BALLASTED ROOFS OR ADHERED ROOF SYSTEMS— SEE NOTE I.C.
C. SALT SPRAY (AND OTHER) TEST RESULTS ARE AVAILABLE ON REQUEST.
D. "PRIME PAINTED" DECK IS MADE FROM ASTM A611 STEEL.

IV A. "GALV. + PAINT" MEANS PRIMER IS FACTORY APPLIED OVER GALVANIZED STEEL. THE PRIMER PAINT IS AS DESCRIBED IN III.
B. THIS FINISH IS MOST ECONOMICAL WHEN A FINAL COAT OF PAINT IS TO BE FIELD APPLIED.
C. USE IN HIGH HUMIDITY AREAS—THE PAINT PLUS GALVANIZING PROVIDES EXTREMELY GOOD MOISTURE PROTECTION.
D. "GALV. + PAINT" USES ASTM A446 STEEL.

V. A. FINISH COATS OF PAINT CAN BE FACTORY APPLIED. THIS IS DONE ON THE COILS OF STEEL BEFORE FORMING INTO DECK. ALMOST ANY COLOR OR PAINT TYPE CAN BE USED—HOWEVER TO BE ECONOMICAL, THE ORDER SHOULD BE FOR AT LEAST 20,000 SQUARE FEET.
B. WHEN INSTALLING DECK WITH A SPECIAL FINISH, SCREWED SIDE LAPS ARE RECOMMENDED: AND, IN MOST CASES, SCREWS, PNEUMATIC OR POWDER DRIVEN FASTENERS SHOULD BE USED AT SUPPORTS.
C. FINISH PAINT IS NORMALLY APPLIED OVER GALVANIZED STEEL CONFORMING TO ASTM A446.

VI. A. UNCOATED STEEL MEANS THERE IS NO COATING AT ALL. IT IS FREQUENTLY REFERRED TO AS "BLACK" STEEL.
B. UNCOATED STEEL CONFORMS TO ASTM A611.

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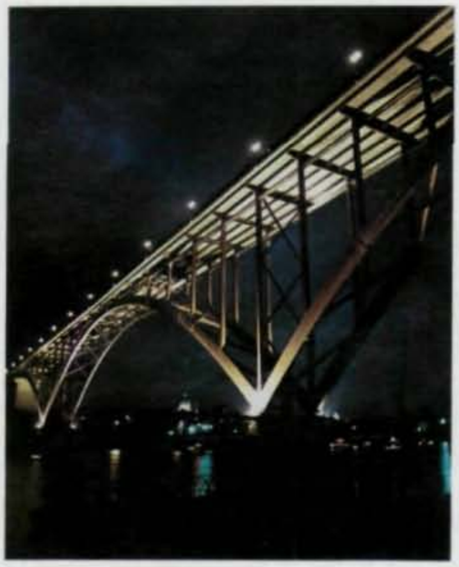
MODERN STEEL CONSTRUCTION

Volume 30, Number 1 January-February 1990



After considering 14 replacement possibilities, the Minnesota DOT chose a steel bridge with three balanced cantilver arch spans to replace the Smith Avenue High Bridge in St. Paul. Aesthetic concerns were paramount to the community, and as this night shot amply demonstrates, the bridge designers were able to meet this need.

FEATURES

24 INVERTED LIBRARY HANGS FROM ROOF
To maximize floor area in the addition to the MIT Library of architecture, the designers created column-free space by suspending the floors from large roof girders

29 BACK TO THE FUTURE
L.A.'s Home Savings Tower demonstrates that combining a masonry sensibility with the practicality of steel can create a flexible building

34 COMPLEX SUPPORTS FOR STEEL SKYWAY
Steel proved to be a less costly option for a Rochestor, N.Y., pedestrian bridge—and it provided the desired aesthetics

38 UNIQUE STEEL BRIDGE MEETS AESTHETIC GOALS
Three balanced cantilever arch spans provided the required visual beauty while staying within the desired budget for the Smith Avenue High Bridge in St. Paul, Minn.

45 TWO STATE-OF-THE-ART BRIDGES FOR MAINE
Under construction is one of the first bridges designed using the ALFD procedure, and just 22 miles away is a recently completed Rigid Frame Steel Girder Bridge

48 EXPANDED ALFD USE POSSIBLE FOR THE FUTURE
A recent test program indicates that expanded use of ALFD design in the future may lead to substantial cost savings

53 WEATHERING STEEL REQUIRES PROPER DETAILING
Weathering steel can substantially reduce bridge construction costs and have a positive effect on the environment

59 TWO GROUPS OFFER AID FOR BRIDGE DESIGNERS
The Bridge And Structures Information Center and the Council for the Advancement of Steel Bridge Technology are both founts of information

DEPARTMENTS

6 EDITORIAL

10 STEEL NEWS

14 CALENDAR

20 STEEL TECHNOLOGY

62 1990 NATIONAL STEEL CONSTRUCTION CONFERENCE

76 PRODUCTS

82 ADVERTISERS' INDEX

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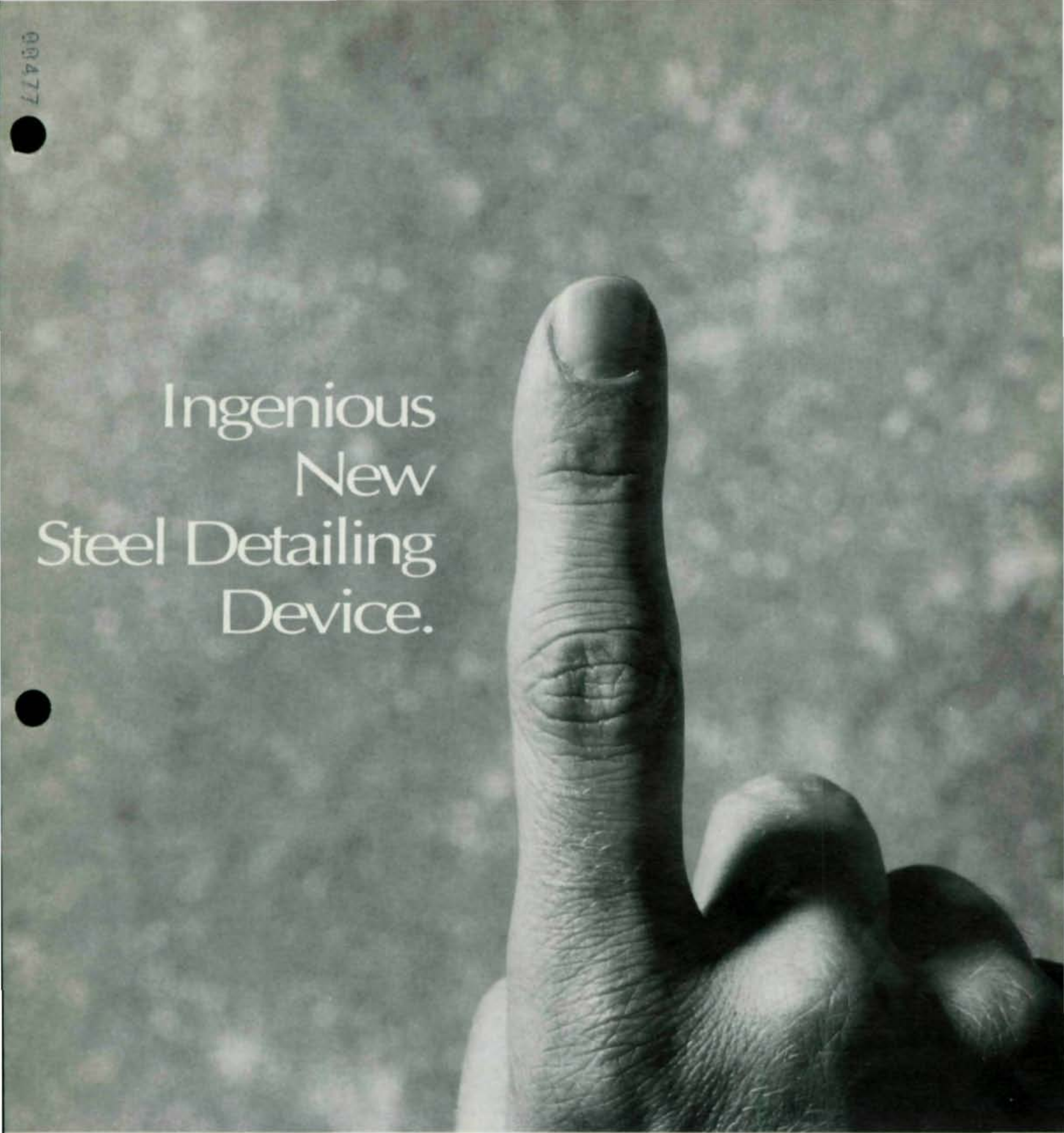

Thirtysomething

"Never trust anyone over 30" was the rallying cry of the "flower-power" generation of the 1960s. But as the Yippies aged into Yuppies, being "Thirtysomething" gained a new acceptability.

That's particularly fortuitous for *Modern Steel Construction* which is commemorating its 30th birthday in 1990. And to celebrate, we've given the magazine a new design. We're shifting to a better balance of staff written and contributed articles, we're expanding our news section and we'll be featuring new product introductions in each issue.

If you're about to begin work on an interesting project, or have just completed a project, drop us a note describing it. Also, if you have any comments—either about something you've read in either this or a previous issue, or about anything pertaining to the fabricated steel industry—write us a letter. You can reach us at: Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

And that brings us to one last change. After many years in Chicago's historic Wrigley Building, we moved in November.

Of course, we couldn't move to just any building. Whether by chance or some pre-ordained plan, we ended up in the first multi-story building to be built with high strength steel with a yield point of 50,000 psi furnished to ASTM specification A440.

One East Wacker Dr.

High-strength steel is used in the columns for the three basement floors and the first 23 floors of the 41-story building. The remaining columns and all floor beams in the 18-year-old building are of structural carbon steel, ASTM specification A7, with a yield point of 33,000 psi.

According to a USS Structural Report from 1962: "The decision to use USS Man-Ten (A440) steel for the columns was based on an economic study. Since the loads in the columns supporting the lower floors are large, the sections, if made of structural carbon steel, would normally require cover plates to provide sufficient material to carry the loads safely."

Another interesting feature of the building is the use of moment resisting beam-to-beam connections instead of diagonal bracing. "Because wind forces require moment resisting beam-to-column connections, it was considered advisable to include this restraining effect in the design of the beams for gravity loads."

During the 1960s, high-strength steel changed the way buildings were designed. And during the 1990s, the increased use of Load & Resistance Factor Design will cause further changes. The fabricated steel industry is constantly evolving, and *Modern Steel Construction* is evolving right along with it. Or to paraphrase a well-known commercial, "We're not getting older, we're getting better."

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All these factors inevitably add up to lower in-place anchoring costs. In some types of construction, most notably composite, further savings result from the use of lighter and less costly beams that also reduce building height and weight.

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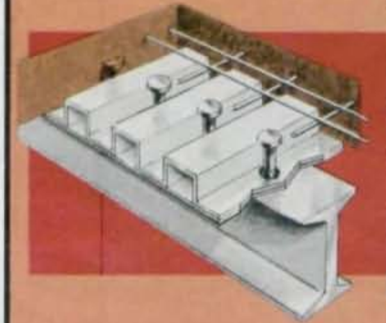
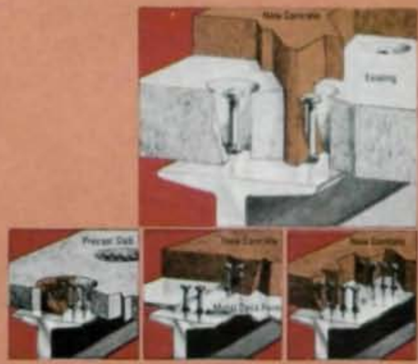
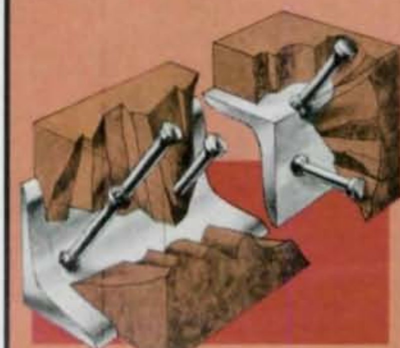
Applications



Composite Bridges – Shear connectors provide equal shear in all directions, eliminate distortion that might result from hand welding and permit more satisfactory compaction of concrete around the connectors.

Composite Buildings – Shear connector studs welded to the beam or through a permanent form steel deck result in increased live load capacity. As much as 20% less steel may be used and shallower floor sections reduce building height.

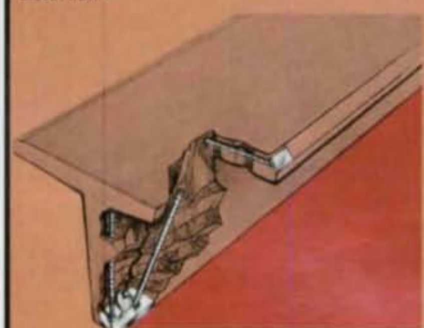




Retrofitting – Bridge retrofitting usually involves removing the old concrete and replacing it with new concrete tied to the beam with stud welded shear connectors.
Applying new facia and interior retrofitting of old buildings requiring installation of new electrical fixtures, sprinklers and piping can be accomplished by welding threaded studs to structural members.

Concrete Anchoring – Stud welded headed concrete anchors deliver specified axial tension and shear strength values and can be applied up to three times faster than hand welded anchoring devices. Other advantages include much higher yield points, elimination of costly set-up time for shearing and bending, stronger welds, reduced material handling and no distortion.

Insulation/Lagging – Stud welded fasteners secure all types of insulation material in all density ranges faster, easier, more economically and better than any other methods.



Precast/Prestressed – Because of their known values, anchor studs can be used in standardized designs for such connections as bearing plates for beams and tees, shear keys for tees, column baseplates, and various other embedded steel elements. In these applications, stud welding reduces cost per plate, ensures consistently high weld quality, frees certified welders for other jobs, eliminates long lead time and storage problems.

Electrical/Mechanical – Threaded studs and a variety of stud configurations are used to fasten conduit clamps, lighting fixtures, outlet boxes, sprinkler systems, cable runs and piping. Fast positive attachment is achieved without holes or costly clamping devices.

Other cost saving construction applications are securing concrete forming and timber shoring, wood nailers, crane and guide rails, grating, refactory and wear resistant materials.

August 8, 1989

When the heat's on, you have another good

What happened in New Jersey on August 8, 1989 was an unusual emergency. But the events that followed may be even more extraordinary.

A fire broke out in 60,000 tons of rubbish under span 17 of Interstate 78–the main highway connecting northern New Jersey and eastern Pennsylvania.

The intense heat buckled girders, the deck dropped and the New Jersey DOT closed 8 of the 10 lanes.

On August 21, the fabricator–High Steel Structures of Lancaster, Pennsylvania–was awarded the contract to replace the steel. Knowing that his company would incur a $10,000 penalty for every day delivery went past October 26, High Steel's purchasing agent, Dale Aulthouse, ordered 332 tons of ASTM A 36 steel plate from Bethlehem the same day.

"We surveyed all the major steel producers and found that Bethlehem could give us the best delivery schedule and the greatest flexibility," Aulthouse said.

October 10, 1989

reason to call Bethlehem.

Just 4 weeks later, on September 15, Bethlehem delivered the steel to Lancaster. Only three weeks later, on October 10, the finished girders were delivered to the site.

"Bethlehem's flexibility and performance allowed us to accelerate our schedule, so what normally takes a year–design, purchase, steel fabrication, delivery and erecting–took only two months", said Patrick Loftus, High Steel's president.

The road reopened on December 15–because everyone involved performed above and beyond the call of duty.

This is an unusual example of our standard operating procedure: we're always ready to do whatever we can to help you get the job done. Even when it might seem too hot to handle.

Bethlehem

Seismic Design Proves Effective In San Francisco

By Rudolph Hofer, Jr.

Despite more than $6 billion in damage and the loss of 63 lives, most experts agree that San Francisco came through the October 1989 Loma Prieta earthquake better than almost any other city in the world would have. As one person put it: "On a scale of one to 10, Armenia is at zero and San Francisco is at nine."

If nothing else, the San Francisco area demonstrated that seismic design works. In contrast, an earthquake of this magnitude on the East Coast, where seismic design is still in its infancy, would have been far more devastating. The inherent seismic capabilities of steel—and the need for greater steel reinforcement in earthquake-resistant concrete structures—was demonstrated.

Predictable Damage

In general, most of the damage to structures in Northern California was predictable. Most of the structures that failed were of wood, unreinforced masonry, or unreinforced masonry in-fill. With few exceptions, buildings designed to modern codes came through with little or no damage. This was particularly true of steel structures.

Some of the most dramatic and devastating building failures occurred in the Marina District. These expensive homes built on Bay fill sometime following the 1916 San Francisco Exposition failed due to a combination of soil liquefaction and "soft story" effect. The latter was caused by the presence of garage openings on the first floor of the

Continued on page 12

The Cypress section of the Nimitz Freeway, which was designed in the early 1950s when the AASHTO specification was void of seismic design provisions, sustained the most damage of any major roadway during the Loma Prieta earthquake. The hinges were placed at the bottom of the upper columns, but above the lower deck roadway. As the structure lifted and fell due to vertical acceleration, forces from the earthquake resulted a high compressive load concentrated below the hinge in the region of discontinuous reinforcement. The result was a downward diagonal shear failure of the lower concrete column just below the hinge, and the structure collapsed. Photos courtesy of Bethlehem Steel Corporation.

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Quake, Cont.

structures. The garage opening eliminates a shear wall to transmit the seismic lateral forces to the ground, resulting in horizontal displacement of the buildings.

On the transportation side, where damage is estimated at between $1.8 and $2 billion, of the 1,500 bridges in Northern California that were subjected to strong motion shaking, only 10 were closed. Another 10 bridges have damage requiring temporary support until they are repaired, and 73 bridges have minor damage, mostly cracks and spalling at the joints. The problems were much more serious in concrete structures than in steel. As an example, the concrete Cypress Section of the Nimitz Freeway will take at least three years to repair, while the damaged steel section of the Bay Bridge was repaired and the bridge reopened only one month after the earthquake.

Of the 10 bridges that were closed, only three experienced complete collapse: a 50' section of the Bay Bridge; a 1.25-mile section of the Nimitz Freeway; and the Strove Slough bridge on Highway 1 near Watsonville. Of the other closed bridges, only one was steel, and the damage to that bridge was not to the steel itself, but rather to the concrete piers.

The Strove Slough bridge is a multi-span flat plate concrete bridge that failed by the piers punching through the deck, probably due to vertical acceleration. The bridge has been redesigned and is now under contract for reconstruction.

The 50' section of the Bay Bridge is repaired and the bridge reopened to traffic on November 17, exactly one month after the quake hit.

The Cypress section of the Nimitz Freeway is being torn down under three demolition contracts and the rest of the bridge is undergoing simulated seismic loads. Rebuilding the Nimitz is probably three years away and there is a developing sentiment in Oakland to re-locate it to the west, closer to the Port of Oakland. In the interim, a six-lane surface street with ramps up to the undamaged portion of the freeway at both the north and south ends is being constructed.

The Nimitz Freeway

Design of the Cypress section was begun in 1951 and it was constructed between 1955 and 1957. The double deck, four lane structure is about 60' wide, 50' high, and 6,800' long, almost without any horizontal curvature. Average daily traffic runs between 150,000 and 200,000 vehicles.

At the time it was designed, the AASHTO specification was void of seismic design provisions. Caltrans had developed their own provisions based on what little was known about earthquakes at the time, which probably came from the 1940 El Centro record. The Cypress section was designed to withstand .06g or 6% of the gravity load that was applied to the structure as a lateral load to simulate earthquake forces. This was obviously insufficient because a two-story building just a few blocks from the site had input motion of .26g horizontal (four times what the Cypress section was designed for) and 0.16g vertical from the Loma Prieta earthquake. A newly published Bedrock Acceleration Map by Greensfelder shows a probably bedrock acceleration at the site of .50g emanating from a 7.5 earthquake on the nearby Hayward fault.

The failure mode of the Cypress section is classic as an example of what is now almost forbidden by the requirements of the Structural Engineers Association of California Blue Book, which requirements have lately been adopted by the 1988 Uniform Building Code. This requirement, called the "Strong

In contrast with the concrete Cypress section of the Nimitz Freeway, which will take more than three years to repair, the failed steel spans of the Bay Bridge were quickly replaced and the bridge opened to traffic just one month after the earthquake.

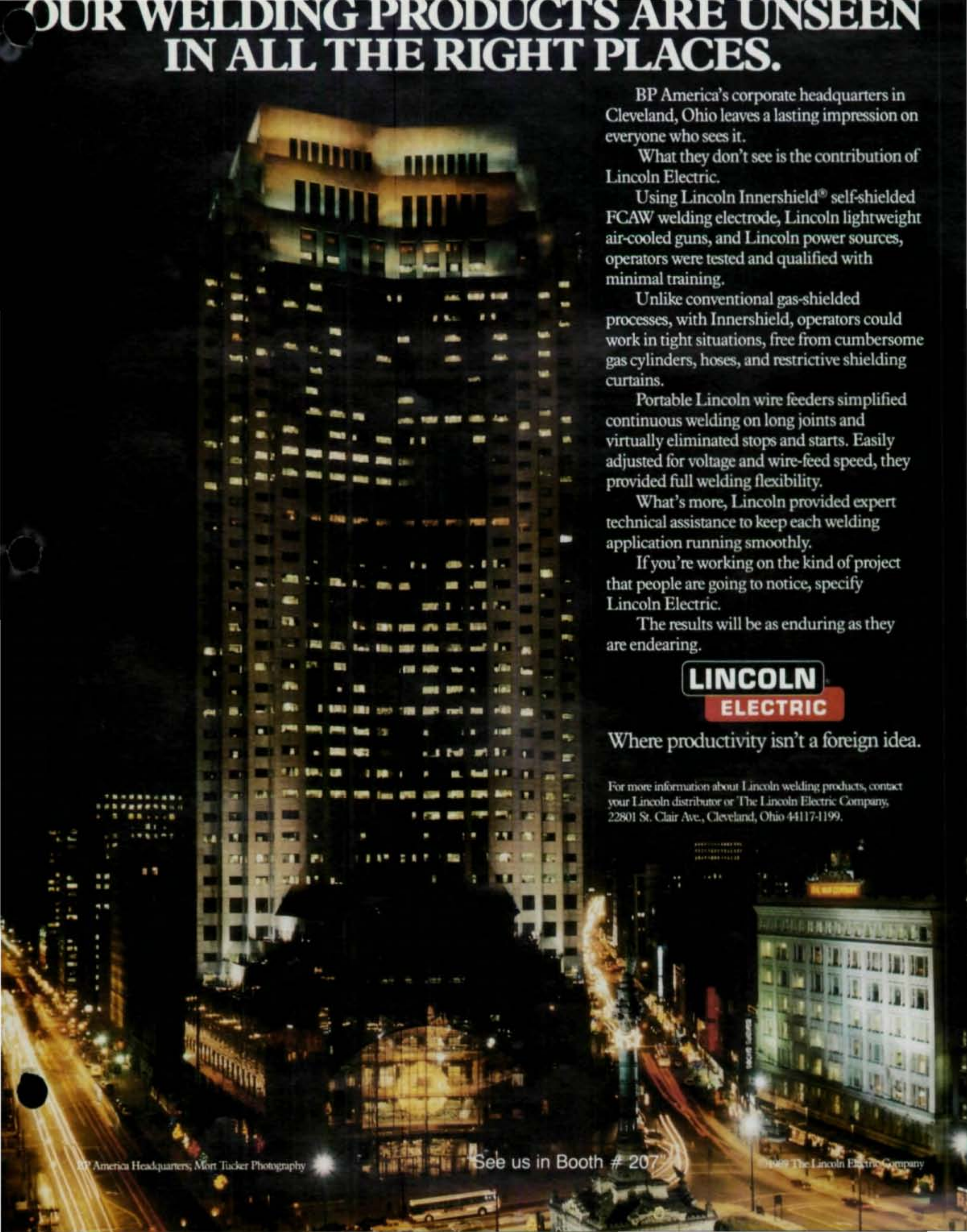
Column, Weak Beam" Theory, requires that if yielding occurs due to seismic or other lateral forces, the yielding should occur in the beam, not the column, and thus preventing possible collapse of the structure.

The "Strong Column, Weak Beam" requirement does have some exceptions depending on the amount of the axial force in the column, but few responsible structural engineers on the West Coast will deviate from the requirement that the beam yields, not the column.

In the Cypress section, the hinges were placed at the bottom of the upper columns, but above the lower deck roadway. The connection between the upper and lower columns consisted of four #11 dowel bars extending 24" above and below the joint. The reinforcing steel in the lower deck roadway was bent down to serve as reinforcement for the lower columns, leaving approximately 3' of lower column above the lower roadway and below the hinge joint without any continuous reinforcement.

As the structure lifts and falls due to vertical acceleration, forces from the earthquake result in a high com-

Continued on page 14



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Quake, Cont.

pressive load concentrated below the hinge in the region of discontinuous reinforcement. As a result, a downward diagonal shear failure of the lower concrete column just below the hinge occurs and the structure collapses.

In instances where bents were skewed with the roadway or for some other reason were required to be longer, the girder supporting the upper roadway deck was post-tensioned in order to keep all the girders a constant depth and maintain vertical clearance above the lower roadway.

The Bay Bridge

The Bay Bridge is an 8.3-mile long collection of structures consisting of suspended spans, cantilevered spans, and deck trusses. The reason for the mix of structures is the variety of sub-soil types. The West Bay suspended spans are founded on bedrock, while the East Bay cantilevered span and deck trusses are founded on dense sands.

While all segments of the bridge showed signs of movement, the only major damage occurred at Pier E9 when 14,290' deck truss spans moved about 7". Pier E9 is a four-column braced steel truss system. The two west columns support the east ends of two 506' span trusses, while the two west columns support the reaction of two 290' deck trusses. The distance between the east and west columns is 50'.

During the earthquake, when the trusses moved eastward about 7", they pulled the steel stringers with them. The steel stringers were supported on seat angles with only a 5" outstanding leg. This caused the steel stringers of both the upper and lower decks to be pulled off their supports resulting in the west end of the upper deck dropping to the lower deck. The west end of the lower deck came to rest on a transformer station, which was located under the lower deck. This transformer station prevented the decks from crashing down onto the Pier and causing serious damage. The Bay Bridge was repaired and opened to traffic a month after the earthquake.

Additional Damage

Other damaged, closed-to-traffic structures include the Embarcadero Freeway I-480 and the Southern freeway I-280 north of Highway 101. These are all double decked freeways of reinforced concrete construction and contain details similar to the Nimitz. They are scheduled to be retrofitted and strengthened with steel plate jacketing around the piers. About 9,000 tons of steel will go into this retrofit program, which was designed by a consortium of consultants including: Bechtel; PBQD; Tudor; CH2M-Hill; T.Y. Lin; and DeLeuw Gather. The initial plan is to have these freeways reopened by late Spring.

The detailing of the hinge joints for the Embarcadero and Southern Freeways was slightly different than that of the Cypress section. The Embarcadero and Southern Freeways, both of which were built later than the Cypress, had the hinges located at the level of the lower roadway, thus eliminating the region of discontinuous reinforcement.

Rudolph Hofer, Jr., is the San Francisco regional engineer with AISC Marketing, Inc. Information for this article was obtained from: testimony given before the Assembly Transportation Committee by J. David Roberts, Rogers/Pacific, Inc.; the proceedings of the Senate Transportation Committee, Nov. 1, 1989; and the Preliminary Report to the Governors Board of Inquiry by Astaneh & McCracken, University of California at Berkeley. □

Calendar

National Steel Construction Conference—March 14-16. Conference and trade show sponsored by the American Institute of Steel Construction. Kansas City Convention Center. Contact: AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001 (312) 670-5432.

National Computer Graphics Association—March 19-22. Anaheim Convention Center. Contact: NCGA, 2722 Merrilee Dr., Suite 200, Fairfax, VA 22031 (800) 225-NCGA.

American General Contractors Convention and Exhibition—March 15-20. Moscone Center, San Francisco. Contact: Rick Brown, AGC, 1957 E. St., NW, Washington, DC (202) 393-2040.

American Subcontractors Association—March 21-25. Sheraton Harbor Island, San Diego. Contact: ASA, 1004 Duke St. Alexandria, VA 22314-3512 (703) 684-3450.

Steel Plate Fabricators Annual Meeting—April 22-25. Ponta Vedra Inn & Club, Ponta Vedra, FL. Contact: Steel Plate Fabricators Assoc., 2400 S. Downing Ave., Westchester, IL 60154 (708) 562-8750.

AWS Welding Show—April 24-26. Anaheim Convention Center. Contact: American Welding Society, 550 N.W. LeJeune Road, Miami, FL 33126 (800) 443-9353.

National Steel Erectors Association—April 24-27. Loews Ventana Canyon Resort, Tucson. Contact: NEA, 1501 Lee Highway, Suite 202, Arlington, VA 22209 (703) 524-3336.

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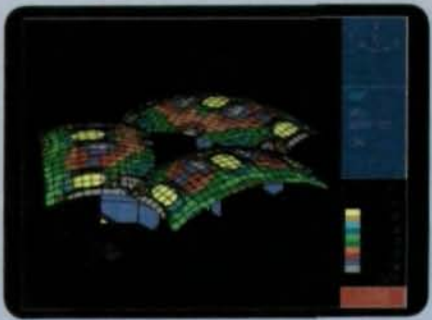
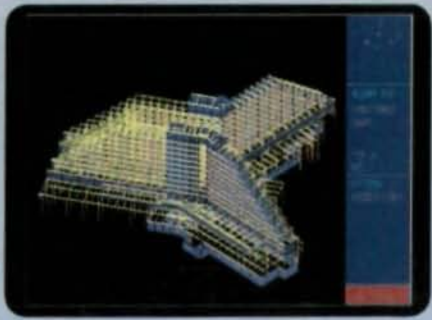
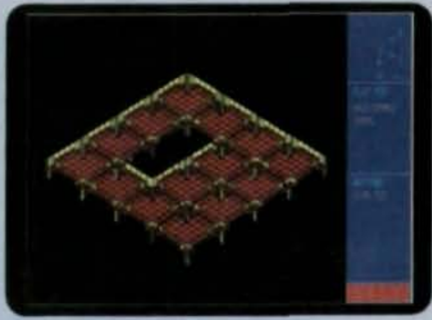


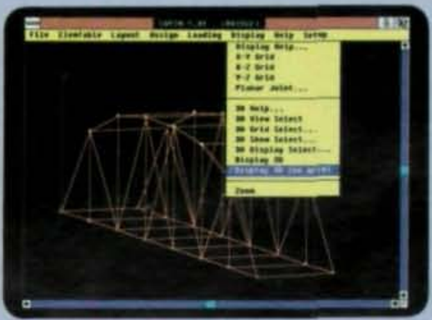
Shell Analysis



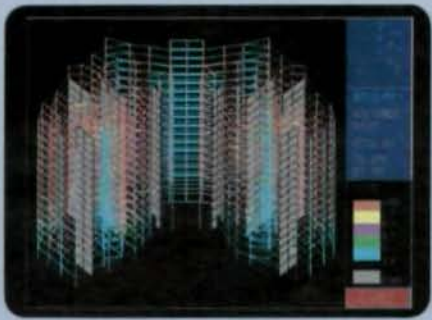
Concrete Building Design



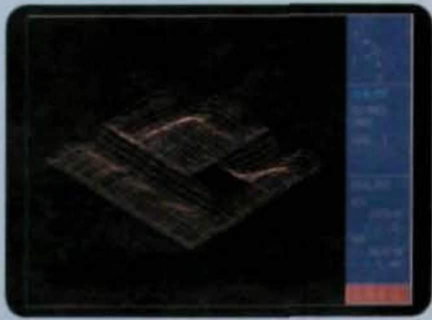
Slab Design



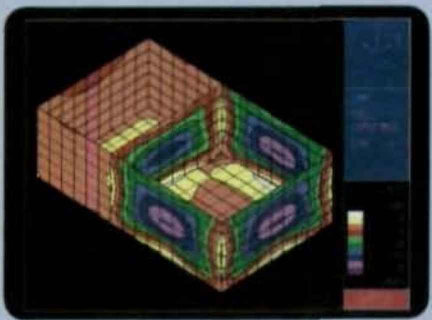
Bridge Analysis



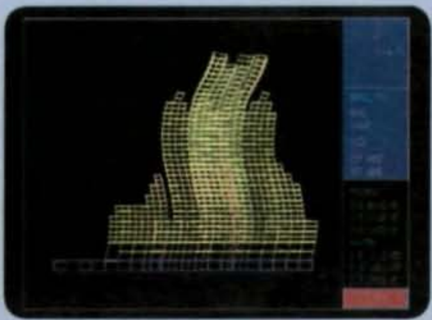
Steel Building Design



Slab Deflection Analysis



Tank Analysis



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The seminar program, which visited 30 cities last year, is geared for use with the new Ninth Edition Manual of Steel Construction from the AISC.

The lectures cover the use of the newly created composite beam tables, beam load tables, small base plate analysis, shear tab connections and, for the first time, semi-rigid connection design.

Also included are updated procedures on traditional steel design methods, such as beam web analysis, moment connections, fatigue and eccentric joints. The seminar also will cover the 1989 AISC Specification, the rules of which have been reframed into a chapter format following the LRFD logic to provide correlation between both Specifications.

Seminars are scheduled in the following cities: New Orleans (Feb. 6); Denver (Feb. 7-8); Frankfurt (Feb. 22); Burlington, Vt., (Feb. 26); San Antonio (Feb. 27); Albany (March 2); Greenville (March 20-21); Boise (March 21); Salt Lake City (March 22); Las Vegas (March 24); San Francisco (March 26-27); Indianapolis (March 26-27); Maim (March 28-29); Harrisburg (April 3); San Juan (April 4); Pittsburgh (April 4-5); Springfield, Ill. (April 10); Memphis (April 10-11); Des Moines (April 11); Nashivlle (April 16-17); Knoxville (April 18-19); Syracuse (April 23-24); Charlotte (April 23-24); Rochester (April 25); Raleigh (April 25-26); Buffalo (April 26); Norfolk (May 7-8); Richmond (May 9-10); Oklahoma City (May 21-22); and Albuquerque (May 23-24).

To register, return the format at left, along with payment, to: AISC-ASD Lectures, P.O. Box 806286, Chicago, IL 60680-4124. Or, for more information, call (312) 670-5422.

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Williamsburg Bridge Set For Rehabilitation

Contracts are expected to be signed soon on the largest bridge repair project in New York's history—the $400 million Williamsburg Bridge rehabilitation.

Design studies are now underway on the massive project and contracts will soon be let. "Both approaches will be replaced and the suspended and side spans will be replaced," said John Lopuch, project chief engineer with Steinman Boynton Gronquist & Birdsall, consulting engineers.

The suspended bridge's main span, which is scheduled for rehabilitation, is 1,600'. The two side spans are 596' each. The two approaches are 2,648' and 2,006', and both will be replaced. "The rehab work will be steel, and we're currently studying what kind of material to use on the approaches," Lopuch said.

The Williamsburg Bridge, which opened in 1903, is scheduled to undergo a $400 million rehabilitation –the largest in New York's history—to be completed in 1998.

The eight lane bridge was opened in 1903, and in addition to auto traffic has two New York City Transit train tracks. During the renovation, six lanes will be kept open at all times, while the transit tracks will be moved to a temporary trestle alongside the existing bridge.

The design phase is expected to be completed by 1993, and construction is scheduled for completion in 1998. The first construction contract, for cable rehabilitation, is expected to be awarded this September. □

State DOTs Increasingly Require CAD

More than half the State DOTs surveyed in a new 270-page study require CAD use on work done by outside engineers or enforce some kind of CAD standard on work done under contract.

The survey, *CAD DOT, The Design Systems Strategies Guide to CAD Activities at Departments of Transportation*, includes information on DOTs in all 50 states. The study is designed to aid engineering firms competing for design contracts by providing information about brand compatibility, electronic deliverables, direct fee and overhead calculations for CAD expenses, and CAD requirements such as layering, lettering and symbol library conventions.

According to the survey, Intergraph is the market leader with 36 of the agencies surveyed reporting Intergraph installations. AutoCAD and McDonnell Douglas GDS were the next most popular systems. While three agencies reported no CAD installations, 14 agencies reported using more than one brand of CAD, with Intergraph and AutoCAD being the leading combination. The states with the most workstations are California (703), Texas (404) and New York (230).

Roadway design and maintenance is the leading installed application, with bridge design and maintenance a close second. More than half the agencies require CAD some CAD use, and another 20% are developing similar policies to be implemented soon.

A complete copy of the directory is available from Design Systems Strategies. For more information, call 1-800-CAD-NEWS or write Design Systems Strategies, P.O. Box 20, Scarborough, ME 04074. □

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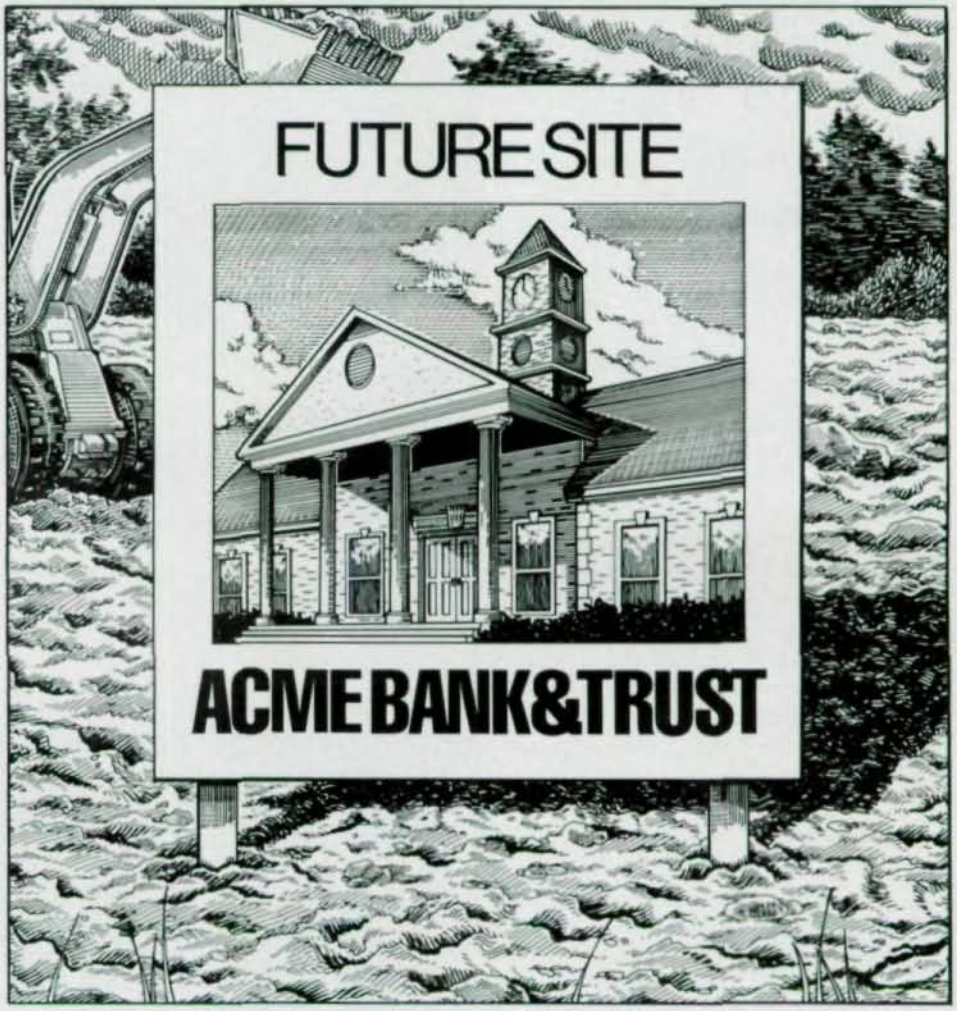

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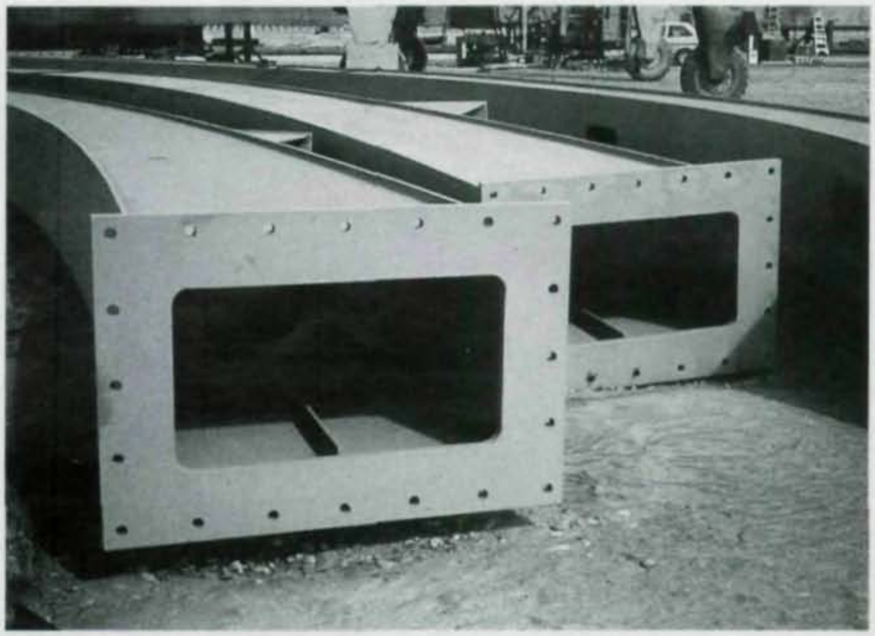
Weldable Primer Improves Bridge Fabrication

The use of a thin pre-construction primer prevents rusting prior to fabrication

By Tom Calzone



Welding plates into box arches results in minimal primer "burn back" (left). Spot surface preparation to welds is required on about 5% of the total surface area. The pre-construction primer provides good corrosion protection inside the boxes (above). Additional coatings may be applied, if desired, without the need for surface preparation.

A technique similar to the one that catapulted the Japanese into the forefront of shipbuilding was recently employed on two Colorado DOT projects to cut costs in the pre-fabrication stage and substantially improve the paint system quality.

A thin coat of inorganic zinc pre-construction primer was used on the projects to prevent rusting of blast cleaned steel during fabrication without interfering with cutting or welding quality and speed. In addition, the pre-construction primer improved the quality of the project by simplifying the required manual blasting.

Manual blasting too costly

The projects—the Sixth Ave. flyover, a tub girder, and the Speer Boulevard suspended arch bridge—benefitted from the use of a weldable primer because the configurations of fabricated tub girders and arches makes it impossible to use automatic abrasive blasting equipment effectively. Manual blasting of the assembled pieces is inefficient, very costly and creates a variety of problems for the fabricator. In addition, the Environmental Protection Agency has voiced concerns about dust generation from blasting operations in the yard.

Continued on page 22

Weldable Primer, Cont.

Pictured at left is a fabricated tub ready for spot blasting and the application of a full primer coat. The rust stain spreading from the exposed weld bead does not inhibit the adhesion or corrosion protection provided by the primers.

Pre-construction primers have a dry film composition similar to the more familiar inorganic zincs used by many fabricators for bridge work. The key difference, however, is the ability to apply pre-construction primers in a thin film with a relatively close tolerance on thickness due primarily to their low solids content and because they are typically applied to large flat plates rather than fabricated pieces.

"The Japanese shipbuilding industry initiated the use of weldable inorganic zinc primers to facilitate block construction in the early 1950s," said John Peart, the research chemist responsible for FHWA research on bridge coatings.

Excellent surface preparation

In block construction, steel panels are abrasive blasted with automatic equipment, primed, welded into units, partially assembled and outfitted and then placed into the hull. This procedure eliminates a great deal of laborious surface preparation otherwise required after the ship is erected. Many areas really couldn't be properly blast cleaned after assembly.

With block construction, these areas received excellent surface preparation and a zinc rich primer. The same principal applies to many modern bridge designs. "The use of block construction is one of the primary reasons Japan became the leader in shipbuilding," Peart said.

Bob LaForce, senior highway engineer with the Colorado DOT, approved the use of an inorganic zinc primed system based on the "excellent corrosion protection reported by other states usage and test data." Experience has shown that large plates are easily blasted and rapidly primed with a high degree of quality control.

The pre-construction inorganic zinc primer is applied much faster than conventional high build inorganic zinc primers can be applied. The pre-construction primer—Carbo Weld 11 from the Carboline Company—was applied in a thin film of 0.75 to 1.0 mils in an assembly line blast/prime operation to plates, stiffeners and brackets. The primed pieces were then shipped to the fabricator for assembly.

After fabrication, the fabricator performed spot blasting on welded areas utilizing a pencil blaster that provides a narrow blast pattern. The area requiring blasting comprised approximately five percent of the overall surface. A full prime coat, in accordance with the Colorado DOT specifications for a three-coat alkyd paint system, was then applied to ready the steel for shipment.

Reduction of problems

The fabricator benefitted from the elimination of problems and from improved quality, while the owner benefitted from outstanding corrosion protection. Other advantages include:

- Blast cleaning costs are greatly reduced;
- Blast cleaning before welding eliminates the possibility of mill scale contaminated welds and therefore there are few rejections;
- Flash rusting is eliminated, reducing concerns of rust contaminated welds;
- The inorganic zinc primer is very resistant to damage and keeps steel rust free during fabrication even on nicked and gouged areas;
- Faying surfaces are protected from rusting and may be bolted without blast cleaning due to high friction values of the inorganic zinc.

Procedure Qualification Tests on A-588 steel are currently being performed with weldable primer. By qualifying procedures on A-588 steel for fillet welds, the procedures will be acceptable for groove welds, and for A-36 and A572 steel as well. The appropriate physical and chemical tests also are being performed to qualify the procedures for fracture critical structures.

Full use of modern coatings technology need not add to the cost of steel construction, and may actually lower the initial and lifetime costs of steel structures.

Tom Calzone is the representative of the Steel Structures Painting Council to the Council for the Advancement of Steel Bridge Technology and highway market manager for the Carboline Company. □

Inverted Library Hangs From Roof

Suspending the floors from the roof created column-free space and maximized the usable area

The steel frame for the new MIT Library of Architecture (opposite and left) serves a dual purpose: form and function. Because the perimeter columns are 15'-on-center, there was inadaquate room for trucks to enter and turn around beneath the library. To accommodate the trucks, two of the columns were split at the fourth level and form an "A" shape. Photography by Curt Berner. Rendering by Don Paine.

The phrase "just hanging around" will take on a whole new meaning when the MIT Library of Architecture is completed this fall. Due to unusual site and size constraints, the building's structural system is essentially inverted, with the six floors "hanging" from the top, rather than being supported from the bottom.

The $5.5 million, 22,000-sq.-ft. addition is being built within the L-shaped crook of a larger building which houses the current 9,000-sq.-ft. library on two of its five floors. The space that the addition will occupy is part of a service courtyard for five surrounding buildings.

Truck access maintained

Because truck access could not be hindered, the bottom floor of the building had to be elevated 18' above the ground. "There is no ground level," said Ruben Morrison, project manager with George B.H. Macomber, the project's Boston-based general contractor and construction manager. "The ground level is a service courtyard for trucks to turn around. You can visualize it as being built above the air rights to a turnpike."

An additional problem was created by the need to fit as much area as possible into the limited available space. To satisfy local code officials, the building's height couldn't exceed that of the connecting building. That meant limiting structural columns and squeezing the floor-to-ceiling height as much as possible.

The solution developed by Simpson Gumpertz & Heger, Arlington, Mass., the project's structural engineers, to the space constraint problems was to suspend the library's six floors from the roof to provide a column-free interior. Design engineer was Susan O'Dell and senior principal on the project was John W. Nevins, P.E. The projects architect was Schwartz/Silver Architects, Boston.

"Because the structural system of the addition is very different from that of the existing building, it was not connected to it," Morrison explained. Instead, two parallel lines of steel columns, the first about 6' from the existing building and the second approximately 36' from it, were erected. The two buildings are connected with a glass-enclosed atrium.

The columns are each supported

on clusters of piles separate from the existing building's foundation. The column erection was difficult, however, since the columns weren't tied together and therefore weren't self-supporting.

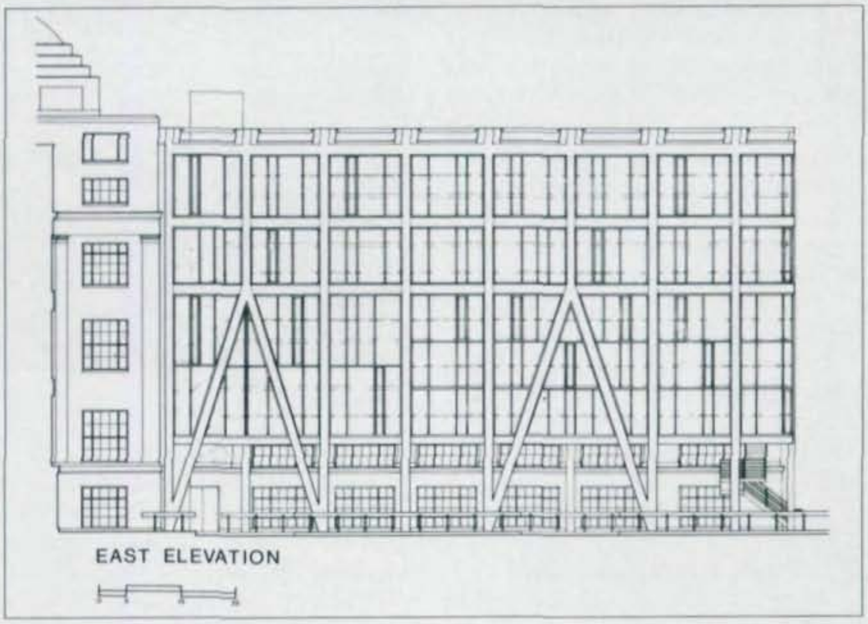
"We needed to erect a temporary network of structural steel to support the columns until the concrete slabs were poured," Morrison said. "There are approximately 3-1/2 floors of X-bracing." The temporary bracing is composed of 5" x 5" steel tubes.

"In addition," he added, "there are so many cables going in so many directions to temporarily brace the building that the formwork for the concrete floor slabs are penetrated 135 times." As part of the almost every subcontract is a clause that prohibits each trade from moving or adjusting the temporary cables in any way.

The structural columns extend to the roof, and each of the seven column lines carries a pair of W36 x 300 steel girders. Three steel straps are hung from each pair of girders, and at each level where a slab occurs, there are shear heads to support the slab. The hangers are 1" x 16" steel plate, and when construction is complete, they will be fireproofed and boxed with drywall. The hangers fit between bookstacks so will not consume valuable floor space.

The hangers are set 6' from the perimeter of the slab, which is cantilevered. "We couldn't attach the floor slab to the columns for vertical load because the perimeter columns are rigid and don't deflect," explained O'Dell. When the books—which have twice the weight of the slab—are added, the cantilever will deflect and a rigid connection could cause cracking. "Instead we used a sliding connector and the slab is only connected to the columns for lateral load."

The first connections will be made 28 days after the roof slab is poured. For the exterior columns,



The entire structure is supported on hangers suspended from pairs of W36 x 300 roof girders (top). The diagram above shows the split of the vertical columns into "A"-shaped members called "rakers" by the building team.

Pictured above is a closup view of the large W36 x 300 girders as they were installed. The girders support hangers, which in turn support the floor slabs.

the slab and column are connected with a single bolt. The interior columns are surrounded by a band which allows for deflection and movements.

The slabs are 7 1/2" thick, which allowed a floor to ceiling height of 8'-1 1/2".

Further complicating the project was the need to allow trucks to pass under the building. Because the columns were 15'-on-center, two of the columns had to be split to provide vehicular access. The columns were split at the fourth level and diagonally braced to transfer the load to the adjacent columns. The two "A" shaped members were referred to as "rakers" and are designed as compression columns and provide a lateral load resisting frame in that area, according to O'-Dell.

Steel erection was recently finished and the construction should be completed in May. The erector was Dorel Steel Erection, and the fabricator was Montague-Betts. The columns will be clad in aluminum, and the exterior of the structure will be a glass and aluminum curtainwall.

"We really wanted to express the structure," said Ann W. Pitt, AIA, the project architect with Schwartz/Silver. "The building is a celebration of the school's architecture collection." □

Back To The Future

The Home Savings Tower utilizes the state-of-the-art versatility of steel to allow the return of a classic expressionism

Not long ago state-of-the-art design meant exposing a building's structural system for all the world to see. Today, however, some designers are rediscovering the techniques of the sculptor: they're using the steel frame not as an architectural statement, but as a framework to support architectural elements.

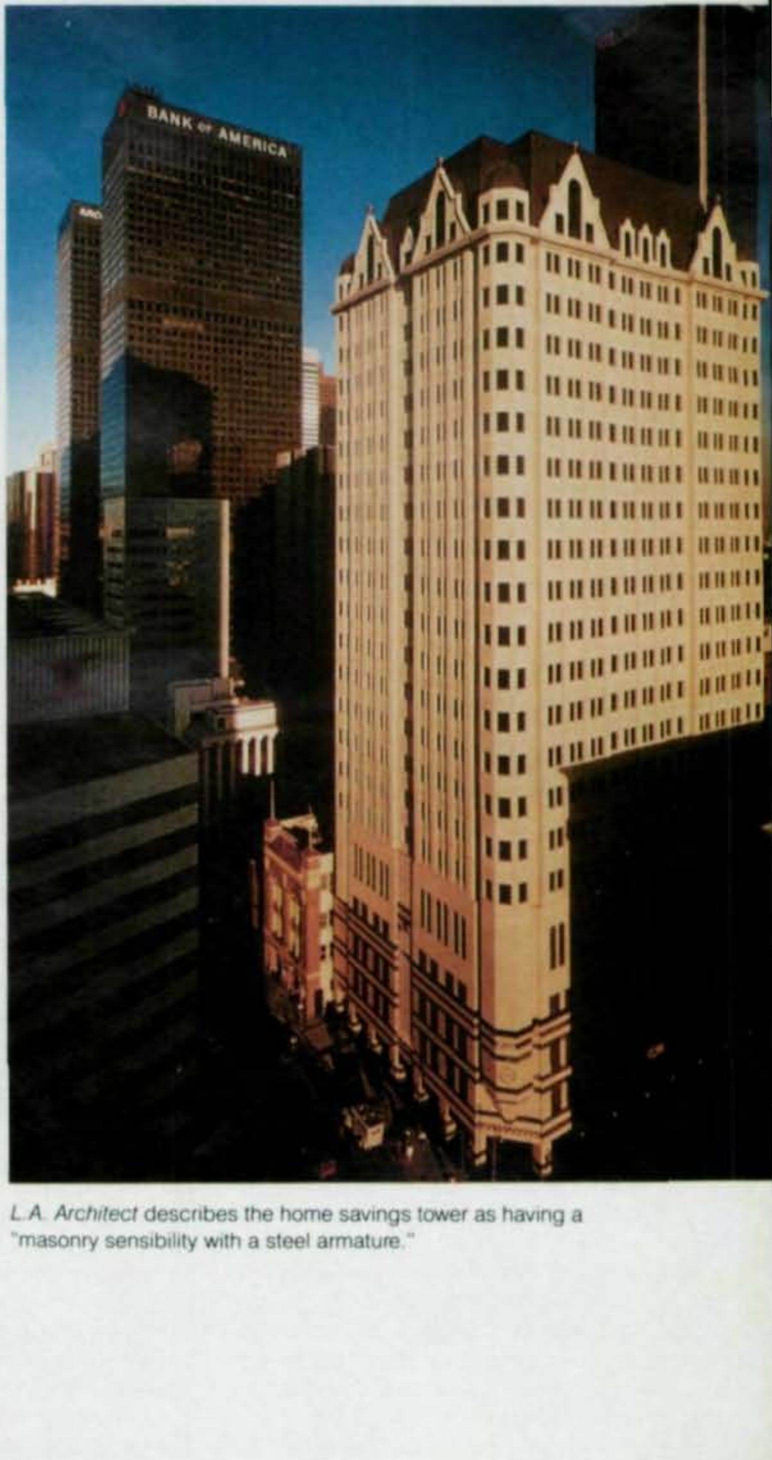
But instead of lessening the importance of the structural system, this style of design is magnifying it. Just as wire and rods are needed to support the extremities of a sculpture, a steel armature is required to buttress complex detailing.

In the recent past, Modernism seem0ed just that: Modern. Now, the move is away from Modernism's coldness towards recapturing the solidity, romance and sculptural quality of the stone tower, but without giving up the versatility and utility of steel.

The Home Savings Tower in Los Angeles demonstrates the new look perfectly. As *L.A. Architect* magazine states, it has a "masonry sensibility with a steel armature."

"When most people think of architecture in Los Angeles, they think of Modern office towers and idiosyncratic buildings influenced by Hollywood," explained Tim Vreeland, FAIA, project designer with Albert C. Martin & Associates, Los Angeles, the building's architect and engineer. "But L.A. had a period in its history, from about 1890 to 1940, when some solid, excellent commercial buildings were built."

Compared with the deserts of plazas that stretch between buildings in the new Bunker Hill section, the area around Seventh Street—L.A.'s old urban core—is



L.A. Architect describes the home savings tower as having a "masonry sensibility with a steel armature."

more of a traditional city, "with buildings tightly packed to the sidewalk and a distinctive facade at steet level," Vreeland said. "The Home Savings Tower, located at the intersection of Seventh and Figueroa Streets, was an ideal opportunity to refer back to that heritage."

Vreeland chose to design the building in the French Chateau Style similar to many older buildings in New York City. "What Nabih Youssef (P.E., director of structural engineering with Albert C. Martin) taught me was that the steel skeleton doesn't have to conform to the outside skin of the building," Vreeland said. "It doesn't have to be the basis of the architecture."

The architect of the Home Savings Tower in Los Angeles chose to design the building in the French Chateau Style to help recapture the positive feeling created by the architecture of the past. The project's structural system was complicated by the need to slope the steel columns at the sixth (see below) and the 24th floors. Construction photography by Annette Del Zoppo.

"Returning to masonry construction would be patently absurd, but adopting a masonry sensibility affords all the advantages of steel construction."

Freeing the skin from the structure allowed Vreeland to get away from designing a glass box and instead allowed him to create a steel-framed building with a masonry appearance. While a glass curtainwall allows the structure to show through, Vreeland modernized the past by hanging marble, precast concrete and ceramic tile on a steel frame to recapture the positive feelings evoked by pre-turn-of-the-century architecture.

The concept relates back to Gustav Eiffel's work with the Statue of Liberty. While the great sculpture has a steel armature, it reflects a masonry sensibility—albeit in copper—with most of its sculptured interest—the torch, the spiked crown, etc.—saved for the top. "Returning to masonry construction would be patently absurd, but adopting a masonry sensibility affords all the advantages of steel construction," Vreeland said.

In the case of the Home Savings Tower, which was developed by Ahmanson Commercial Development, Los Angeles, steel's flexibility was crucial to the building's concept—which included both above-grade parking and office space.

The steel frame slopes inward at the sixth floor and again at the 24th. In addition to providing the aesthetic advantages that Vreeland desired, the slopes reflect the various functions of the building.

The structure is actually built atop a new Metro subway station, which meant that parking could not be put below ground. Instead, the first five floors are essentially a carefully disguised parking garage. Above that, however, is office space.

"Starting at the base of the building, this is structurally a departure from traditional steel towers," said Youssef. "Because of the Metro rail, we had to have a concrete base and start the steel just below grade instead of at the foundation." The use of a concrete base was mandated by the L.A. Rapid Transit District, which required a concrete mat foundation to resist hydrostatic pressure and eliminate the need for sump pumps to maintain a dry space. To minimize trade conflicts, a concrete foundation was the logical extension from the concrete mat.

Seismic and wind shear forces were transferred at the plaza level to concrete ductile columns at the perimeter of the building. "The steel columns go below the plaza 5' to 6'," Youssef said. "Lateral fixity is provided by horizontal beams in this area that fix the rotation of the columns." The perimeter steel beams, which are 36" in depth, are welded to the steel columns. The steel base plates are connected to the concrete pilaster with anchor bolts and grout.

Because the first five stories are a parking structure, the columns were pushed to the perimeter to minimize interference with vehicles. "When we got to the skylobby (on the sixth floor), there was a transition of these columns to fit the office space function," Youssef said. Rather than making an abrupt transition in the ductile frame, the decision was made to slope the frame and build up outriggers to pick up the vertical cladding—marble at the base and precast concrete above.

Complicating the project, however, was the need for the columns to slope in two directions. "There's a setback, but there's also some lateral slope based on the office layout," Youssef explained. Because they slope in more than one direc-

A computer-generated rendering of the Home Savings Tower's structural system reveals the positioning of the sloping members (left). The building's base (above) is designed to complement the urban nature of Seventh Street. The lobby (top) reflects the traditional nature of the building's exterior.

tion, double beveled cuts were required, which meant very tight controls in the fabrication stage. "When the gaps at the column splice were too large, they had to be built up with welding," he said.

The sloping structural steel was complex enough that the fabricator created a mockup of the angles prior to fabrication, according to Roger Maki, project manager for Swinerton & Walberg Co., Los Angeles, the project's general contractor.

The columns slope 2'-6" per story, and some slope through just one story, while others slope through the entire two-story skylobby. The sloping columns are not limited to axial load in one direction and still work as sloping members within a rigid frame.

"They form the new plate for the office space, and then they go straight up to the penthouse level where they sharply slope in to form the mansard roof," Youssef said.

"The columns above the sixth floor are moved 5' back into the building. It allowed me to cantilever some bays, while inverting others, and it allowed me to have rounded cornices," said Vreeland. The 42"-in. wide cornice at the top of the building is both decorative and functional. "Window washing equipment is suspended from the cornice, and it provides a ledge for washing the windows above, on the sloping portion of the roof. Also, it provides a mount for lighting the roof at night," he said. The copper roof slopes sharply and has oversized, 20'-high windows. Some of the space is used for executive offices and meeting rooms, while the rest of the roof conceals the mechanical equipment.

"The conventional wisdom is that you need to have closely spaced columns on the periphery of the building," Vreeland said. "But that's not sacrosanct. You can move the columns back so you can get any shape you want on the exterior of the building." □

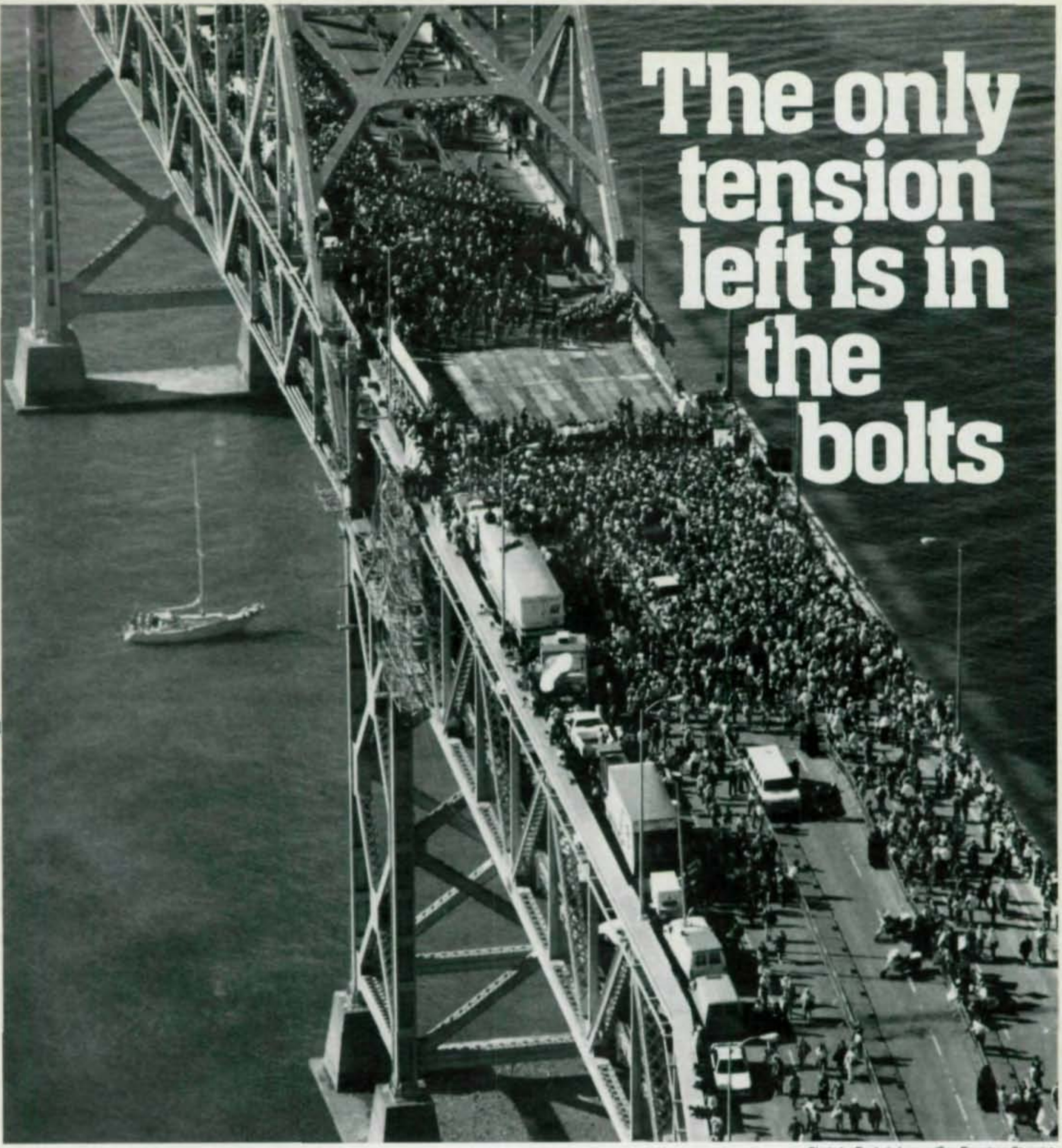


Photo by Frederic Larson/San Francisco Chronicle

CALTRANS-specified Direct Tension Indicators facilitate Oakland Bay Bridge reconstruction.

Less than one month after October's 7.1-magnitude earthquake struck northern California, a crucial traffic artery was reopened for use by the public. The California Department of Transportation (CALTRANS) facilitated reconstruction of the collapsed section of the Oakland Bay Bridge by specifying use of Direct Tension Indicators (DTIs) from J&M Turner.

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Complex Supports For Steel-Girded Skyway

Steel was a less costly option than concrete and still allowed the structure to complement a nearby existing skyway

A new pedestrian walkway in Rochester, N.Y., hooked two large department stores into the city's elaborate latticework of skyways. The 96'-8" pedestrian bridge was designed in steel after it was determined that a concrete bridge would not be completed on time or on budget.

By John E. Isbell, P.E., and Ted Krol

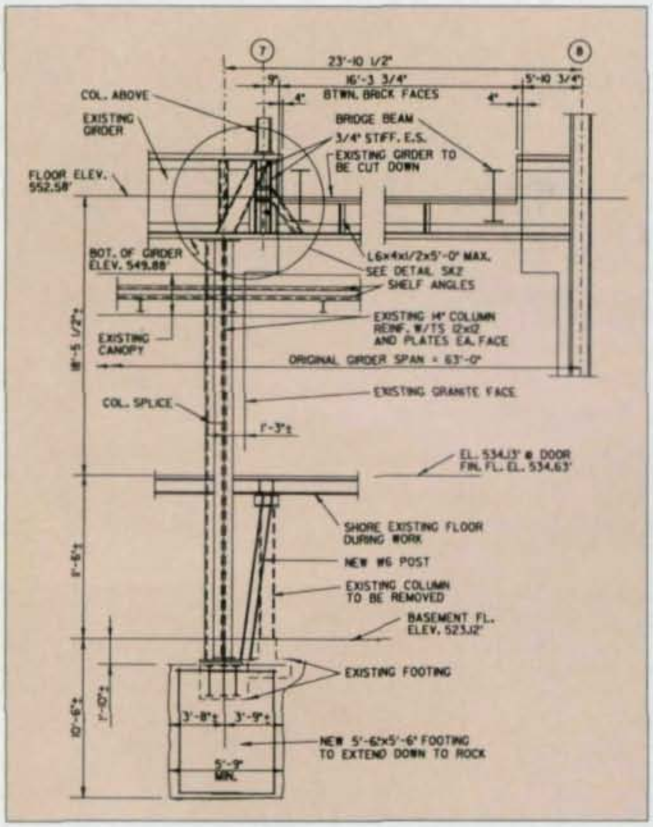
During the 1920s and '30s, some architectural visionaries imagined the future city as a place of tall buildings connected at varying levels with pedestrian "skyways."

While no city has created the complete linkup that these early 20th century urban planners imagined, many northern cities have constructed elaborate skyway networks connecting office, retail, and residential buildings.

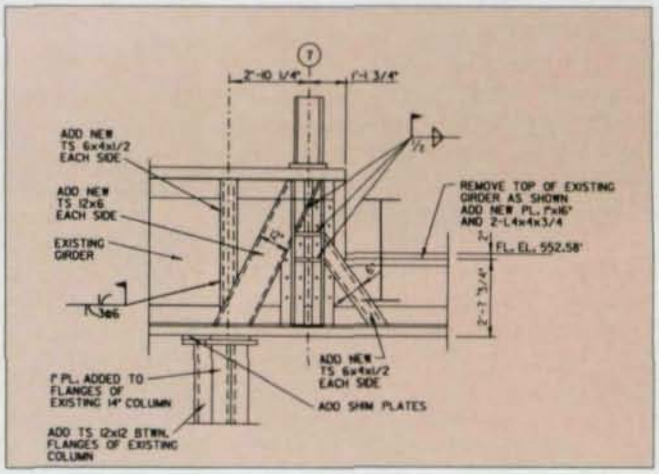
In Rochester, N.Y., a major gap in its latticework of skyways was recently filled by the completion of 96'-8" pedestrian bridge between two major downtown department stores, Sibley's and McCurdy's.

Aesthetics prove crucial

Of utmost importance for the clients—who included the two department stores and the city—was the skyway's aesthetics, since it would be a major architectural feature on Main Street in the city's Central Business District. The original proposal was for a concrete bridge that would closely match a nearby precast skyway in appearance. However, after the project was bid, the Canadian supplier indicated there would be problems related to shipping and availability of



Shown at left is the south elevation of the new pedestrian skyway illustrating girder removal and column foundation reinforcing. The diagram above is a detail of the circled area on the south elevation and illustrates the necessary girder cutting during construction.



the major unit of precast concrete within the necessary time frame. As a result, the cost of using precast concrete increased substantially.

The Sear-Brown Group, the project's architect and structural engineer, advised their clients to reevaluate their initial concept. Instead of a concrete bridge, Sear-Brown proposed a panel-clad steel bridge. After consideration of the availability and delivery advantages of using steel, a redesign was completed on a negotiated basis, bringing the cost of the skyway construction down $60,000 to $550,000.

Another advantage of steel was that the girders could be readily spliced. The 96'-8" span was broken into two pieces, facilitating transportation and erection of the girders for contractor David L. Christa Construction, Inc., and its steel subcontractor, F.L. Heughes and Company, Inc., both of Rochester.

The most challenging aspect of the project, however, was not the design of the skyway, but rather its support system. At the Sibley's end, the existing building could not support the bridge and introducing new columns near the building's exterior would interfere with the store entrance. Fortunately, the sidewalk was wide enough to place a concrete bent near the street and the steel girders cantilevered from there to the building.

Reinforcing the column was complicated by numerous beams and relieving angles that framed into it.

At the McCurdy's end, a steel transfer girder at the second-floor building wall projected about 3' above the second floor line. This 63' span girder carried two major perimeter columns. A 16' x 3' segment of the girder had to be removed to allow the bridge to connect at the proper elevation. Since this modification to the existing girder was required, Sear-Brown decided to provide additional reinforcement, so the existing structure could support the bridge.

Developing a support scheme was hampered throughout the process by the lack of structural drawings for the 1912 building. It was apparent that numerous modifications had been made through the years, yet the only drawings found were architectural details from modifications completed in 1968. Traditional research methods, such as destructive testing and investigation, were not permitted because the stores had to maintain normal operations throughout the project. However, with careful investigation in accessible areas, the engineers uncovered enough information to develop a scheme, though some data required verification during construction.

The support scheme called for extending an existing W14 column, which had been introduced during a 1968 canopy addition, up to the underside of the transfer girder, making the girder two-span continuous. The column capacity was upgraded from about 400 kips to a total load of 1,300 kips by adding a tube section between the column flanges on one side of the column and adding flange plates. Since the column footing also was deficient, the column was shored, the old footing was removed, a new footing was added about 10′ below the basement on bedrock and the new column was seated on the new footing.

Complex support system

Reinforcing the column was complicated by numerous beams and relieving angles that framed into it. These members were shored, cut back and reconnected after the reinforcement was in place. Since working room was extremely limited, the reinforcing members were cut into pieces, threaded through the structure and spliced into place. To prepare the column to carry its new load, design engineers had to ensure that the reinforcement was welded in place; that the new base plate was installed bearing on the new footing; that the column

Construction of Rochester, N.Y.'s newest pedestrian skyway was facilitated by the use of steel. Because steel can be redily spliced, the 96′-8″ span was broken into two pieces, facilitating transportation and erection of the girders.

was attached to the underside of the girder; and that the existing members were reconnected.

The transfer girder consisted of two 5/8" by 60" web plates (9.5" apart) and flanges built up from two 8" x 8" x 1" angles and four 7/8" x 26" plates, all riveted together. The added support, coupled with the loss of a segment of the original girder, resulted in significant changes in the stress distribution within the girder, which then required analysis and reinforcement.

Horizontal shear trouble

Where the girder segment was removed, a new top flange was designed of angle and plate to receive the bridge girders and develop sufficient section properties for the new span. For proper load transfer to the new column, significant reinforcement in the form of tubes and plates was added to the girder web. Shears, moments, deflections and connections were all checked.

Unfortunately, it was not until after the contractor removed the building facade at the girder that a major problem was discovered. The rivet spacing in some areas was reduced to the point where the horizontal shear between flange plates and flange and web exceeded the existing capacity.

Chemical, strength and welding tests showed the 77-year-old steel to be similar to A-36. Given this and the field conditions, the engineers decided to have the plates welded together to achieve the needed shear strength.

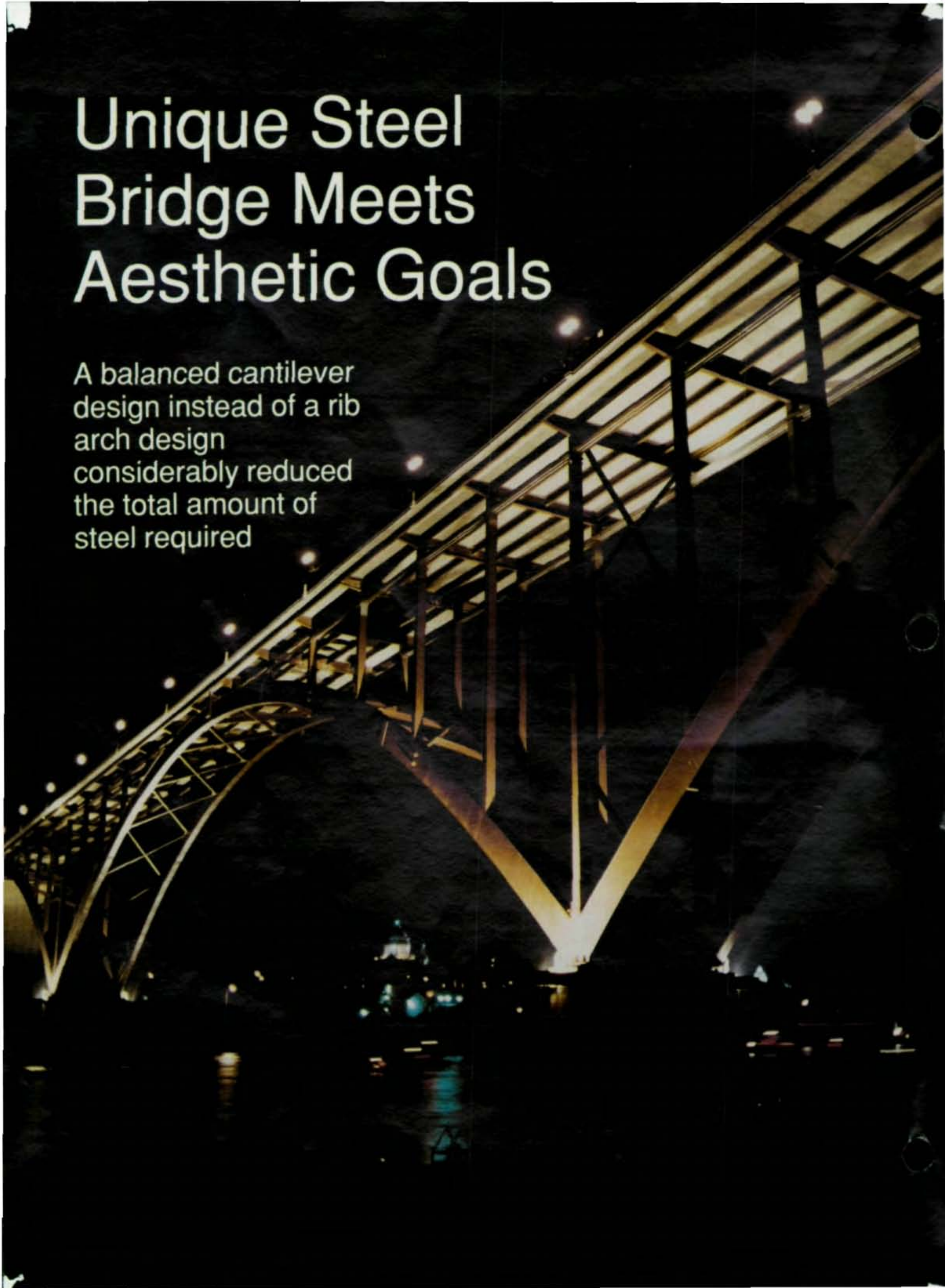
Welding of the flange plates along the edge was followed by magnetic particle testing, which revealed another problem. It appeared that shrinkage of the welds was causing lamellar tearing of the plates. If extensive, the welds would be ineffective. Since lamellar tearing was also found in plates where no welding was done, it was determined that the welding was not the initial cause but it could aggravate the problem.

To ensure a proper shear transfer, tears or cracks near the welds were ground out and rewelded if deeper than 1/8". Fortunately, many of the cracks were shallower than 1/8" and did not require welding.

John E. Isbell, P.E., is the Structural Engineering Department Manager and Ted Krol is a senior engineer with The Sear-Brown Group, a 400-person, full-service design professional firm headquartered in Rochester, N.Y.

Unique Steel Bridge Meets Aesthetic Goals

A balanced cantilever design instead of a rib arch design considerably reduced the total amount of steel required

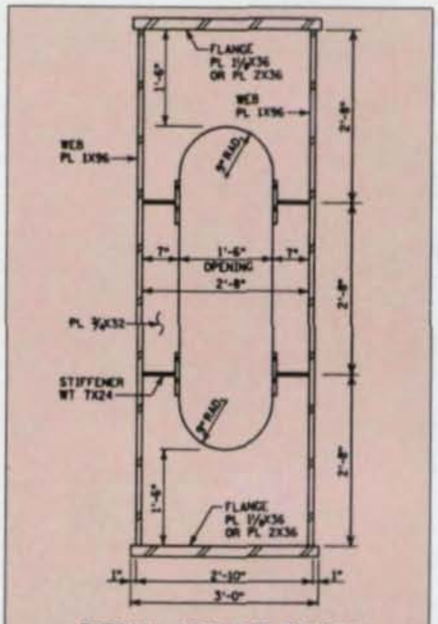


By Donald J. Fleming and Craig E. Lenz

Despite it historic status, deterioration forced the closing of the Smith Avenue High Bridge in St. Paul, Minn., in 1984. The bridge's importance to its community—both economically and socially—was such that a Citizen Task Force was formed to add significant input into the replacement choice.

The Minnesota DOT, aided by a design consultant, initially considered 14 replacement possibilities, ranging from a steel plate girder bridge to steel or concrete arches, to a steel suspension bridge. While the citizens group clearly favored a series of arches for its traditional aesthetics, it proved too costly. Instead, a compromise decision was reached in favor of a steel bridge with three balanced cantilever arch spans, including a remarkable 520' tied arch main span designed as a double balanced cantilever over each river pier.

Aesthetic concerns still played a large part in the bridges design, however. The massive river piers were painted a light tan to blend with the native sandstone outcroppings, underbridge lighting was installed to enhance the bridges prominence in the evenings, a special ornamental lighting and railing system was designed, and pedestrian overlooks were constructed.



TYPICAL ARCH RIB SECTION

(Above) A view of the unique balanced cantilevers during the construction of the Smith Avenue High Bridge. Originally, the design featured a deck tied arch utilizing a tension tie to accommodate the horizontal thrust produced by the arch. However, an analysis of arch rib forces during erection indicated that a balanced cantilever design instead of a tied arch design would reduce the arch rib depth from 12' to 8'. (Left) A diagram of the typical arch rib section. (Opposite) A dramatic night photo of the $20.1 million bridge. Photos by Neil D. Kveberg

Complex geometry

Complicating the design of the bridge was the complex geometry of the Mississippi River Valley at the crossing point. The roadway lies on a four percent grade that requires the arches be designed asymmetrically.

The multi-span, high level steel structure is 2,760' long and has a 520' span tied full arch with 280' and 240' half arch side spans. The bridge carries two lanes of traffic with pedestrian sidewalks on either side. The width varies from 54' to 68' to accommodate a median and left turn lane at both ends.

The river tied arch spans are actually designed as double balanced cantilevers over each river pier. Initially, the design featured a deck tied arch utilizing a tension tie to accommodate the horizontal thrust produced by the arch. However, an analysis of arch rib forces during erection stages indicated that a balanced cantilever design instead of a tied arch design would reduce the arch rib depth from 12' to 8' and save a considerable amount of steel. To eliminate all "fracture critical" members, a composite steel tension tie was used to provide significant structural redundancy.

The 500'-long tension ties are located below the deck and consist of a wide flange section (W33 x 118, A588) with the web in the horizontal plane and four cable tendons, two above and below, attached to the web. Each cable tendon has 19 prestressing strands.

The cable tendons carry the dead load while the rolled steel sections carry the live load. This results in significantly lower live load deflections. Another method used to reduce dead load effects was to fabricate the arch ribs slightly longer than the required span lengths, and jack them into place during erection. The horizontal shortening from the jacking counteracts the vertical shortening due to gravity.

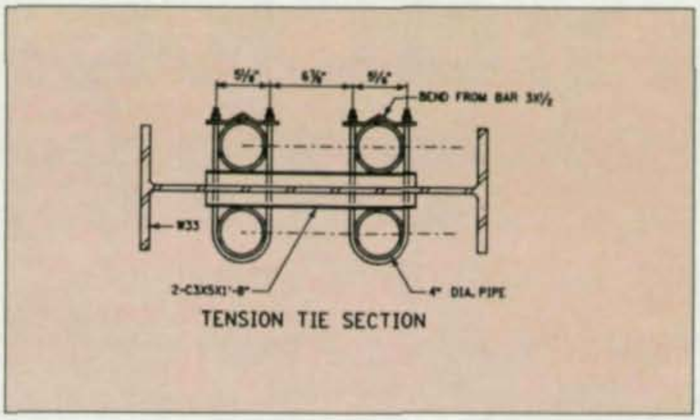
Cantilever design

The unique double balanced cantilever design of the arches also simplified the design of the river piers. Instead of requiring the piers to carry the thrust of the main arch, the thrust was transferred through the side span half arches to the tension ties to create a statically balanced structure. Both piers measure 63' x 20' x 46' high and were constructed int he dry with cofferdams. The piers are founded on steel piles driven into bedrock and are designed to withstand barge collisions and break up ice flows.

Because the bridge is built on a

The arch ribs are a welded box section with internal diaphrams and stiffeners. The deck above the arches consists of welded plate girders placed transversely at 40' centers, and the stringers spanning between them were rolled steel sections.

steep 4% grade, and it expands and contracts up to 32" due to thermal forces, the designers were concerned it might "walk downhill" with the cyclic changes in temperature. The problem was alleviated by using expansion joints only at the abutments. The bridge arches are fixed at the large river piers so the bridge moves about the main arch. The main arch deflects vertically and the tall piers under the approach spans are designed to deflect horizontally to accommodate the superstructure's thermal movements.



The tension ties consist of a wide flange section with the web in the horizontal position and four cable tendons, two above and two below, attached to the web.

Another unique aspect of the project was its erection sequence that kept the river open to navigation throughout the construction process. After the piers were constructed, the side span half arches were erected. By attaching temporary high strength bar stays to the main arch rib and to the wide flange tension ties at the top of the spandrel columns, the side spans could be used to support the weight of the spandrel columns and tension ties and act as a counterweight during the erection of the cantilevered main span arch ribs.

This article is condensed from a paper presented by Donald J. Fleming, state bridge engineer, and Craig E. Lenz, senior engineer, both of the Minnesota DOT, at The National Symposium on Steel Bridge Construction. The design firm on the project was Strgar-Roscoe-Fausch, Inc., Minneapolis, and the consultant was T.Y. Lin International, San Francisco. General contractor for the river piers was Deward Kraemer and Sons, Plain, Wisc., and for the superstructure was Lunda Construction Co., Black River Falls, Wisc. Fabricator for the approach spans was Phoenix Steel in Eau Claire, Wisc., while the fabricator for the arch spans was Vincennes Steel Corp. in Vincennes, Ind. □

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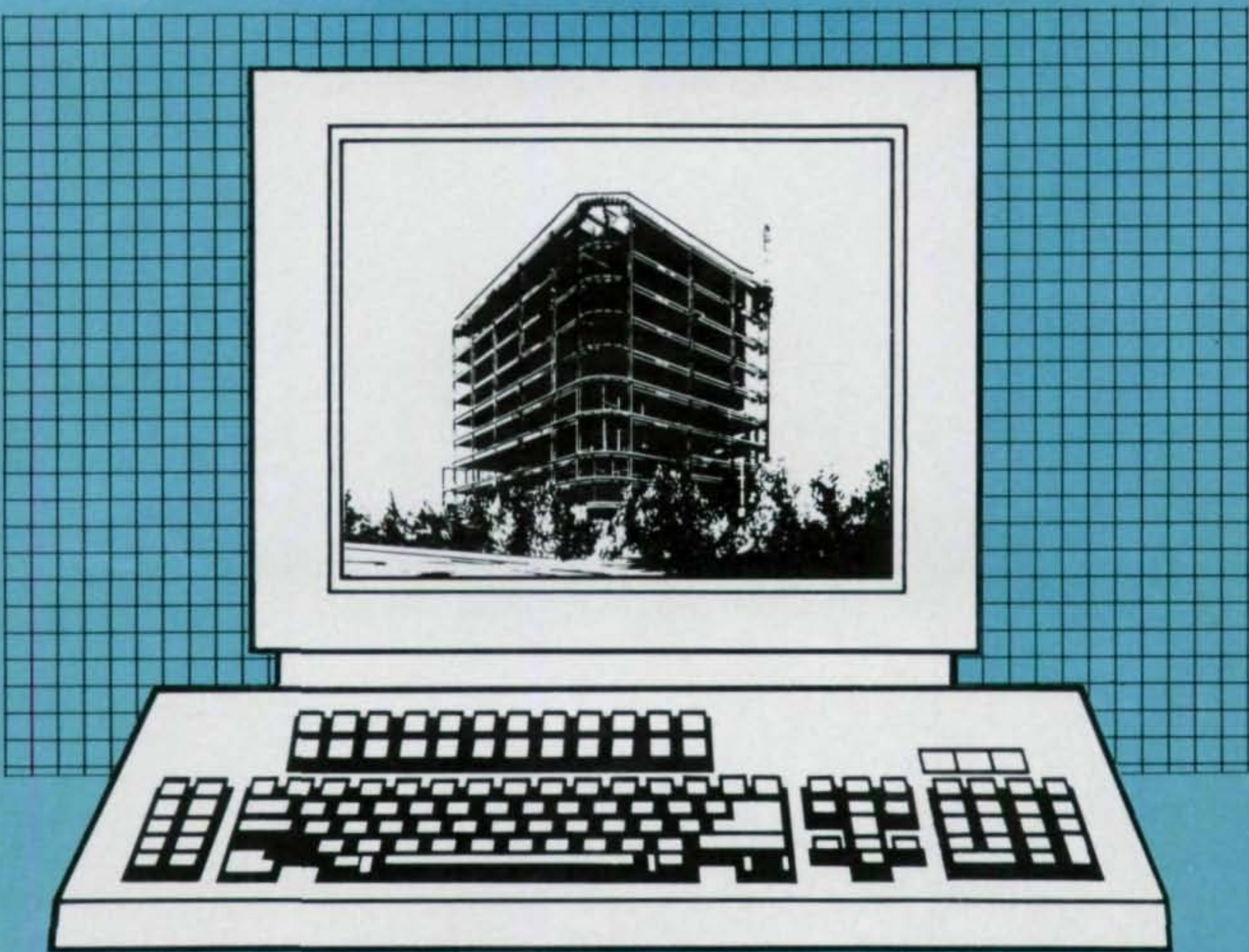


Two State-Of-The-Art Bridges For Maine

Under construction is one of the first bridges designed using the ALFD procedure and recently completed is a Rigid Frame Steel Girder Bridge

By Larry Roberts, PE

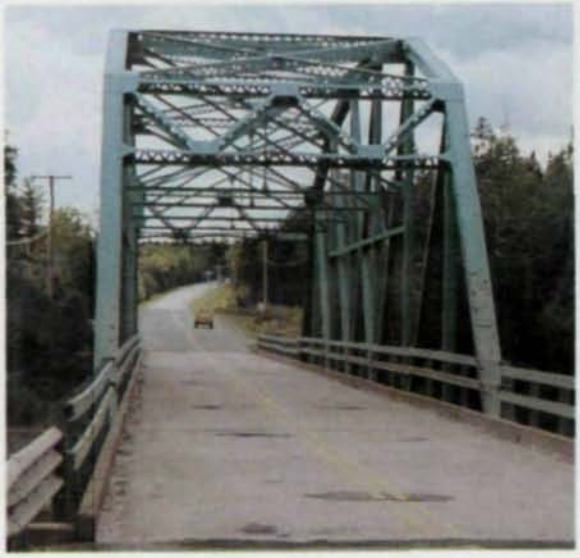
The Kennebec River in Maine will soon be a bridge connoisseurs delight. Two new replacement bridges, located just 22 miles apart, represent the state-of-the-art in design.

Still under construction is the East Outlet Bridge between the Townships of Big Squaw and Sapling. When complete, it will be one of the first bridges to be designed using the new Alternate Load Factor Design (ALFD), or Autostress Design, procedures.

The existing bridge had three 57′ simple approach spans of steel stringers with a concrete deck and a 170′ main span consisting of a steel thru-truss. The total bridge length is 350′ carrying a 20′-wide roadway. The bridge was scheduled for replacement due to its narrow width and the low inventory rating of the steel truss span.

A three-span bridge of continuous welded girders using A588 steel was selected as the most economical replacement structure. However, the Maine Department of Transportation chose this project to test the benefits of ALFD procedures.

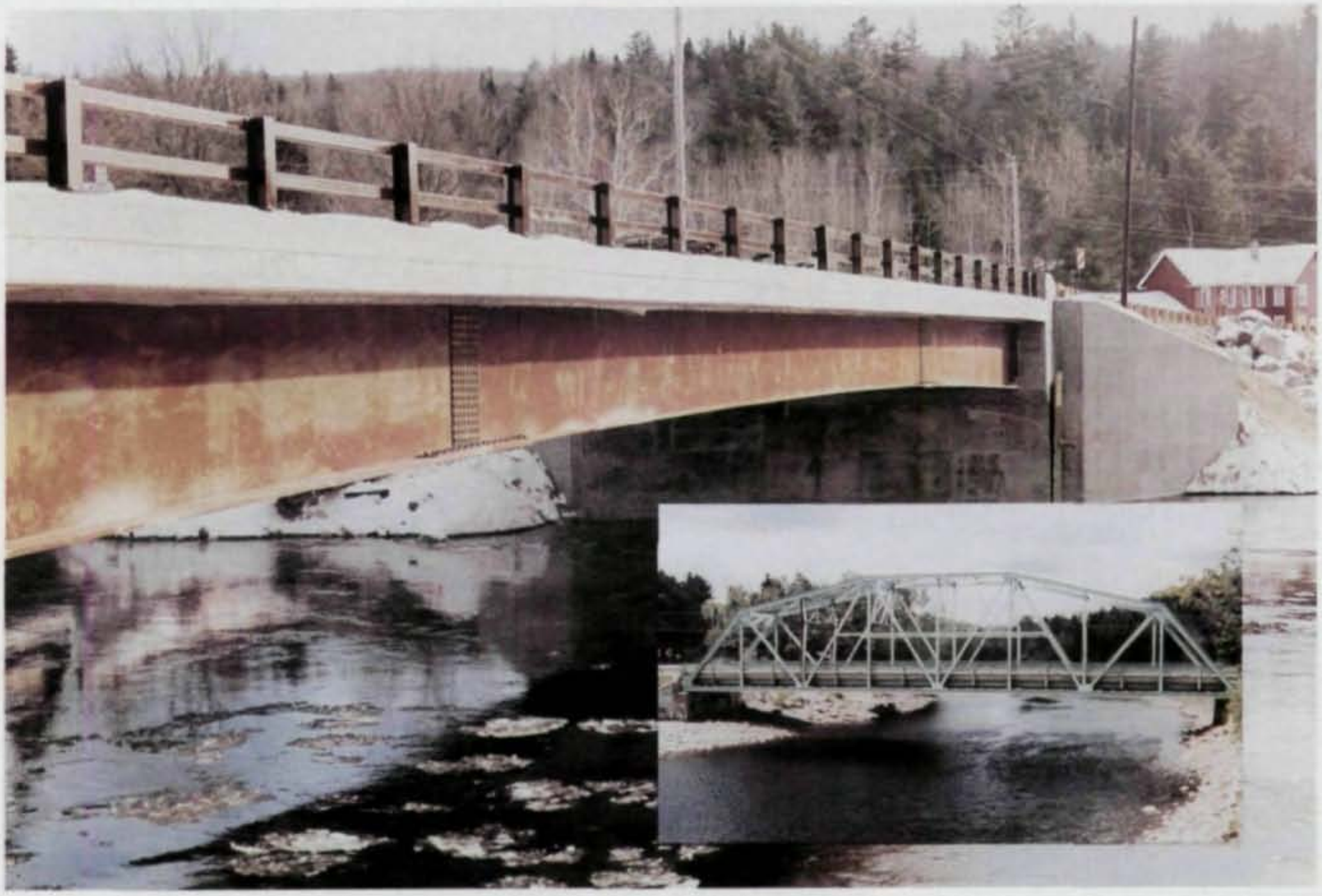
The recent AASHTO specification for ALFD utilizes modified plastic-design theory to determine the strength of the structure at Maximum Load, and shakedown theory to account for redistribution of elas-



The old East Outlet Bridge (left) is being replaced by a new bridge designed using the new Alternate Load Factor Design, or Autostress, procedures. Prior to making the decision to use ALFD, four different design alternatives were considered (chart below compares cost benefits).

	Max. Live Load Defl.		Total Steel Weight*	Weight / SF	Cost Ratio	
	Inches	Span / Defl. Ratio			Fab. 1	Fab. 2
AUTOSTRESS DESIGN METHOD WELDED BEAM ALTERNATE NO. 1	1.4"	1105	283,500*	25.1 psf	1.0	1.0
AUTOSTRESS DESIGN METHOD ROLLED BEAM ALTERNATE NO. 2	1.9"	783	360,000*	31.9 psf	1.04	1.08
LOAD FACTOR DESIGN METHOD WELDED BEAM ALTERNATE NO. 3	1.2"	1273	317,000*	28.1 psf	1.11	1.06
LOAD FACTOR DESIGN METHOD ROLLED BEAM ALTERNATE NO. 4	1.9"	797	414,500*	36.8 psf	1.29	1.27

* Total weight of steel includes estimated weight of diaphragms and beams.



tic negative bending moments at Overload to the positive bending regions of the structure. This redistribution occurs through the development of positive automoments that are caused by the controlled local yielding at interior piers.

For comparison, four alternatives were considered for the new bridge's design: compact welded beam designed by Autostress procedures; rolled beam designed by Autostress procedures; welded beam designed by Load Factor procedures; and rolled beam designed by Load Factor procedures.

The relative cost ratios (see chart on previous page) for materials and fabrication clearly pointed towards the compact welded beam designed by Autostress procedures as the most economical design, with the rolled beam Autostress design a close second.

It was apparent that Autostress design would result in more than a 10% reduction in structural steel cost for this project. In addition, it is expected that further savings may be realized in erection costs due to the possible elimination of two field splices per beam.

Based on this study, the Maine DOT designed the project using the Autostress Design Procedure. A further refinement in the final design was made by adjusting the span arrangements to 103'-124'-103-', which balanced the stresses between the end spans and the center span, and which in combination with Autostress procedures allowed the use of the same size bottom flange plates throughout the entire bridge. The project is scheduled for completion by the Fall of 1990.

A rectangular rigid frame steel girder bridge, the only one of its kind in Maine, was selected as the most practical design for the new The Forks Bridge (above). The inset shows the old span thru truss bridge constructed in 1932.

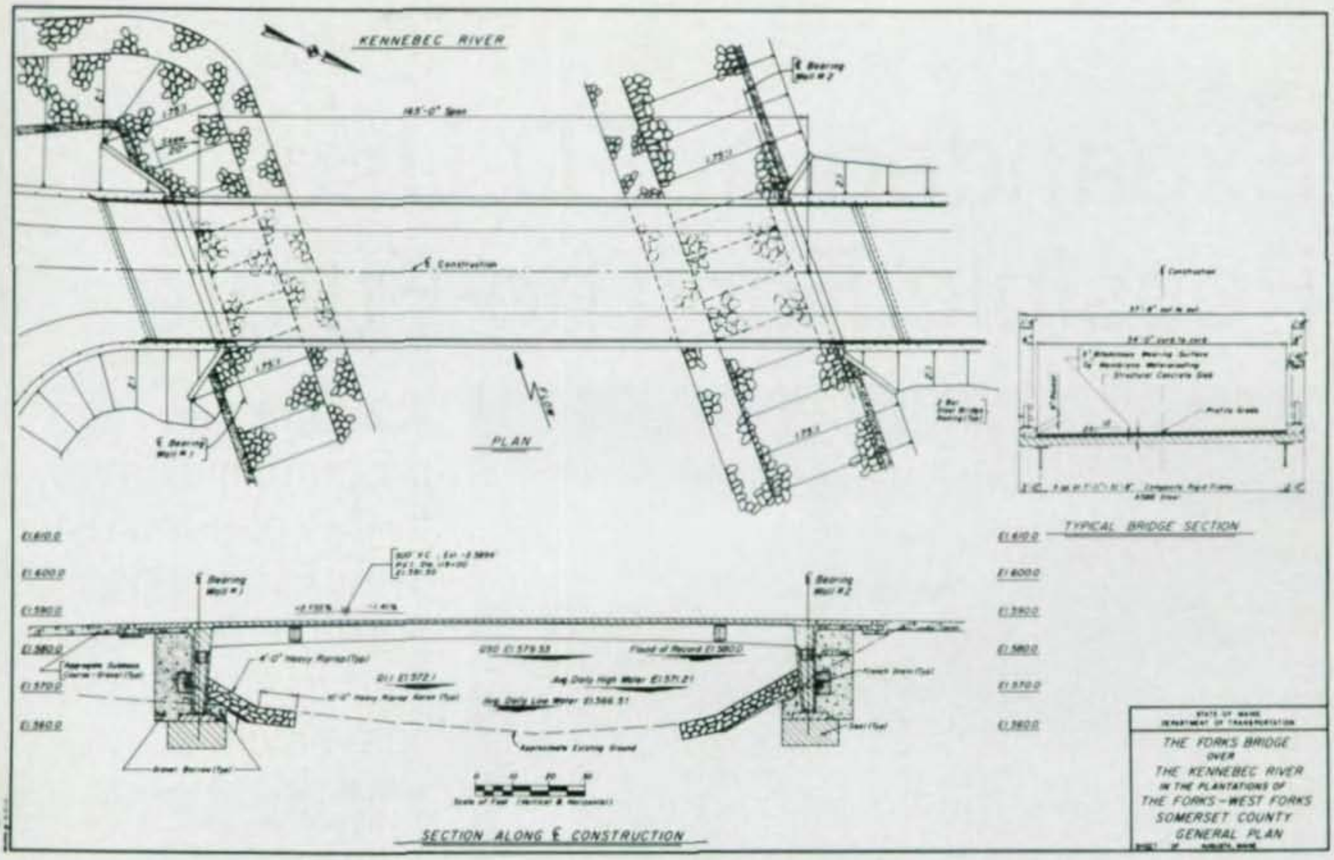
The Forks Bridge

Recently completed is The Forks Bridge, located between the plantations of The Forks and West Forks.

The existing bridge was a 168' span thru truss that was built in 1932. The bridge was narrow with a 22'-wide roadway, had poor alignment with its curved approaches, and had been posted due to its low operating and inventory ratings.

While the placement of a pier in the river channel would have simplified the bridge replacement, it wasn't practical due to the large number of whitewater rafters that use that portion of the river and the possibility of excessive scour. The streambed, which is composed of round cobblestones and gravel, is subject to scour from high stream velocities at flood flows. Calculations indicated that scour could be from 10' to 50' if a pier was located in the river channel.

In order to eliminate any interference to the more than 23,000 annual whitewater rafters and to reduce any scour concerns, the Maine DOT decided to construct a replacement structure that would



Due to gradeline considerations, a conventional single-span bridge would not accommodate the necessary beam depth. Instead, a rectangular rigid frame steel girder bridge was designed.

span the river from bank-to-bank. The DOT considered a conventional single-span bridge, but the gradeline limitations would not accommodate the necessary beam depth. The decision to span the river from bank-to-bank eliminated from consideration any conventional two-span or three-span structures.

Instead, a rectangular rigid frame steel girder bridge—the only one of its kind in Maine—was selected as the most practical design. The structure is a 165-ft. span, Rigid Frame Steel Girder Bridge, with a 34-ft. clear roadway width. The bridge is composed of five girders of composite, ASTM A588 unpainted weathering steel on a 20 degree skew.

An innovative design was needed due to the forces from soil pressures on the vertical legs. After consulting the University of Maine's Civil Engineering Department, it was suggested that full "passive" earth pressure should be applied at the upper third point of the wall, transitioning to an "at rest" pressure at the wall base.

Due to very dense soils in the streambed, the structure will be supported by concrete seals and a distribution slab. For simplification of erection and constructability, a field splice was located at about mid-height of the vertical legs. Falsework was required to temporarily support the 45' x 11' steel knees that form the corners of the rigid frame. The 75' center portion of the frame was then connected at the field splice locations.

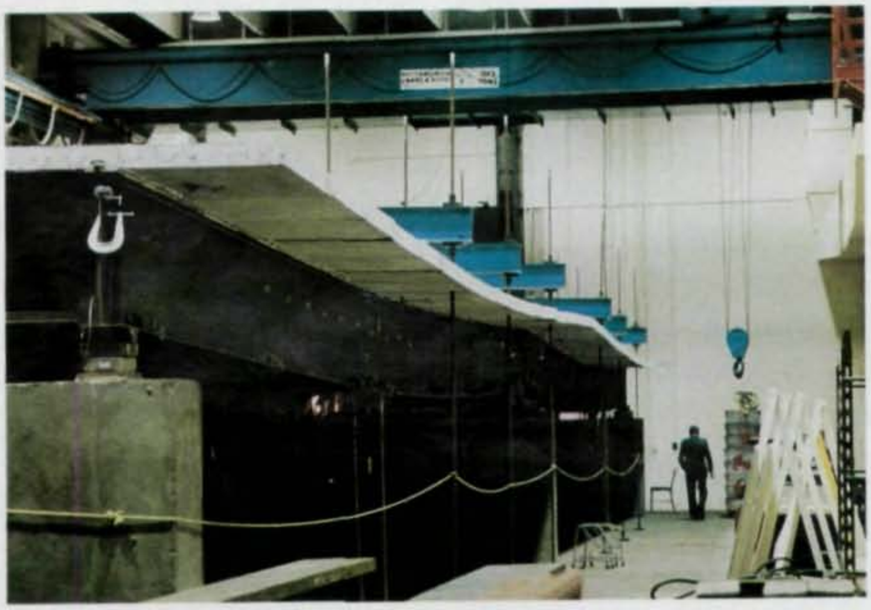
After the steel was erected and all diaphragms and connections tightened, the vertical legs were cast in concrete. All falsework was removed prior to placement of the superstructure slab. The portions of the steel legs encased in concrete were painted to provide protection from corrosion of the A588 steel, and all reinforcement in the bridge were epoxy coated for protection against corrosion.

Reed & Reed, Inc., of Woolwich, Maine, was the contractor on the $1.66 million project. The structural steel was bid at $550,000, which equates to $86 per sq. ft. of deck area. The bridge related items totaled $1.165 million, which equates to $182 per sq. ft. of deck area.

It was estimated that a one-span bridge with deep abutments at this location would have cost $150 to $180 per sq. ft., while a multi-span bridge would have cost $120 to $145 per sq. ft. By comparison, the rigid frame unit cost $145 per sq. ft., or about 10% more than conventional construction.

This article is based on a paper by Larry L. Roberts, PE, assistant bridge engineer, Maine DOT. Roberts was the project manager on both projects. Norman Baker, PE, was the project designer for the East Outlet Bridge, while Michael Burns, PE, and Norman Baker, PE, were co-designers of The Forks Bridge. □

Expanded ALFD Use Possible For The Future



Pictured above is the model bridge after a verticle load test. The bridge was able to sustain total additional live load approximately 240 percent above the rated live load, which already included all required factors of safety, before reaching the limit of jack capacity. Shown opposite is a cross-section and elevation of the prototype bridge on which the four-tenths scale model is based. ALFD based on Autostress principles shows that the Maximum Load requirements for strength rarely govern the design of continuous members. Even when Overload criteria govern the design, there is no need for a stress check in negative moment sections at intermediate supports. This means that the distribution of material along the length of members can be chosen to minimize fabrication costs. Eliminating flange splices in field sections, as was done in the prototype bridge, is an effective bridge, is an effective means of reducing the cost of fabrication. The bottom flange of each girder is constant along the entire length of the bridge except near the end supports. The top flange is sonstnat along the entire length. Any size section could have been chosen at the pier because the required automoment will be created automatically by whatever yielding takes place at Overload. The prototype bridge probably represents the minimum size section that would be chosen in actual design. Because optimum design depends to a large extent on fabrication costs, it is important that fabricators become involved in the design phase. The advantage of the new method is that dimensions of continuous members can be selected on the basis of common sense. The design procedures will make it work as long as there is sufficient steel somewhere in the structure.

A recent test program indicates that expanded use of ALFD design is possible and could lead to substantial cost savings

By Mark Moore, Michael A. Grubb and Lloyd R. Cayes

The first bridges designed according to the current AASHTO guide specifications for Autostress or Alternate Load Factor Design (ALFD) dramatically reduced construction costs, and a recently completed test program indicates the potential for expanding ALFD use.

In ALFD, a designer is permitted to utilize some of the substantial post-yielding reserve strength that is available in composite continuous steel-girder bridges. The concepts of shakedown at Overload and plastic mechanism analysis at Maximum Load are introduced in ALFD as more realistic limit-state criteria for continuous steel bridges under heavy loads.

Current AASHTO guide specifications allow the use of ALFD only for the design of continuous steel-girder bridges using rolled-beam and comparable welded-beam sections that satisfy specific compactness requirements. The first bridges designed according to this guide specification have been completed in New York and Tennessee, and both cost substantially less than they would have if designed by conventional procedures. Additional projects are now underway in Maine and Illinois.

An extensive two-year test program was completed last year as part of a research program to extend the ALFD concepts to non-compact plate-girder sections with slender webs that fall outside the compactness limits of the present guide specifications. The experimental study involved the laboratory testing of a 0.4 scale model of a two-span continuous steel-girder highway bridge.

The model bridge consisted of two 56' spans each with three plate girders transversely spaced at approximately 6'-9". Each plate girder was approximately 28" deep, and the girders supported 4"-thick modular precast concrete deck panels with approximately 2'-10" overhangs. The model bridge corresponded to a two-span continuous prototype bridge with equal spans of 140' and an overall depth of 48'.

The precast panels were made composite with the plate girders using stud shear connectors. The panels were prestressed both transverse and parallel to the bridge axis.

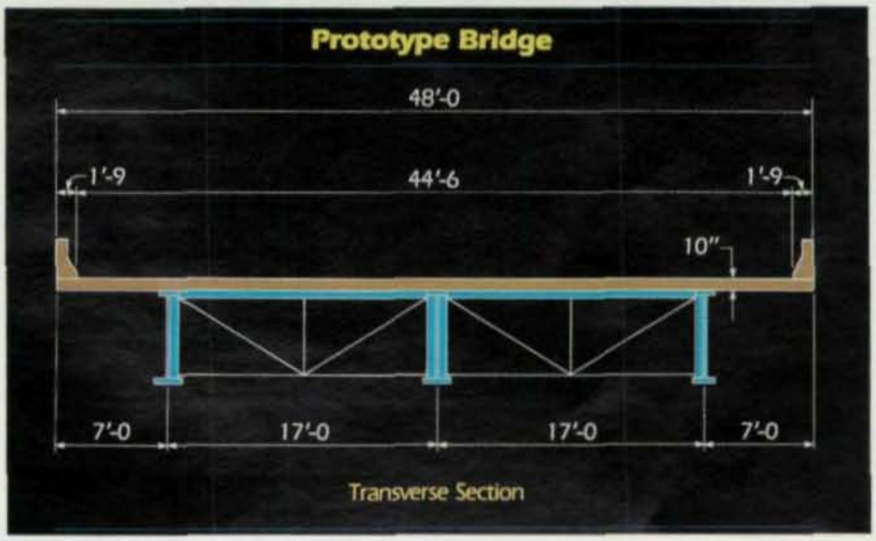
Because the girders were designed using ALFD procedures, it was possible to use a prismatic girder section over the interior pier. ALFD procedures recognize the ability of continuous steel members to adjust automatically for effects of controlled local yielding at Overload. The local yielding causes automoments to form that remain in the structure. Because of the automoments, the structure shakes down after several passages of an Overload vehicle; that is, the local yielding is stabilized.

As a result of automoment formation caused by local yielding, a portion of the peak elastic bending moment at interior piers is automatically redistributed by the structure to lower-stressed positive-moment sections. Taking advantage of this inherent ability permits the designer to consider using prismatic steel members in continuous spans along the entire bridge length or between the field splices. The concomitant benefits include lower fabrication costs and elimination of structural details with undesirable fatigue characteristics.

The model bridge was subjected to a series of tests to evaluate specific responses at simulated AASHTO Service Load, Overload and Maximum Load levels.

At elastic Service-Load stress levels, live-load lateral-distribution factors for the exterior and interior girders in positive and negative bending were computed from experimentally developed influence surfaces. These factors were compared to factors computed from a finite-element model, from proposed empirical formulas that were developed as part of an NCHRP research project on lateral live-load distribution, and from present AASHTO procedures.

The agreement between the factors computed from the experimental and MSC/NASTRAN data was generally good. The factors computed from the proposed empirical formulas also gave good agreement



Transverse Section

with experimental data, especially for the interior girder, even though the prototype bridge fell outside some of the limits of applicability for the proposed empirical formulas.

The factors computed using present AASHTO procedures were quite conservative for the interior girder, and less so for the exterior girders. Neither the proposed nor the present AASHTO procedures accounted for the observed variation of the distribution factor along the span. The data would seem to indicate that finite-element analysis is a plausible method for computing elastic wheel-load girder distribution factors.

At Overload, shakedown with the formation of automoments was experimentally observed. Shakedown with stabilization of local yielding occurred after about three cycles of alternating simulated AASHTO Overload truck and lane loading, with the maximum stress in positive bending in the most heavily loaded girder at approximately the Overload limit state of 0.95 F_y. The girders behaved satisfactorily, even with controlled local yielding allowed at interior piers.

The precast prestressed modular panels behaved exceptionally well. Thus, the ALFD limit-state criteria appeared to adequately satisfy the Overload structural performance requirement of unobjectionable riding quality for this bridge, designed using non-compact girder sections. Refinements to the inelastic moment-rotation curve used to compute the automoments for non-compact girder sections will be made in upcoming related AISI research projects.

At Maximum Load, the bridge had significant reserve strength under simulated AASHTO Maximum Load lane loading. The bridge was able to sustain three applications of a live load over two times the approximate rated AASHTO Maximum Load live load. The ALFD plastic mechanism analysis using an effective plastic moment was adequate to ensure that this bridge had sufficient strength to resist the load.

The above results were based on analysis of the numerous data available from this model-bridge test program. Additional analysis is underway, and when complete, the results will be made available.

This article is condensed from a paper by Mark Moore, senior structural engineer, Wiss, Janney, Elstner Associates, Inc.; Michael A. Grubb, assistant manager—bridge engineering, AISC Marketing, Inc.; and Lloyd R. Cayes, laboratory engineer, FHWA. □

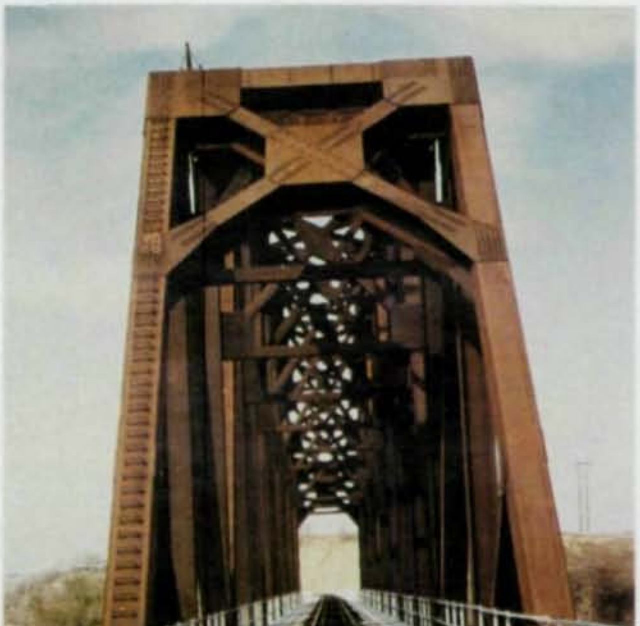
Weathering Steel Requires Proper Detailing

Weathering steel can substantially reduce bridge construction costs and have a positive environmental impact

By Robert L. Nickerson

While the cost effectiveness of uncoated weathering grade steels has been clearly demonstrated, the use of the material in improper locations or conditions has resulted in several cases of poor performance.

The best publicized example occurred in Detroit, where the weathering steel rusted at an excessive rate. And as a result, Michigan, which was soon followed by others, put a full or partial ban on the use of unpainted weathering steels.



Shown above is a railroad truss bridge for Burlington Northern Railroad that used weathering steel.

Guidelines ignored

But an analysis of several of these "failures" showed several areas where industry supplied guidelines were not followed, or there were some additional limitations that had to be adhered to for proper performance, but were ignored.

The elimination of unpainted weathering steel for bridge construction would be a mistake. In addition to the economic benefits that accrue from the elimination of the cost of the initial painting, there are substantial environmental benefits. Because the bridges are unpainted, there is a reduction in the emission of volatile organic compounds when volatile-based coatings are specified.

Also, because no coatings need to be removed during maintenance and repainting, there is no disposal problem of contaminated blast cleaning debris. There are documented cases where the estimated cost of the collection and disposal of materials from a structure repainting project were so great the structure was either abandoned or the structure placed into a "terminal maintenance" phase.

New guidelines

At a July 1988, "Weathering Steel Forum," the Federal Highway Administration, together with AASHTO and the steel industry, developed prelimary guidelines which were used as the foundation for the publication of FHWA Technical Advisory T5140.22 (Oct. 3, 1989): "Uncoated Weathering Steel in Structures."

The guidelines specifically advise that caution be exercised in specifying weathering steel in: marine coastal areas; frequent high rainfall, high humidity or persistent fog environments; industrial areas where concentrated chemical fumes may drift directly onto the structure; grade separations in "tunnel-like" conditions; and low level water crossings, where there is 10' or less over stagnant, sheltered water, or 8' or less over moving water.

Salt-laden air a problem

The problem in marine coastal areas is caused by the chloride concentration in salt-laden air. The effect on weathering steel structures depends on the distance to the shore line, the direction of the prevailing winds, and the topographical and environmental characteristics of the area.

The United Kingdom DOT Standard BD/7/81 suggests that uncoated steel should not be used when the chloride level exceeds 0.1 mg/100 cm^2/day, average. However, because corrosion rates in the United States are substantially lower than in the United Kingdom, a higher level of chloride con-

Weathering steel, when properly engineered, can often be used economically and without problems on a wide variety of structures. Shown above is a plate girder bridge.

tamination can be tolerated. Based on available information, it is estimated that weathering steel can be used safely in the United States at chloride levels up to 0.5 mg/100 cm^2/day, average.

Beware of high humidity

High rainfall or high humidity areas should be evaluated using ASTM Test G84, "Time of Wetness Determination (On Surfaces Exposed to Cyclic Atmospheric Conditions)." If the yearly average time of wetness exceeds 60%, caution should be used in specifying bare weathering steel.

Likewise, weathering steel should not be used in industrial areas where the threshold level for sulfur trioxide exceeds 2.1 mg/100 cm^2/day average.

According to the guidelines, certain design details require special attention when weathering steel is specified. These include:

- eliminate bridge joints where possible;
- expansion joints must be able to control water that is on the deck (consider the use of a trough under the deck joint to divert water away from vulnerable elements);
- paint all superstructure steel within a distance of 1-1/2 times the depth of girder from bridge joints;
- do not use welded drip bars where fatigue stresses may be critical;
- minimize the number of bridge deck scuppers.
- eliminate details that serve as water and debris "traps".
- Hermetically seal box members when possible, or provide weep holes to allow proper drainage and circulation of air;
- Cover or screen all openings in boxes that are not sealed;
- Consider protecting pier caps and abutment walls to minimize staining;
- Seal overlapping surfaces exposed to water to prevent capillary penetration action.

The proper design details, especially for controlling roadway drainage and facilitating maintenance, also are crucial to the success of a weathering steel structure.

Bridge joints should be eliminated to the fullest extent possible. Jointless steel bridges have been used to lengths of 400' and greater, with some up to 1,600' with joints only at the ends. Because it is practically impossible to create a leak-proof joint, when joints are absolutely necessary, all steel within a minimum distance of 1-1/2 times the depth of the girder from the joint should be coated.

Drip bars on the top and bottom of the lower flanges can be effective in intercepting drainage. However, welding of any attachment to the tension flange should be considered only after a through analysis of the impact of the attachment on the fatigue life of the girder.

Scuppers

The spacing between drainage scuppers should be maximized in accordance with the established hydrologic and hydraulic design. For more information, consult the FHWA Report No. FHWA/RD/87 014 "Bridge Deck Drainage Guidelines."

Scupper downspouts should be designed and placed so the drainage will not contact the steel surface. However, details used to connect scuppers to drain pipes have often created more problems than they have solved. Caution is needed to insure that details do not provide flat runs of piping, elbows that clog or connections that separate.

Other features

After passing over uncoated weathering steel, drainage leaves dark, non-uniform and often unsightly stains on concrete surfaces. This problem can be mitigated, however, by one of three methods: wrapping the piers and abutments during construction to minimize staining while the steel is open to rainfall; allowing/requiring the contractor to remove staining with a commercial solvent after construction is complete; or applying epoxy or some other material to coat and/or seal the concrete surfaces against staining.

If water is allowed to flow over overlapping joints, capillary action can draw the water into the joint and cause "rust-pack" to form. Therefore, the contact surfaces must be protected from intrusion of rainfall and runoff.

With billions of dollars needed to repair and upgrade the nation's infrastructure, it is crucial that every dollar be spent wisely.

While there have been documented cases where unpainted steel has performed poorly, these are typically due to its use in improper locations or conditions. Weathering steel, when properly engineered, is one cost effective solution to this nation's infrastructure dilemna.

This article is condensed from a paper presented by Robert L. Nickerson, P.E., chief of structures for the Federal Highway Administration, at The National Symposium on Steel Bridge Construction. □

Two New Groups Offer Aid For Bridge Designers

Information Center Is More Than Just The BASICS

Reidar Bjorhovde, P.E., is the director of the Bridge And Structures Information Center and a professor at the University of Pittsburgh.

By Reidar Bjorhovde

Engineers faced with problems on steel structures can now turn to a new source for help: The Bridge And Structures Information Center (BASIC).

The center, which was formed in Spring 1989, is designed to speed the transfer of acquired and available knowledge through the establishment of a data base on past design, fabrication and construction problems and solutions. Though still in its infancy, when completely established, the data base will be indexed and cross-referenced. The center's progress will be charted in a quarterly newsletter, the first issue of which is scheduled to be distributed during the first few months of 1990.

In addition, BASIC is set up to resolve problems and help avoid costly and time-consuming litigation. Upon inquiry and/or request for information on a problem for which previous experience is limited or non-existent, the director of the center will work with his advisory council to identify consultants who would be willing to assist in solving the problem and who are acceptable to all parties of the dispute.

For example, if an engineer has a problem with welded joints on a steel structure and the owner's consultants don't agree with the proposed solution, the problem can be brought to the center. If the particular problem is not contained within the existing database, BASIC will refer the engineer to one or more technical consultants on the center's "Panel of Experts".

Continued on page 60

Council presents information on designing with steel

Bridge construction and technology should receive a boost in the 1990s due to the formation last year of the Council for the Advancement of Steel Bridge Technology.

The organization's goals are to advance steel design concepts, construction techniques, aesthetics, economical solutions and reliable service performance.

Membership is derived from the American Iron and Steel Institute, American Institute of Steel Construction, AISC Marketing, Inc., National Erectors Association, Steel Structures Painting Council, National Electrical Manufacturers Association and Industrial Fasteners Institute. Future membership will include organizations representing bridge components such as bearings, decks and expansion joints. Also, various industry experts will be invited to participate in quarterly meetings.

According to Thomas D. Heimerl, vice chairman for the council and a marketing manager at United States Steel, the council's objectives include:

- acting as a clearinghouse to disseminate the latest information on developments in steel bridge technology;
- promoting the development of technical documents, design concept aids and other criteria that illustrate the best current practice and techniques;
- addressing the requirements of the total bridge system through sponsorship of programs and activities that enhance steel's position in the bridge market;
- interfacing with specification establishing authorities to encompass the latest, most economical design and construction methods for steel bridges;
- supporting existing regional interest groups and encouraging the establishment of additional local groups throughout the nation;
- and consolidating the needs of regional interest groups and providing the resources to satisfy

Continued on page 61

BASIC, Cont.

The experts, who currently number more than 50, serve on one of 10 task forces: (TF1) Design, analysis and material selection considerations; (TF2) Metallurgical considerations (bolts, welds, and materials; fatigue and fracture); (TF3) Special structural components (cables, bridge and support decks, guard rails, bearings, etc.); (TF4) Fabrication and other shop operations; (TF5) Non-destructive testing techniques; (TF6) Shop and field inspection; (TF7) Corrosion; (TF8) Cleaning and paint systems; (TF9) Transportation and structural components; and (TF10) Erection.

The cost for the service varies, but includes an administrative fee and a negotiated price for the consultants' services. BASIC is more than simply a referral service, however, because it includes an advisory council.

The advisory council is composed of bridge experts from a variety of organizations and includes: Arthur W. Hedgren, vice president, HDR/Richardson-Gordon, Pittsburgh; Dean C. Krouse, senior metallurgical applications engineer, Bethlehem Steel Corp., Bethlehem, Pa.; Clellon L. Loveall, assistant executive director, Bureau of Planning and Development, Tennessee Department of Transportation, Nashville; Stanley T. Rolfe, professor and chairman, Department of Civil Engineering, University of Kansas, Lawrence, Kan.; and Robert P. Stupp, executive vice president, Stupp Bros. Bridge & Iron Co., St. Louis.

Special advisors from the American Institute of Steel Construction (AISC) include: Fredrick Beckmann, director of bridges; Geerhard Haaijer, vice president for technology and research; and Neil W. Zundel, president.

Another service the center plans to offer in the future is software evaluation. The software evaluation will be by request only, and the analysis will be available only to the engineer who pays it. Typically, the software programs to be evaluated will be new introductions.

In conjunction with BASIC, the University of Pittsburgh plans to develop short courses for engineers. The first course, a one-day session on welding for structural engineers, was offered last November and was sponsored by both the University and the Association of Bridge Construction and Design. These courses are designed as continuing education programs for engineers who don't have the time to attend a full-blown graduate program.

For more information on BASIC, contact: Reidar Bjorhovde, director, BASIC, Department of Civil Engineering, University of Pittsburgh, 934 Benedum Hall, Pittsburgh, PA 15261-2294 (412) 624-9870/9879.

This article is based on a paper presented by Reidar Bjorhovde, director of BASIC, at the National Symposium on Steel Bridge Construction. □

Council, Cont.

ters for the Highway Structures Design Handbook on fasteners and welding of bridge structures; establishing uniform specifications for the surface preparation and painting of steel bridges; creating uniform implementation of bolting specifications for bridge structures; and developing a steel vs. concrete promotion video.

The council's first quarterly newsletter, *Bridge Technology News*, was distributed in September. For more information on the council or newsletter, contact Fredrick Beckmann, director of bridges, AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001 (312) 670-5413. □

The 1990 National Steel Construction Conference

Economical bridge design, Load and Resistance Factor Design (LRFD) frame analysis and long-span composite joists are just some of the subjects scheduled for the 1990 National Steel Construction Conference. Scheduled for March 14-17 in Kansas City, Mo., it is the only "All-Steel" conference and trade show in the U.S.

In addition to technical seminars and workshops, there will be more than 50 product exhibitors. For the fourth consecutive year, the AISC National Engineering Conference and AISC Conference of Operating Personnel are being held concurrently with the show. As such, the conference remains the premiere meeting place for engineering professionals, and the best place to obtain the most up-to-date information about buildings and bridges designed and built in steel.

Workshops are designed as intensive educational sessions where the nuts-and-bolts details of designing, fabricating and erecting structural steel are discussed. Every aspect of the construction process—from concept to competition to completion—receives attention. Of special interest are sessions on computerized design, LRFD, Autostress Design, project management, shop and field inspection and safety, quality certification, productivity, welding, bolting, cleaning and painting.

The focus at the conference is on practical solutions to common problems. In addition, the conference is often the first forum for introducing the latest research on structural steel design, recent code changes and technological advances.

A condensed program, along with abstracts of some of the papers to be presented at the conference is included in this special preview section. To register for the 1990 conference, fill out the form on page 64. A Hotel information and a reservation form is included on page 68. For more information, call (312) 670-5432.

Schedule Of Events

Monday and Tuesday, March 12-13

Exhibitor Move-In—Bartle Hall, Second Floor, Kansas City Convention Center (Registration in First Floor Foyer, K.C. Convention Center)

Wednesday, March 14

8:00 - Noon	Partners in Education Meeting —Allis Plaza
9:00 - Noon	AISC Committee on Research Meeting—Allis Plaza
9:00 - Noon	ASCE Committee on Steel Building Structures—Allis Plaza
9:30 - Noon	Optional Event #1: Tour of Kansas City (Charge: $10)
12:00 - 1:30 p.m.	Partners in Education luncheon Allis Plaza

Continued on page 65

REGISTRATION FORM

Registration Fees: (Please circle appropriate fees)

AISC Member Fee: $275.00 (before February 1)
$325.00 (After February 1)
(Includes AISC Active, Associate & Professional Members)

Non-Member Fee: $325.00 (before February 1)
$375.00 (after February 1)

Educator fee: $100.00
(Employed full-time at accredited architectural or engineering college or university.)

Student Fee: $ 75.00
(Letter from faculty advisor or equivalent required)

Exhibitor, in Booth (no charge)

Added Exhibitor: $ 75.00

Spouse's Fee: $ 75.00

Registration Fees Include all General and Plenary Sessions, workshops, seminars, coffee breaks, luncheons Thursday and Friday, the Get-acquainted Cocktail Reception Wednesday evening and a printed, bound copy of the Proceedings. Exhibitors are entitled to one registration for each 10-ft × 10-ft exhibit space reserved. "Added Exhibitor" fee is payable **ONLY** if in excess of one person per 10-ft × 10-ft.

Registration Cancellation Policy: Cancellations received before March 9, 1990, 100% of pre-paid registration fees will be refunded; after March 9, 50% will be refunded. (Those cancelling after March 9 will receive their copy of the Conference Proceedings.)

Partial Registration Fees
(You may also pre-register for one day or half day. Circle your choice below.)

Half Day Sessions: (Lunch not included)	
Wednesday Afternoon	$ 50.00
Thursday Morning	$ 65.00
Thursday Afternoon	$ 65.00
Friday Morning	$ 65.00
Friday Afternoon	$ 65.00
Saturday Morning	$ 25.00
One Day Sessions:	
Thursday (includes Lunch)	$150.00
Friday (includes Lunch)	$150.00
Exhibitor Visitor:	$ 5.00
Total Partial Registration Fees	**$**______

Registration for Optional Events

Event	**No Tickets**	**Total Price**
#1—Kansas City Tour (Wed., 9:30 a.m.)	___@ $10.00	$ ______
#2—Jazz Show/Dinner (Thurs., 7:45 p.m.)	___@ $37.00	$ ______
#3—Dinner Theatre (Fri., 6:30 p.m.)	___@ $30.00	$ ______
#4—Tour, Block 111 (Sat., 9:30 a.m.)	___@ No fee	
#5—Tour Fab. Plant (Sat., 9:30 a.m.)	___@ No fee	
#6—Tour Steel Bridge (Sat., 9:30 a.m.)	___@ No fee	
#7—Independence (Sat., 1:00 p.m.)	___@ $12.00	$ ______
#A—Secrets of the Stones (w/lunch) (Thurs., 11:30 a.m.)	___@ $20.00	$ ______
#B—Weston Tour (Thurs., 1:30 p.m.)	___@ $16.00	$ ______
#C—Nelson Gallery/K.C. Museum (Fri., 9:15 a.m.)	___@ $16.00	$ ______
#D—Independence (w/lunch) 12:30 p.m.	___@ $25.00	$ ______
Total Optional Event Fees		**$** ______

PLEASE REGISTER (Type or Print)

Name ________ Nickname (for badge) ________

Company ________ Title ________

Mailing Address ________

City and State/Zip ________ ( ) Bus. Phone ________ ( ) Home Phone ________

If spouse or other guest is registering for Complete Spouse's Program, or individual Spouses' or Optional Events, please complete next line for badge.

Name of Individual Registering for Other Events ________ Nickname (for badge) ________

Conference Fees Payable:

Registration Fee:	$ ______
Spouse's Fee:	$ ______
Partial Registration Fees:	$ ______
Optional Events:	$ ______
Total Registration Fees:	**$** ______

___ I enclose check (U.S. funds) payable to AISC in amount of total fees.
___ Please charge my credit card # ________
Visa or MasterCard only—Circle one: VISA MasterCard

Expiration Date (Month and Year) ________
Signature (if any credit card charges): ________

MAIL COMPLETED FORM AND CONFERENCE FEES TO:

American Institute of Steel Construction, Inc.
1990 National Steel Construction Conference
P.O. Box 806286
Chicago, Illinois 60680-4124

Phone inquiries and information: 312/670-5432

Schedule Of Events, Cont.

1:30 - 5:00 p.m. Educator Meeting—Allis Plaza
1:30 - 3:00 p.m. Professional Member Forum—Allis Plaza
1:30 - 3:00 p.m. AISC Research/ASCE Steel Structures Joint Meeting—Allis Plaza
1:30 - 3:00 p.m. General Session: Construction Claims—Bartle Hall
3:00 - 5:00 p.m. Exhibits Open—Bartle Hall
5:15 - 6:00 p.m. Exhibitor Workshops—Bartle Hall
A. TradeARBED
B. Welded Tube Co. of America
C. Vernon Tool Co.
D. EGYPT Structural Steel Processing
E. Design Data
Additional workshops t.b.a.
6:30 - 8:00 p.m. AISC Welcome Cocktail Party—Bartle Hall (exhibits open)

Thursday Morning, March 15

7:00 - 8:00 a.m. SASF Educator Breakfast—Allis Plaza
7:00 - 8:00 a.m. VCSSF Educator Breakfast—Allis Plaza
7:30 - 8:15 a.m. Exhibitor Workshops—Bartle Hall
H. Structural Software
I. The Rawls Co.
Additional workshops t.b.a.
8:30 - 9:00 a.m. Award Presentations—Bartle Hall
9:00 - 9:15 a.m. General Session: Bridge & Structures Information Center—Bartle Hall
9:15 - 10:00 a.m. General Session: Case History (27-Story LRFD Office Building)—Bartle Hall
10:00 - 3:00 p.m. Exhibits Open (Lunch served at 11:45 a.m.)
10:45 - 12:15 p.m. Technical Seminars—Bartle Hall
1. Construction Claims
2. Products Liability Insurance
4. Frame Analysis, LRFD
5. Connection Design and Detailing
7. Recruitment and Training of Steel Detailers
17. Economical Bridge Design

Thursday Afternoon, March 15

11:30 - 1:00 p.m. Spouses' Event #A (Lunch and Program)—Allis Plaza
1:30 - 5:30 p.m. Spouses' Event #B - Historical Tour to Weston, Missouri
1:00 - 2:15 p.m. Poster Session - Bartle Hall
2:30 - 4:00 p.m. Technical Seminars—Bartle Hall
3. What the First Line Supervisor Should Know
6. PR Connections
7R. Recruitment and Training of Steel Detailers
8. Plant Automation and Layout
10. Floor Serviceability and Constructability
11. Beam Camber
4:10 - 5:25 p.m. Technical Seminars—Bartle Hall
1R. Construction Claims
12. Building Design
13. Tubular Connections
14. Connection Design Responsibility—Present Status
16. Economical Framing System
18. Fire Protection—Australian Case Studies
5:30 - 6:15 p.m. Exhibitor Workshops—Bartle Hall
O. Mountain Enterprises
P. IKG/Greulich
Additional workshops t.b.a.
7:00 - 7:45 p.m. Reception (cash bar)—Basie Foyer, Allis Plaza
7:45 - 10:00 p.m. Optional Event #2: Conference Dinner and Entertainment—"All That Jazz" (charge: $37)

Friday Morning, March 16

7:30 - 8:15 a.m. Exhibitor Workshops—Bartle Hall (to be announced)
8:30 - 10:00 a.m. Technical Seminars—Bartle Hall
2R. Products Liability Insurance
4R. Frame Analysis, LRFD
5R. Connection Design and Detailing
9R. Protecting Your Workers Against Welding Fumes
15. Scope of Business
17R. Economical Bridge Design
9:15 - Noon Spouses' Event C—Nelson Gallery/Kansas City Museum
10:00 - 3:00 p.m. Exhibits Open (Lunch served at 12:15 p.m.)
10:45 - 12:15 p.m. Technical Seminars—Bartle Hall
3R. What the First Line Supervisor Should Know
6R. PR Connections

8R. Plant Automation and Layout
10R. Floor Serviceability and Constructability
11R. Beam Camber
15R. Scope of Business

Friday Afternoon, March 16

12:30 - 4:30 p.m. Spouses' Event D—Trip to Independence, Mo.
3:15 p.m. Exhibitor moveout begins
3:00 - 3:45 p.m. General Session: T.R. Higgins Lecture (winner to be announced)—Bartle Hall
4:00 - 5:30 p.m. Technical Sessions—Bartle Hall
9R. Protecting Your Workers Against Welding Fumes
12R. Building Design
13R. Tubular Connections
14R. Connection Design Responsibility
16R. Economical Framing Systems
18R. Fire Protection—Australian Case Studies

6:30 - 10:45 p.m. Optional Event #3—Dinner Theater, Waldo Astoria (charge: $30)

Saturday, March 17

8:30 - 9:30 a.m. General Session: "Unusual Steel Framing—34 Story Office Building in Kansas City: Block 111" —Bartle Hall
9:30 - Noon Choice of:
Optional Event #4—Hard Hat Tour of Block 111 (no charge)
Optional Event #5—Tour of Fabrication Plant (no charge)
Optional Event #6—Hard Hat Bridge Tour (no charge)
1:00 - 4:00 p.m. Optional Event #7—Trip to Independence, Mo. (charge: $12) (Repeat of Spouses' Event C, but does not include lunch.)

(For Program Detail, see the November-December issue of *Modern Steel Construction*, or call AISC, 312-670-2400, for a copy of the program and information on exhibits.)

See Page 64 for Registration Form; Page 68 for Hotel Reservations

1990 Steel Construction Conference:

Technical Seminar Preview

Design Innovations Reduce Floor Weight

Substantial weight reductions are possible with new design systems according to the two papers scheduled to be presented at a seminar on floor serviceability and constructability.

"Innovative Light-Weight Floors for Steel-Framed Buildings," by John R. Hillman and Thomas M. Murray, both of the Virginia Polytechnic Institute and State University, presents several innovative concepts that have excellent potential as alternative light-weight floor systems.

The concepts described in the paper include: profiled deck/dry board composite floors; precast concrete plank systems; concrete filled/fiber reinforced plastic deck; a long-span cold-formed deck/composite concrete slab system; and several variations of modified steel bridge grid systems. The advantages and disadvantages of each system are discussed with emphasis on serviceability criteria.

In addition to the conceptual investigation, full scale models were constructed of two of the proposed floor systems: a long-span cold-formed deck with a 2" composite concrete slab and a scaled down, steel grid, bridge deck with a thin concrete slab. Both prototypes are 30' x 30' single bay floor systems.

Serviceability Issues

"Innovative Floor Systems," by Roberto T. Leon of the Department of Civil and Mineral Engineering at the University of Minnesota in Minneapolis, addresses serviceability issues, primarily short and long-term deflections, for composite beam floors designed to American specifications. According to the

Roberto T. Leon

paper, the trend towards limit state design codes for steel and composite structures has resulted in an emphasis of strength requirements over serviceability requirements. As a result, LRFD specifications can produce substantial economies in materials (10% to 15%) for LRFD-designed composite beams over ASD-designed beams.

However, because many of the savings come from utilizing very shallow sections over long spans, the question is raised as to potential serviceability problems. The paper addresses these concerns and describes the results of tests conducted to determine the effects of: cambering and shoring; creep and shrinkage; and end restraint on deflections of slender composite girders. The paper also offers recommendations and guidance on how to calculate deflections for composite floors. □

Reducing Products Liability Insurance Costs

What factors cause increases in insurance premiums and what can be done to reduce premiums? Several experts will participate in a panel discussion to help fabricators deal with "Products Liability Insurance."

Representatives from a major insurance company, a national brokerage firm and a steel fabricator will discuss how fabricators should pursue insurance coverage to insure the lowest prices. A question-and-answer period will follow the presentation.

Insurance purchasing is a major businness decision and cannot be taken to lightly. Regardless of firm size, attendees will leave this session with cost-saving tips easily implemented in their business. The session is designed to present a common-sense approach to the topic of purchasing products liability insurance.

The moderator of the panel is Morris H. Caminer, AISC, and speakers will include representatives from the CNA Insurance Co. and the Hiatt Agency, Inc. □

HOTEL RESERVATION FORM

MAIL COMPLETED FORM DIRECTLY TO HOTEL SELECTED. RESERVATIONS MUST BE ACCOMPANIED BY ONE NIGHT ROOM DEPOSIT: HOTELS ACCEPT DEPOSITS MADE BY CHECK OR BY AMERICAN EXPRESS, VISA, MASTERCARD, DINERS and DISCOVER. NOTE: Amenities will vary; you may wish to call individual hotels for information on complimentary breakfasts, cocktails, etc., which are included in some rates. If reserving rooms by phone, advise hotel you are attending the **AISC National Steel Construction Conference.**

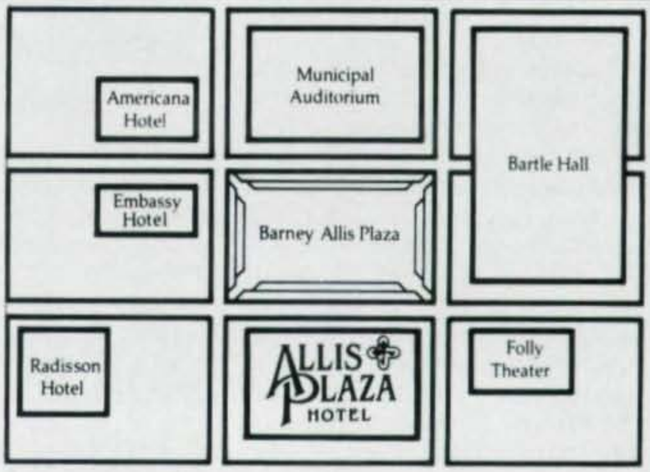
Check (X) Hotel Selected and Circle Room Rate Under Room Type Selected:

Hotel Selection	Single	Double/ Double (2 beds)	Double/ King/Queen	Studio Suite	1-Bdrm. Suite	2-Bdrm. Suite
__**Allis Plaza** 200 W. 12th St. Kansas City, MO 64105 Phone: 816/421-6800	$79	$79	$79	N/A	$260	$335
__**Radisson Suites** 106 W. 12th St. Kansas City, MO 64105 Phone: 816/221-7000	68	80	80	78*	88*	N/A
__**Embassy on the Park** 1214 Wyandotte Kansas City, MO 64105 Pone: 816/471-1333	62	74**	74	N/A	88*	N/A
__**Americana Hotel** 1301 Wynadotte Kansas City, MO 64105 Phone: 816/221-8800	55	65	65	N/A	N/A	N/A

Special Requirements: ______________________________

*Rate quoted is for single occupancy; if additional person in room, add $12. Rates do not include room tax of 9¾%.
**Limited Availability.
N/A: None available.

Hotels will honor and guarantee reservations received by February 12, 1990, so mail this form promptly. Refunds will be made only when cancellations are received at least 24 hours prior to scheduled arrival.



Please reserve the accommodations indicated above for:

Guest Name: ______________________ #Adults ________ #Children ________

OR sharing room (dividing bill) with: ______________________

Organization or company: ______________________

Mailing Address: ______________________

City: ______________________ State: ________ Zip: ________

Phone Office ( ) ______________________ Home ( ) ______________________

Arrival Date: ____________ Approx. Arrival Time: ____________ Departure Date: ____________
(Check-in time at all hotels is 3:00 PM; check-out by noon.)

I enclose check for $ ______ payable to Hotel selected **OR**

Please charge deposit to my Credit Card # ______________________
(Circle card used:) American Express VISA MasterCard Diners Discover Carte Blanche

Expiration Date: ______________________ Signature: ______________________

(For information only, call Lona Babbington at AISC, Phone: 312/670-5432)

Construction Claims Knowledge Proves Important For Fabricators

Construction claims is a complex process, but one which can be simplified if the necessary steps are taken. In "A Steel Fabricator's Recipe for Construction Claim Sausage," David B. Ratterman of Goldberg & Simpson, P.S.C., the general counsel for the AISC, describes the "recipe" for a successful claims process.

The needed steps range from simply being more aware of contractual language and obligations to conducting an in-depth analysis to determine what caused the problem in the first place.

The paper also discusses various resolution procedures, including negotiation, arbitration and litigation. The paper forms the basis of a scheduled panel discussion. The moderator for the panel is Robert B. Nelson, AFCO Steel, Little Rock, Ark. The other participants are James R. Jones, Havens Steel Co., Kansas City, and Frank Goldenberg, Mantague-Betts Co., Inc., Lynchburg, Va. □

Advice for the First-Line Supervisor

Participatory management teaches supervisors and managers to build cooperative relationships with employees. This panel discussion, moderated by Robert H. Woolf, Cives Steel Company, Roswell, GA, is designed to give managers the information they need to implement such a program.

Dorman S. Conklin, Employee Development Services, Jackson, MS, will show managers how to nurture the environment needed for producing a quality product or service. James E. Self, Cives Steel Co., will discuss employee motivation, the legal restrictions a supervisor must be aware of in managing today's workforce. □

Elastic Analysis Shows Its Practical Side

Several new studies on frame analysis using LRFD may have substantial benefits for structural practitioners.

"Application of Second-Order Elastic Analysis in LRFD: Research to Practice," by D.W. White and J.F. Hajjar of Purdue University, emphasizes the practical use of matrix analysis approaches in a design office setting. The paper includes: one-step strategies for obtaining second-order elastic forces; live-load reduction and use of second-order elastic analysis; second-order elastic analysis of large structural systems; and representation of three-dimensional leaner column effects.

"One Approach to Inelastic Analysis and Design," by Ronald D. Ziemian, Gregory G. Deierlein and William McGuire, all of Cornell University, and White, describes the development of an integrated analysis and design system based in an interactive graphics medium.

The paper presents a few of the present and future possibilities for inelastic analysis.

"An Inelastic Analysis and Design System," by Deierlein, White and McGuire, outlines the internal features of computer programs for the application of interactive computer graphics to the inelastic analysis and design of steel structures. Included are brief descriptions of the finite element (beam-to-column) models used, methods for modelling non-linear geometric effects, methods for modelling inelastic effects using concentrated plasticity (plastic hinge) formulation, schemes for accounting for imperfections, residual stresses, and variability of resistances, and non-linear analysis solution procedures.

"Studies in Inelastic Analysis and Design" by Ziemian, White, Deierlein and McGuire, takes the topic of the previous paper one step further and discusses: verification of the computer programs; the influence of second order effects; differences between two- and three-dimensional behavior; and member versus system behavior in limit state design.

"Ideas on Inelastic Systems Design," by McGuire and Deierlein, is intended to take stock of where the structural engineering profession stands in the use of inelastic design concepts and where it might be going.

McGuire is panel moderator. □

Design Aids For Standard Details and Connections

A discussion of the new design aids for pre-engineered web bolted connections in buildings from the AISC will be presented by Cynthia Zahn, AISC. The new version is similar to the 7th Edition format with values given for A36 and A572 Gr. steel beams. For a specific beam strength, bolt size and number of horizontal bolt rows, three tables are given and the allowable end reaction is the lower of the three resulting capacities.

Charles L. Chambers

Also, Charles L. Chambers, director, Region 3 Office of Structures, FHWA, will present a paper on "Economical Standard Specifications and Details for Bridges." While most state highway agencies have developed standard specifications and details for bridges, these may vary widely from state to state. Chambers will discuss a committee effort currently underway to review these variances and decide if economies are possible through regional standardization and discuss what effect these initial efforts have had so far. □

Partially Restrained Connections

European and American speakers are scheduled to discuss recent developments in the theory and practice of Partially Restrained Connections in LRFD.

R. Zandonini and P. Zanon of the Department of Structural Mechanics, University of Trento, Italy, are presenting a paper called "Beam Design in PR Braced Steel Frames." The paper presents the main features of a method that allows the limit state analysis of partially end restrained beams. Also, a

Continued on page 72

Fire Protection of Steel Frames Is Advancing In Both The U.S. And Australia

In the design of fire protection for steel buildings, there has been much confusion in interpreting the restraint provisions of ASTM E-119 and deciding which structural members are restrained and which are unrestrained. This issue is important because E-119 requires greater amounts of fireproofing for assemblies categorized as unrestrained.

"Effect of Restraint Conditions on Fire Endurance of Steel Framed Construction", by Robert H. Iding, consultant, Wiss, Janney, Elstner Associates, Emeryville, Calif., and Boris Bresler, an affiliated consultant with Wiss, Janney, Elstner, proposes guidelines to aid designers of fireproofing for steel framed buildings in identifying restrained and unrestrained construction.

As a first step in providing quantitative guidelines for degree of restraint, a series of analytical studies using the nonlinear computer program FASBUS II were made. In particular, the performance of steel beams and floor assemblies connected by single plate "shear tab" framing connections was compared to the performance of restrained and unrestrained specimens in a fire test. It was found that only moderate amounts of rotational restraint are needed for an assembly to perform like a restrained specimen in a fire test.

"Load Combinations for Buildings Exposed to Fires", by Bruce Ellingwood, Johns Hopkins University, and Ross B. Corotis, present an improved methodology for determining loads and load combinations for use in fire-resistant structural design. The basic methodology is illustrated by determining load combinations involving fire and live loads for limit states design and for use with the standard ASTM E-119 fire test.

Robert H. Iding

Kathleen Almand

Initial results indicate that a reduced nominal live load along with the structural action due to fire provides a load combination that is risk-consistent with other currently accepted load combinations.

"Design for Structural Fire Endurance—Australian Case Studies", by K. H. Almand, AISI, and I. D. Bennetts and I.R. Thomas, both of BHP Melbourne Research Laboratories, reviews the unique design approaches adopted for several recently completed high rise buildings in Australia. The paper illustrates the principles of fire safety design and includes: load combinations with fire; design for real fire exposure; and composite column behavior in fire. The acceptance of structural fire design procedures in Australia and their potential in North America also is discussed. □

P.R. Connections, Cont.

limited series of examples of its use in design practice are presented.

"Computer-Aided Design of Steel Structures with Flexible Connections," by Gregory G. Deierlein, Shang-Hsien Hsieh and Yi-Jiun Shen, all of Cornell University, describes the development and application of a computer-aided design system for including semi-rigid connection behavior in the analysis and design of two- and three-dimensional buildings. The system utilizes interactive computer-graphics to provide a convenient means of defining nd characterizing joint behavior for design.

Seminar moderator is Robert F. Lorenz, AISC. □

Economical Use Of Cambered Steel Beams

Jay W. Larson

Robert K. Huzzard

While the use of lighter, high-strength steel beams spanning greater distances produces more economical steel frames, it also result in larger deflections to be accommodated.

The overall economy and quality of a steel-framed building depends partially on the method used to compensate for deflections of the beams during the placement of the concrete slab. Current practice ranges from cambering or shoring beams to placing extra concrete above the deflected beam. For many buildings, cambering is the most cost effective solution. And while it could also result in the most level floor, some misconceptions regarding cambering still exist.

In "Economical Use of Cambered Steel Beams," authors Jay W. Larson, P.E., and Robert K. Huzzard, P.E., both of the Bethlehem Steel Corp., offer guidelines to assist in evaluating the cost effectiveness of cambering, correctly determining expected beam deflections, understanding mill camber tolerances and limits, specifying camber properly, and maintaining quality during construction. To illustrate, the authors present several recent steel-framed building projects as examples. In addition, field measured data is presented to support the suggested guidelines.

The fabricator's viewpoint is presented by Lawrence A. Kloiber, L.L. LeJeune Co., Minneapolis. ☐

Recruiting Steel Detailers

One of the key limiting factors of a fabricator's ability to expand and be successful is the capacity to quickly produce accurate shop drawings. Moderator Terry Peshia, Garbe Iron Works, Aurora, IL, and speakers Ken Volte, Paxton-Vierling Steel Co., Omaha, and Leonard Ross, L.N. Ross Engineering Co., Atlanta, discuss the need for recruiting and training of qualified detailers to help solve this potential problem. ☐

Steel Plate Shear Walls Produce Substantial Savings

Mohamed Elgaaly

Vincent Caccese

Steel savings on high-rise buildings employing steel plate shear walls rather than moment-resistance frames have been as great as 50%. And when the alternative is reinforced concrete, steel offers reduced foundation costs.

However, to date, almost all steel plate shear walls are designed ignoring the post buckling strength of the plates. In addition, an over designed steel plate will result in overstressing the columns in the event of a major earthquake.

"Design of Steel Plate Shear Walls," by Mohamed Elgaaly and Vincent Caccese of the University of Maine at Orono, describes the results of extensive research work on the seismic behavior of thin steel plate shear walls and makes design recommendations. These studies have shown stable hysteresis loops with no strength degradation and high energy absorbing capabilities.

The research work includes testing of 10 quarter-scale models of three stories single-bay shear wall to 2% drift. Analytical models also were developed to predict the behavior of thin steel plate shear walls under cyclic loading, and studies were made to extend the experimental data base.

D. Stanton Korista, a partner with Skidmore, Owings & Merrill, Chicago, will present a paper titled: "Interaction of Cladding and Steel Frame." □

Tubular Connections Design

The choice of joint types and specific joint details have a significant effect on structures, especially for those designed with square or rectangular structural steel tubing. "Box-Tube Connections; Choices of Joint Details and Their Influence on Cost," by J. W. Post of J. W. Post & Associates, presents a matrix of choices with regard to the available connection geometry and joint details of box tubing.

The paper, which is aimed at architects, designers, detailers and fabricators, includes information on: pipe versus square or rectangular tubing; matched versus stepped box connections; gapped versus overlapping branch members; and the selection of joint details (i.e. complete joint penetration groove welds, partial joint penetration groove welds, or fillet welded connections).

In a companion discussion, Frederick J. Palmer, director and consulting engineer for the newly formed American Institute for Hollow Structural Sections, will review the current state of tubular member design, design criteria, manufacture and typical applications. In addition, he is scheduled to present an update on his associations activities and goals. □

Composite Joists Help Bring Project In Under Budget

Long span composite joists have great potential for use as a lightweight steel alternative in long span applications, especially in commercial construction where large column-free areas are desired by many owners.

"Long-Span Composite Joists," by Kurt D. Swensson, Ph.D., P.E., of Stanley D. Lindsey and Associates in Nashville, outlines the design and construction of the Sovran Bank Building in Knoxville, Tenn. The building is a six-story precast municipal parking structure with an eight-story office building above. It was originally designed as a cast-in-place concrete structure, but the design came in over budget. Switching to composite joists in a steel alternative for the office building brought the project in under budget, however.

This paper describes the special aspects of the design, specifications and construction of the system.

In another paper, "Tests of Two 38' Span Composite Trusses," D. J. Laurie Kennedy, Ph.D., of the Civil Engineering department at the University of Alberta, Edmonton, Canada, gives test results and makes design recommendations for the use of light steel trusses.

Relatively light steel trusses acting compositely with concrete slabs cast on wide-rib profile steel deck can provide an economical system for long span floors. Fabrication is further simplified when double angle web members are welded directly to either side of rectangular hollow structural section top and bottom chords.

This paper describes shrinkage and destructive tests performed on two essentially identical composite trusses designed on the premise that the internal couple is developed solely by the bottom chord in tension and the concrete slab in compression. □

Economical Bridge Design

Timing is always critical on infrastructure projects, and this seminar presents papers on two bridge projects that both finished ahead of schedule.

Two alternatives for the three level interchange for I-95/I595 in Ft. Lauderdale, FL, were considered: Steel and Concrete. Bogdan O. Kuzmanovic, P.E., vice president and director of special projects for Beiswenger, Hoch and Associates, Inc., North Miami Beach, describes the structure and why steel was chosen. Construction is nearly complete and is several months ahead of the contract's closing date.

Another interesting project was the dual 4,223' bridges that carry Virginia Route 150 over the James River, the Kanawaha Canal and railroad tracks of CSX Transportation. Three members of the project design team from Hayes, Seay, Mattern & Mattern, Inc., Roanoke, VA, (Steven J. Chapin, P.E., James L. Fowler, P.E., and Robert H. White, P.E.) present a paper focusing on the bridge superstructure, which consists of a 9.5" reinforced concrete deck supported by four lines of A572, Grade 50 steel-plate girders. Also discussed are various construction and steel fabrication methods that were used to facilitate the project schedule. This bridge structure was opened to traffic in December, 1989, a full six months ahead of schedule. □

Exhibitor Products

Associated Industrial Technologies Corp.

Contemporary architecture and design is once again calling on curved structural steel for its grace, beauty and design flexibility. But unlike pre-turn-of-the-century cast iron, which was created in the intense heat of a blacksmith's forge, today's curved structural steel is created using induction bending by such firms as Associated Industrial Technologies Co. (formerly Associated Piping and Engineering Co.). Induction bending, such as was used on the architectural steel for the recently renovated American Airlines terminal at Chicago's O'Hare International Airport, offers a number of advantages over other techniques, including: exceptional dimensional stability; improved accuracy; and improved process control. See AITC at Booth 215 or for more information, write AITC, 851 S. Freeport Industrial Parkway, Clearfield, UT 84015-4209 (800) 453-2170.

Design Data

Greater speed, support for higher resolution graphics and more powerful computers, and new features providing increased flexibility and speed are the hallmarks of the SDS/2 Detailing Module Version 5.0 from Design Data. The software, designed to be run on newer Model 332, 340, 360 and 370 Hewlett Packard computers, supports higher-resolution graphics. See Design Data at Booth J or for more information, write: Design Data, 1033 O St., Suite 324, Lincoln, NB 68508 (402) 476-8278.

D.C.A. Engineering Software, Inc.

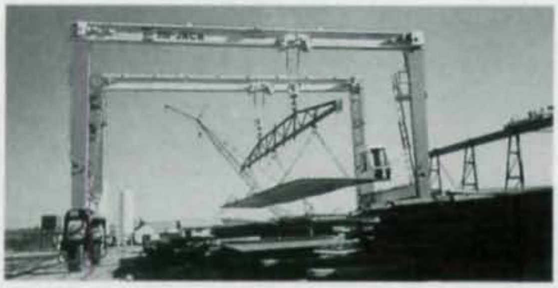
The Steel Detailer, D.C.A.'s latest introduction, is an advanced drawing production system that operates inside AutoCAD for the steel detailer. The program, which supports both metric and imperial units, produces complete shop fabrication drawings of beams, columns, bracing and anchor bolts. It also creates erection plans, elevations and anchor bolt setting plans. Standard steel data bases for American, Canadian, British and European countries provide intelligent detailing systems inside AutoCAD. Included in the system are interactive bill of material programs, with weight calculations as well as field bolt and project summary reports. For more information, write: D.C.A. Engineering Software, Inc., P.O. Box 955, Henniker, NH 03242 (603) 428-3199.

EGYPT Structural Steel Processing Corp.

The firm, which supplies cut, drilled, and coped beams and subcontracts fabrication services to structural fabricators will conduct a Product Service Workshop on Wednesday, March 14 at the conference prior to the Welcome cocktail party. The product workshop will advise fabricators how to increase productivity and profitability utilizing cut, drilled and coped beams. In addition, the firm's recent expansion of its facilities and new processing service capabilities will be introduced. See EGYPT at Booth 102 or for more information, write: EGYPT, 480 Osterloh Road, Minter, OH 45865 (419) 628-3893.

The Lincoln Electric Co.

Questions about structural design and innovative ways to reduce welding costs without any additional capital expenditures will be answered by Lincoln representatives at the National Steel Construction Conference. The firm's design engineers will be available to discuss how Lincoln Innershield self-shielded FCAW welding electrodes can be used on any outside or inside application; the advantage of Lincoln Outshield gas-shielded FCAW welding electrode for shop fabricating; and Lincoln's complete line of submerged arc fluxes and electrodes for SAW welding of carbon, low alloy, and higher strength steels. See Lincoln at Booth 207 or for more information write: The Lincoln Electric Co., 22801 St. Clair Ave., Cleveland, Ohio 44117-1199 (216) 481-8100.



Mi-Jack Products

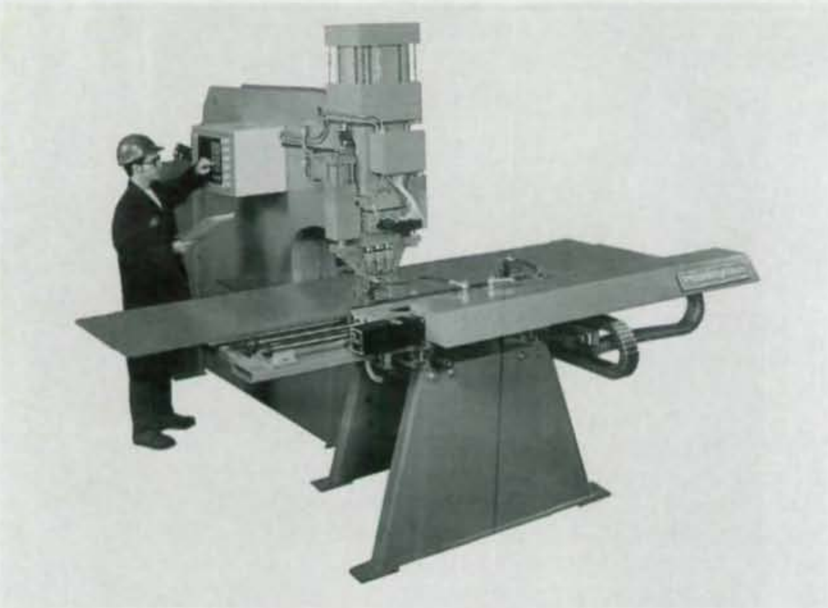
The company has been involved in the Intermodal Industry for more than 20 years and is responsible for more than 500 pieces of container handling equipment. Pictured is a Travelift. Travelift capacity ranges from 5 to 325 tons, and dimensions are customized to individual users specifications. New, remanufactured and used equipment is available for both purchase and rental. See Mi-Jack products at Booth 136 or for more information, write: Mi-Jack Products, 3111 W. 167th St., Hazel Crest, IL 60429 (312) 596-7461.

LOHR Structural Fasteners

The firm has recently opened a new world class domestic manufacturing plant to produce high-strength fasteners (A325 & A490). LOHR's product line includes tension control, hex head and 2H nuts. See LOHR at Booth M or for more information, write: LOHR Structural Fasteners, P.O. Box 1387, Humble, TX 77347 (713) 821-3509.

Nucor Fastener

The company's products include grade 2, 5 and 8 hex head cap screws and A325 and A490 structural bolts and nuts. Nucor makes cold-formed products in diameters from 1/4" through 11/4' and lengths from 1/2" through 10" In addition, Nucor produces hot-forged bolts in diameters from 3/4" through 21/2" in lengths up to 22". Due to automated operations, extensive use of robotics and employee pay incentives, the company claims to have met its goal of being the lowest cost manufacturer of standard commercial fasteners in the industry. For more information, write: Nucor Fastener, P.O. Box 6100, St. Joe, IN 46785 (219) 337-5611.



Peddinghaus Corporation

The ABCM 1000 4-C applies the latest in CNC technology to efficiently burn structural shapes, which results in a dramatic reduction in man-hours per ton for the structural steel fabricator. The ABCM 1000 4-C uses a series of probe mechanisms to determine the exact flange and web location for exacting cuts. In addition to such standard burn shapes as copes, blocks and notches, the ABCM has the capability for flange weld preparation as well as cut-to-length, beam splitting, haunch and castellation. The firm will also display its CNC Anglemaster product line and the Fabripunch (pictured above) 1154-30 CNC Detail Punch. See Peddinghaus at Booth N or for more information, write: Peddinghaus Corp., 300 N. Washington Ave., Bradley, IL 60915 (815) 937-3800.

Pangborn Corporation

The firm, which is the world leader in surface finishing and blasting cleaning for the basic and fabricated steel industry, will display models showing structural and plate descaling machines. Also, a video showing actual operating equipment cleaning a variety of products will be available. Brochures and product data sheets describe the variety of steel descaling systems Pangborn can provide. See Pangborn at Booth P or for more information, write: Pangborn Corp., P.O. Box 380, Hagerstown, MD 21741-0380 (301) 739-3500.

The Rawls Company

This firm specializes in financial advice for family-owned businesses. In addition to exhibiting, the company will present a seminar on succession planning for family-owned businesses. Other areas of expertise include: estate planning and tax financing, business structuring, executive benefits and retirement plans, group employee benefits, investment brokerage and life insurance brokerage. See The Rawls Company at Booth 212 or for more information, write: The Rawls Company, 111 North Orange Ave., Suite 1075, Orlando, FL 32801-2316 (305) 839-3075.

Tnemec Company, Inc.

For an extended painting season or fast steel fabricating shop turnaround, the Tnemec Co. announces a new low-temperature cure epoxy system. Tneme-Fasprime and Tneme-Fascure will cure at ambient temperatures as low as 35° F. *Both can be recoated quickly in* normal temperatures. At 65° F they can be recoated in just five to six hours, and at 75° F and above, the substrate can be recoated within three hours. The new fast-cure epoxy coatings are abrasion- and moisture-resistant, making them ideally suited for structural steel, tank exteriors and equipment. See Tnemec at Booth 148 or for more information, write: Tnemec, P.O. Box 411749, Kansas City, MO 64141 (816) 483-3400.

Structural Software Co.

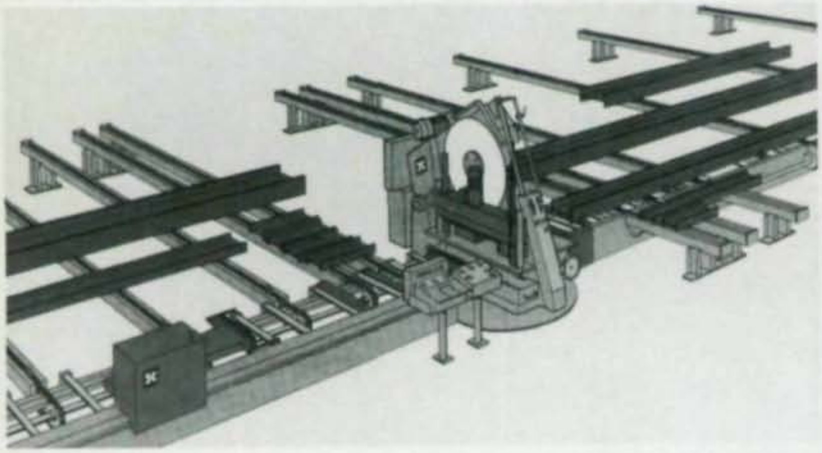
Representatives from this firm will be demonstrating a totally integrated system, including the release of FabriCad II, the automated detailing system, as well as the new Material Allocation System. Each of Structural Software's programs, which are used by more than 350 fabricators, has undergone major changes since last year's conference. See Structural Software at Booth F, or for more information, write: Structural Software Co., 5012 Plantation Road N.E., P.O. Box 19220, Roanoke, VA 24019-1022 (703) 362-9118.

Vernon Tool Company

The company is best known for the automated pipe cutting machines it has manufactured for more than 40 years and its line of automated pipe cutters and beam profilers manufactured since the early 1980s. Last year, Vernon provided the first automated beam profiling machines of their type ever installed in Europe. The profilers burn all shapes in H-beam, channel, flat stock, and rectangular tubing. See Vernon at Booth 145 or for more information, write: Vernon Tool Co., 503 Jones Road, Oceanside, CA 92054.

Other Exhibitors

Amer. Welding Institute	202
Amer. Welding Society	103
Armstrong Blum Manuf.	218
AISC Marketing, Inc.	132
Cleveland Tool Co.	101
Cold Saws of America	213
Dogwood Technologies	G
Franklin Manufacturing	
Geometric Data Flow	139-141
Haydon Bolts	105
IKE/Greulich	223
Inorganic Coatings	118-120
Intergraph Corp.	142
JH Engineering	137
Kaltenbach	144
Lake Erie Screw	147
LeJeune Bolt	146
Metalmizer	A
Mountain Enterprises	E
Nat. Inst. of Steel Detailing	112
Pettitt Lawrence (Steelcad)	I
Research Council on Struct. Connections	133
St. Louis Screw & Bolt	113
Silver Collar Systems	201
Steel Solutions	138
TradeARBED	C
J & M Turner	143
User Friendly Computers	217
Welded Tube Co. of Amer.	214
W. A. Whitney	U
Yamazen	O



Kaltenbach

The world's largest manufacturer of circular old sawing machines has introduced two models to its line of structural circular cold saws. The all new HDM-1000 and HDM-1400 structural saws are designed for use in a "tandem system" with Kaltenbach's structural CNC drill. Features include a travelling saw arm and fixed datum fence to include maximum capacity of cutting range. See Kaltenbach at Booth 144 or for more information, write: Kaltenbach, Inc., P.O. Box 1629, 6775 Inwood Dr., Columbus, IN 47202 (812) 342-4471.

Dogwood Technologies

To achieve the same technological superiority common to the high-tech leaders in the fields of aerospace and communications, the steel industry is moving away from MS-DOS systems in favor of Technical Engineering Workstations powered by UNIX. Dogwood Technologies' system is available in cost effective modules to form a single integrated system supported by a single dedicated vendor. In addition, PDS Detailing Systems provide the user with the diversity to meet the needs of a demanding detailing environment. See Dogwood Technologies at Booth G, or for more information, write: Dogwood Technologies, P.O. Box 52831, Knoxville, TN 37950-9928 (615) 523-5634.

TradeARBED

Histar Steel, a new generation of rolled beams and column shapes for economical construction, have been introduced by TradeARBED. To achieve this, the company has developed a revolutionary new "QST" process in line after the last rolling pass. According to the manufacturer, this new development, which features a highly weldable, high-strength steel with up to 65 ksi, improves both the performance and cost effectiveness of steel products. Histar features a trend-setting combination of mechanical, chemical, and technological properties considered incompatible up to now. See TradeARBED at Booth C, or for more information, write: TradeARBED, 825 3rd Ave., New York, NY 10022 (212) 486-9890.

Advertisers' Index

A
AISC Manual of Steel Construction 57
AISC Marketing, Inc. ..... Cover IV
American Computers 62
ASC Pacific 11

B
Bethlehem Steel 8-9
Bouras, Nicholas J. ..... Cover 2

C
C.A.S.T. 79
Chaparral Steel 51
Cleveland Steel Tool 60
Computer Detailing 60
Computer & Structures 16

D
Data Management Systems 37
Design Data 41
Dogwood Technologies 54

E
ECOM 80
Epic Metals ..... Cover III

G
Geometric Data Flow, Inc. 44

J
Jancy Engineering 78
Jobber Instruments 71

L
LeJeune Bolt 27
Lincoln Electric 13
Lohr Fasteners/Unytite 40
L-Tec 61

M
Mountain Enterprises 15
MCAE 81

N
National Metal Abrasives 77
NISD 73
Nucor Fasteners 52

O
Omnitec 78

P
Platt Bros. 56
Prairie Technologies 74
Precision Programming 80

R
J.C. Renfroe 32
Research Engineers 28
Riopelle Engineering Sales 50
RISA Technologies 81

S
St. Louis Screw & Bolt 55
Smith-Emery 32
Steelcad 5
Steel Deck 75
Steel Solutions 58
Structural Analysis 61
Struct Fast, Inc. 79

T
TradeARBED 21
J&M Turner, Inc. 33
TRW 7

U
User Friendly 77

V
Valmont 69
Vulcraft 42-43

W
Walker Manufacturing 23
W.A. Whitney 70

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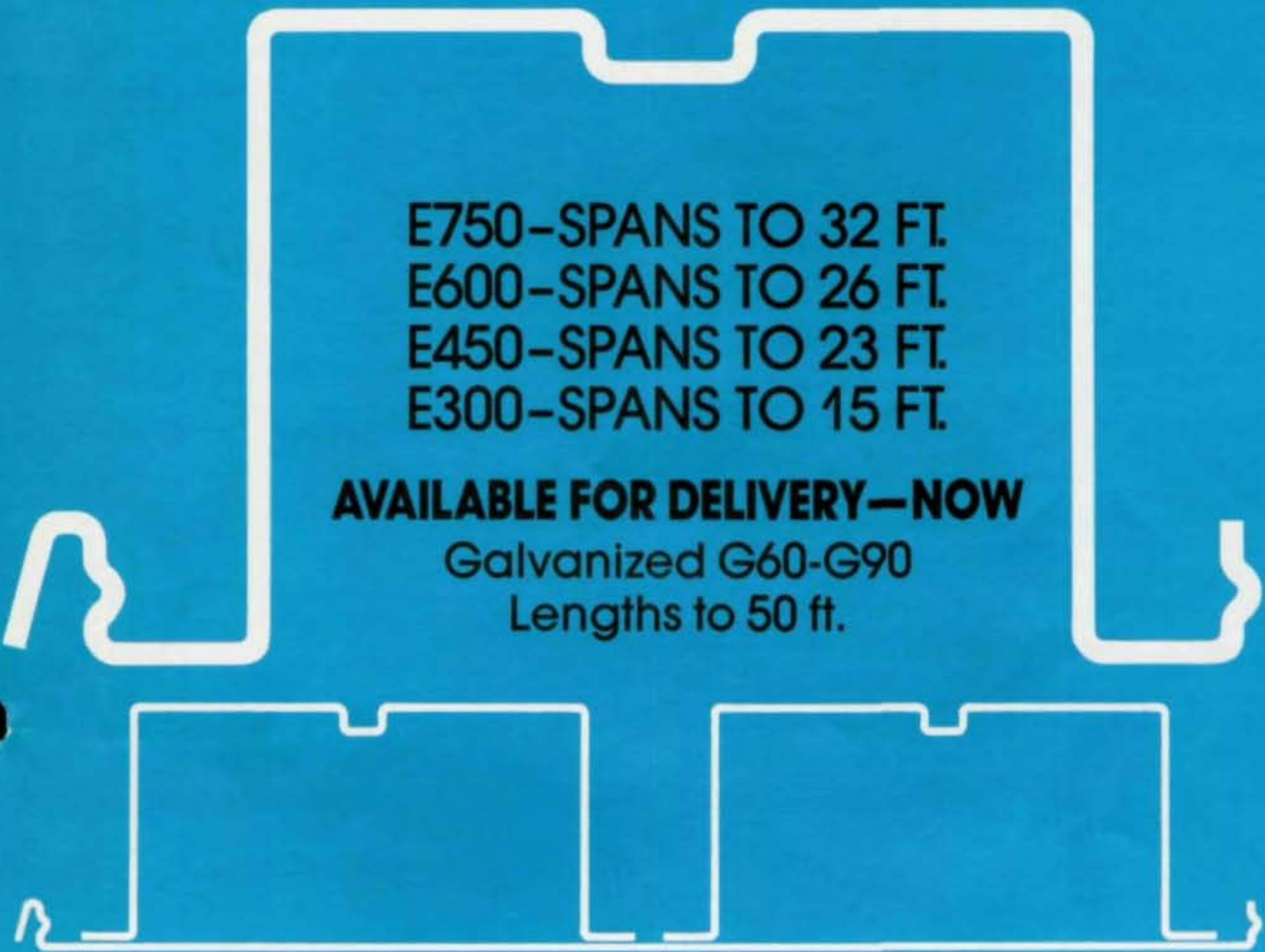

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