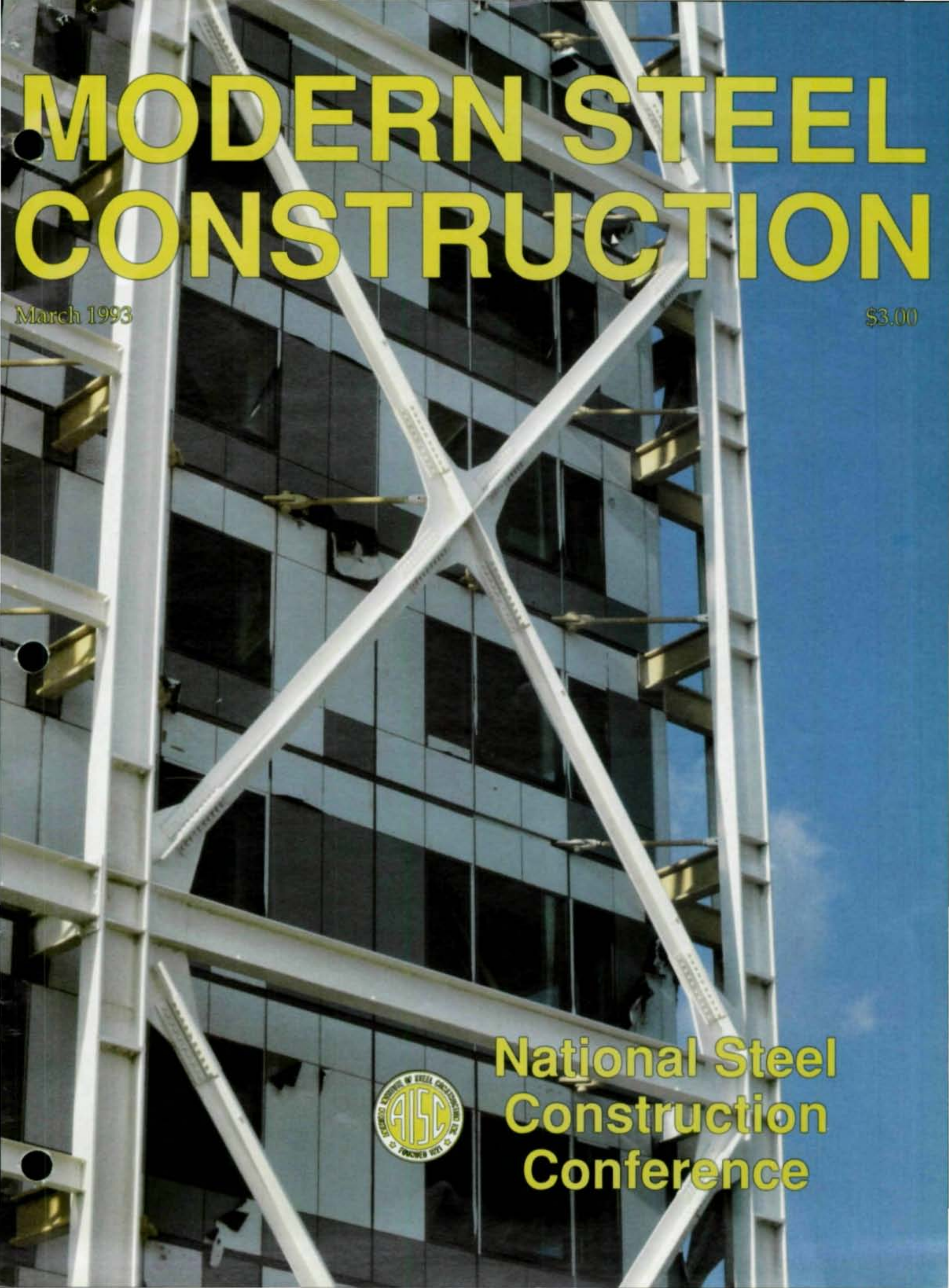
MODERN STEEL CONSTRUCTION

March 1993

$3.00



National Steel Construction Conference

NEW COLUMBIA JOIST CO.

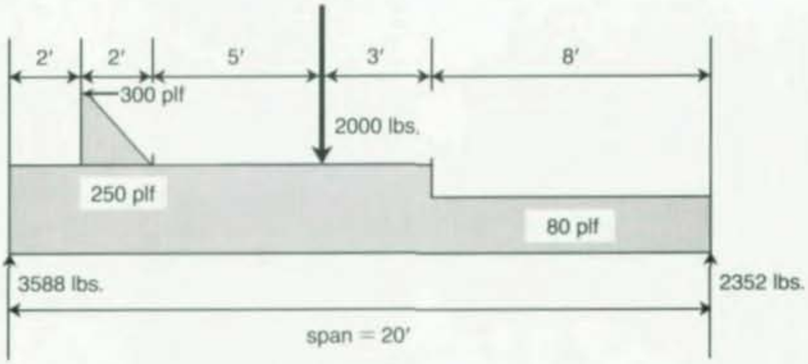
P.O. Box 31 • New Columbia, PA 17856-0031
717-568-6761 • 717-568-1001 (FAX)

JOIST DESIGN DATA SHEET

No. 1

HOW TO SPECIFY SPECIAL JOISTS

To obtain correct and competitive bids the specifying professional must show any special loading on the structural drawings. Special loadings could include concentrated loads and any non-uniform load distribution such as:



The maximum moment for this loading is 20267 lb. ft. The equivalent uniform load to produce this moment would be 405 lbs. per ft.

The maximum end reaction is 3588 lbs. The equivalent uniform load to produce this reaction would be 359 lbs. per ft.*

The K series joist tables show a 16K3 (with a 20′ span) has a total load capacity of 410 lbs. per ft. The designer should show a 16K3SP on the drawings.

The SP indicates special requirements for the joist. The joist manufacturer will review the designated joist for its ability to carry the special loads shown.

Joist Girders with unequal panel point loads must also be defined by showing the load diagram on the structural drawings.

*For all standard K series joists the maximum end reaction is 8700 lbs. If more reaction capacity is needed, consider using two (or more) joists to share the load. For LH and DLH joists a conservative end reaction can be found by dividing the tabulated SAFE LOAD by two.

NICHOLAS J. BOURAS, INC.
P.O. BOX 662, 475 SPRINGFIELD AVE.
SUMMIT, NEW JERSEY 07902-0662 (908) 277-1617

00394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A new generation of rolled beams and column shapes for economical steel construction.
Once again, ARBED leads the industry by featuring a trendsetting combination of mechanical, chemical and technological properties:
• HIGH YIELD STRENGTHS (up to 65 KSI) - even for ultra-heavy sections.
• OUTSTANDING TOUGHNESS PROPERTIES.
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The secret is in ARBED's revolutionary new in-line QST process.
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ARBED-ROLLED 40", 44", and "TAILOR-MADE" (WTM) series — famous for high section moduli, great lateral buckling resistance, and big savings in fabrication costs and weights. These products are also available in the new HISTAR quality as is our standard WF series and H BEARING PILES.
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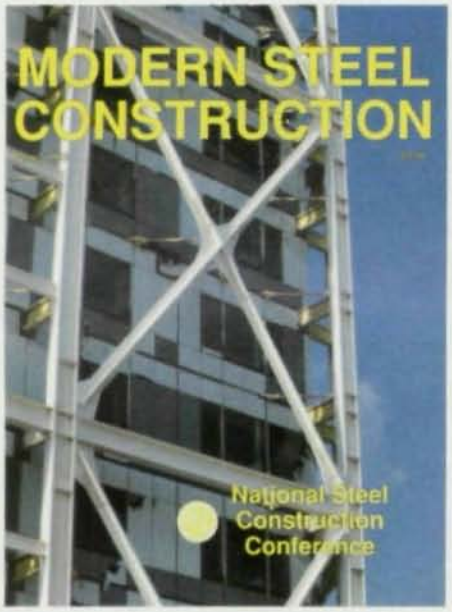
MODERN STEEL CONSTRUCTION

Volume 33, Number 3 March 1993



In order to create an exoskelaton form on the Hotel de las Artes complex, the braced frame was disassociated and pulled away from the plane of the window wall. To find out more about this innovative project, *turn to page 18.*

NSCC FEATURES

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NEWS AND DEPARTMENTS

Taking Chances

It was nearly a decade ago, but I can still remember reading in a leading trade magazine how the coming trend in the construction industry was design/build. And I can remember how some of my older and perhaps wiser colleagues at the time scoffed and pointed out that just about every decade people start talking about design/build as an up-and-coming trend.

Unfortunately, it always seems to be just talk.

I say *unfortunately* because for some types of construction, design/build can substantially reduce project costs. Yet few people seem willing to devote the effort needed to make the procedure work.

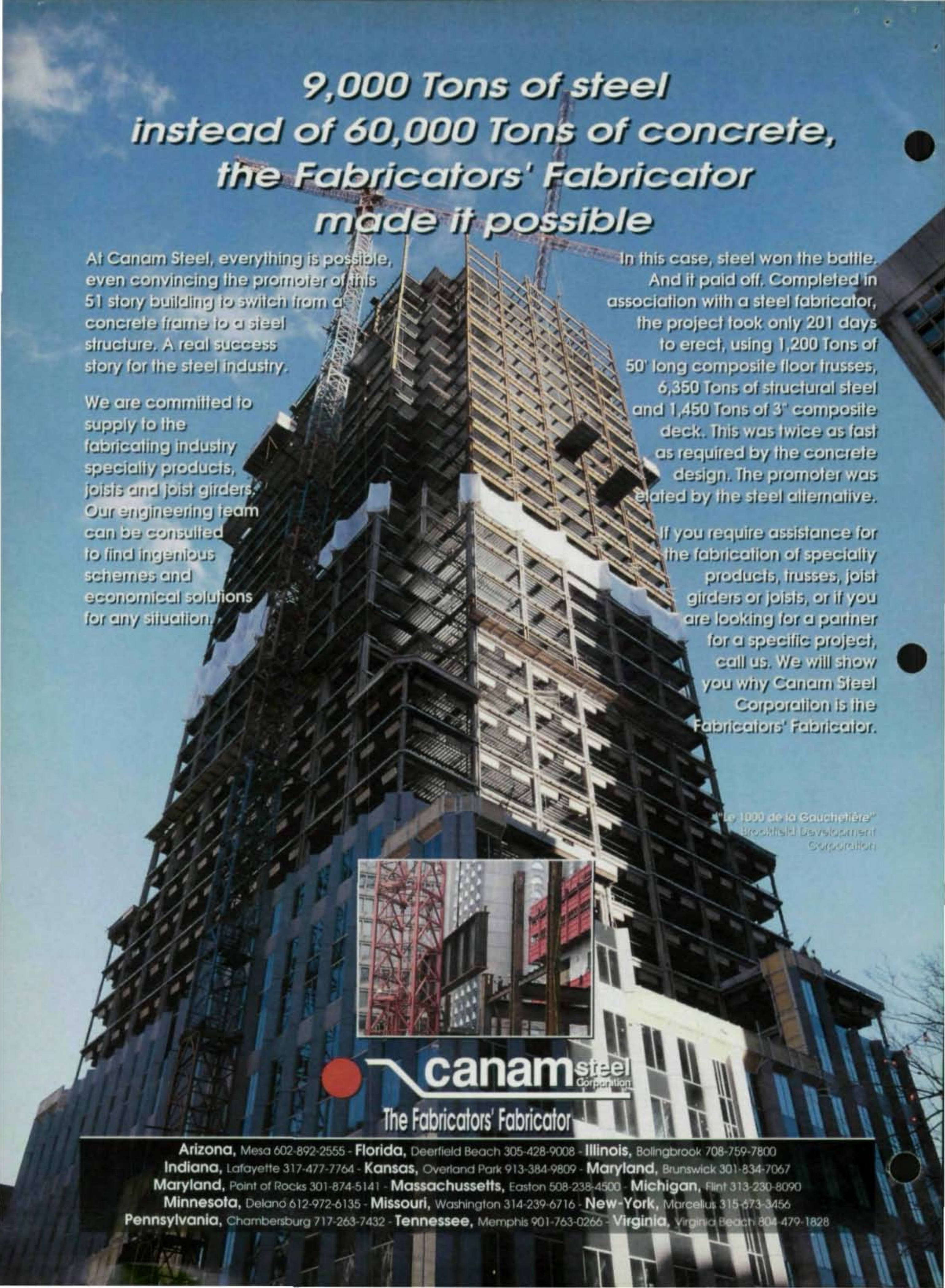
On page 47 of this issue is a detailed description of a new parking structure system design by Mulach Parking Structures. What is implied—but never actually stated in the article—is that to make the system work the steel fabricator must be willing to assume lead responsibility. Essentially, the project must be bid and built using a design/build procedure.

Does the system work? Yes. Mulach has documented savings of nearly 15% compared to any conventionally designed and built parking structure.

So why aren't more firms involved in design/build? I'm afraid that the answer is a lack of willingness to change and adopt innovative procedures, and not just by fabricators but also by engineers and building owners. How many fabricators are truly taking advantage in the latest CNC and CAD equipment? How many engineers have completely switched over to LRFD? How many companies are willing to examine their management structure with an eye to making comprehensive changes?

Maybe I'm wrong. Maybe there are companies out there that have taken a serious look at design/build and have determined it to be ineffective. Maybe there are more than a half-dozen up-to-date fabricators out there willing to take advantage of the latest technologies to reduce cost and increase efficiency. And maybe the number of engineers keeping up-to-date on the latest design trends is growing. If so, I'd like to hear from them. Put pen to paper (or fingers to keyboard) and tell me about innovations within your firm.

I know the entrepreneurs and innovators are out there. I know the talent is out there. And I'd like to hear what you're doing. **SM**


9,000 Tons of steel
instead of 60,000 Tons of concrete,
the Fabricators' Fabricator
made it possible
At Canam Steel, everything is possible, even convincing the promoter of this 51 story building to switch from a concrete frame to a steel structure. A real success story for the steel industry.
We are committed to supply to the fabricating industry specialty products, joists and joist girders. Our engineering team can be consulted to find ingenious schemes and economical solutions for any situation.
In this case, steel won the battle. And it paid off. Completed in association with a steel fabricator, the project took only 201 days to erect, using 1,200 Tons of 50' long composite floor trusses, 6,350 Tons of structural steel and 1,450 Tons of 3" composite deck. This was twice as fast as required by the concrete design. The promoter was elated by the steel alternative.
If you require assistance for the fabrication of specialty products, trusses, joist girders or joists, or if you are looking for a partner for a specific project, call us. We will show you why Canam Steel Corporation is the Fabricators' Fabricator.
"Le 1000 de la Gauchetière"
Brookfield Development Corporation
canam steel Corporation
The Fabricators' Fabricator
Arizona, Mesa 602-892-2555 - Florida, Deerfield Beach 305-428-9008 - Illinois, Bolingbrook 708-759-7800
Indiana, Lafayette 317-477-7764 - Kansas, Overland Park 913-384-9809 - Maryland, Brunswick 301-834-7067
Maryland, Point of Rocks 301-874-5141 - Massachussetts, Easton 508-238-4500 - Michigan, Flint 313-230-8090
Minnesota, Delano 612-972-6135 - Missouri, Washington 314-239-6716 - New-York, Marcellus 315-673-3456
Pennsylvania, Chambersburg 717-263-7432 - Tennessee, Memphis 901-763-0266 - Virginia, Virginia Beach 804-479-1828

69397

Steel Interchange

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

Steel Interchange
Modern Steel Construction
1 East Wacker Dr.
Suite 3100
Chicago, IL 60601

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

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

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

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

Where can one get information on stainless steel bolts?

Information on stainless steel bolts can be found in at least two very useful design documents:

1. *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ANSI/ASCE 8-90), American Society of Civil Engineers, New York, NY, 114 pages. (Stock No. 794) (Phone 800 548-ASCE)

This document, which is an update of the 1974 AISI specification, gives nominal (ultimate) shear and tension stresses for several types (alloys) and conditions (annealed, cold-worked, etc.) of stainless steel fasteners. Recommended resistance factors are also given, consistent with the LRFD format. Stresses apply to bolts less than, and equal to or greater than 1/2 in. diameter.

2. *Metal Curtain Wall Fasteners* (AAMA TIR-A9-1991), American Architectural Manufacturers Association, Palatine, IL, 47 pages. (Phone 708/202-1350)

This second document, which is based on allowable stress design, is devoted entirely to various types of fasteners (up to 1 in. diameter). In addition to background and design examples, this booklet presents tables of allowable shear and tension values for seven different "alloy group/condition" combinations of stainless steel fasteners in fourteen diameters (#6 up to 1 in.)

This booklet also reprints the abstract of ASTM F593 which gives the mechanical properties for seven alloy groups and seven conditions. In addition, a table cross references "type" (e.g. 304 alloy) to "group" (e.g. group one).

James C. LaBelle, Doc.E., P.E.
Computerized Structural Design, Inc.
Milwaukee, WI

What are some references that address the design of "curved" beams (supporting members rolled to a radius) frequently encountered at the perimeter of buildings, canopies, etc. as well as highway bridges and overpasses? Sources dealing with hot rolled (WF, C, L, etc.) sections are preferred, although any information regarding built-up members would also be appreciated.

I offer the following sources of information regarding built-up members:

1. *Guide to Stability Design Criteria for Metal Structures*, 4th Edition, Edited by Theodore V. Galambos, John Wiley & Sons, Inc., 1988.

2. *Guide Specifications for Horizontally Curved Highway Bridges*, Published by the American Association of State Highway and Transportation Officials, Washington D.C.

3. *Analysis and Design of Curved Steel Bridges*, Hiroshi Nakai and Chai Hong "Jay" Yoo, McGraw-Hill Book Co., 1988.

4. *Highway Structures Design Handbook*, AISC Marketing, Inc., Chapter 12-Horizontally Curved Girders, Pittsburgh, PA.

Additionally, please note that the Federal Highway Administration and the National Research Council of the Transportation Research Board are planning/undertaking research in the area of horizontally curved steel girders:

NCHRP 12-38, *Improved Design Specifications for Horizontally Curved steel Girders*, and

FHWA Contract DTFH61-92-C-00136, *Engineering Services for Curved Steel Bridge Research Project*.

John M. Yadlosky, P.E.
HDR Engineering, Inc.
Pittsburgh, PA

Steel Interchange

The AISC Specification Section A4.2 includes increases to live loads which induce impact. What is an appropriate increase for jib cranes?

The publication of the Department of the Navy, Naval Facilities Engineering Command, Alexandria, VA entitled *NAVFAC DM-38, Design Manual, Weight-Handling Equipment and Service Craft*, has an excellent source of equivalent static increases in shock-load and hook-load reactions for crane impact loads in Section 6, Part 1, Subsection 2.d. Table 1-10 shows percentages of increases for impact for a revolving structure as 35% for 25 tons or less, 30% for 26 to 50 tons, 25% for 51 to 80 tons, 20% for 81-120 tons, and 15% for 121 tons or more.

Trolley impact percentages run approximately +15% higher than those values shown in Table 1-10 for the given revolving structure increases.

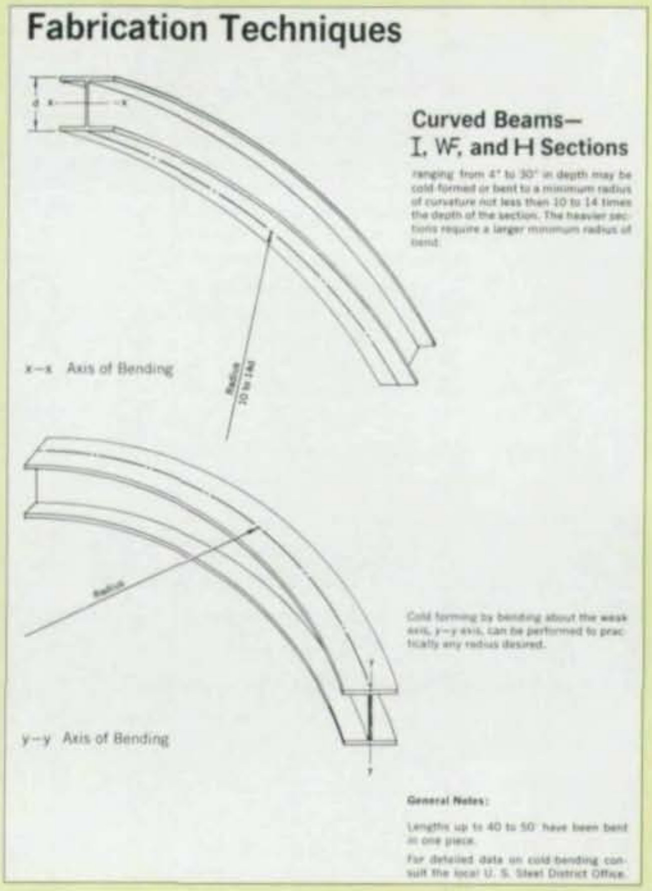
It should be recognized that special application jib cranes, such as for the nuclear industry or for severe service, may require more specific design factors. Likewise, hoist manufacturers may recommend different impact factors for various line speeds and hoist line brake systems.

Dale H. Curtis, P.E.
Curtis Engineering Corp.
National City, CA

Are there limits on bending a wide flange beam into a radius?

The figures shown (right hand column) were published by US Steel Corp. many years ago and can be used as a guide.

Heinz Pak
AISC Marketing, Inc.



New Questions

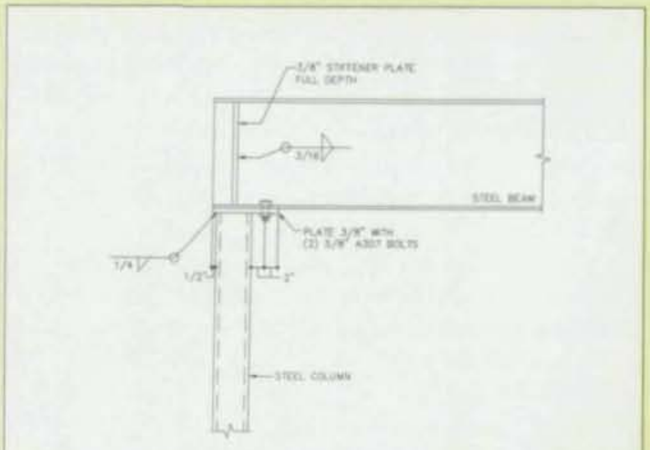
Listed below are questions that we would like our readers to answer or discuss. If you have an answer or suggestion please send it to the Steel Interchange Editor. Questions and responses will be printed in future editions of Steel Interchange. Also, if you have a question or problem that readers might help solve, send these to the Steel Interchange Editor.

Is it permissible to weld nuts to bolts to prevent then from backing off? Are any special welding procedures required? Is the bolt/nut strength affected?

William C. Sherman, P.E.
Denver, CO

When designing a horizontal beam resting on columns with an unbraced compression top flange, may full-height web stiffeners at the bearing ends provide bracing to the compression flange without any intersecting beams (see detail below)?

John W. Lawson
Kramer & Associates Structural Engineers, Inc.
Tustin, CA



COLUMN TO BEAM CONNECTION

New Ideas In Structural Steel

Beginning in March, AISC Marketing, Inc. will offer a new lecture series focusing on innovations in structural steel design. *New Ideas In Structural Steel* will present practical design concepts for engineers and fabricators.

The four-part seminar covers:

- Low-rise buildings.
- Design of Connections.
- Eccentric Braced Frames.
- Partially Restrained Connections.

Registration fee is $60 ($45 for AISC members). Included in the registration fee are a dozen handouts and publications plus a meal. For information, contact: Colleen Hays, AISC, Inc., One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001 (312) 670-2400.

WEST	Date	MIDWEST	Date	NORTHEAST	Date
Seattle	3/30	St. Louis	4/13	New York	4/29
Los Angeles	5/4	Detroit	5/11	Newark	6/22
Irvine	6/10	Indianapolis	5/13	Rochestor	9/22
Sacramento	6/22	Minneapolis	5/25	Albany	9/23
San Francisco	6/24	Milwaukee	5/27		
		Chicago	6/3		
SOUTHWEST	**Date**	**SOUTH**	**Date**	**MID-ATLANTIC**	**Date**
Denver	4/1	Greenville	4/20	Pittsburgh	4/27
Kansas City	4/15	Charlotte	4/21	Baltimore	5/18
Dallas	5/6	Birmingham	9/9	Washington	5/20
San Antonio	6/1	Miami	9/14	Philadelphia	6/23
Houston	6/8	Orlando	9/16	Cleveland	10/19
		Atlanta	9/28	Cincinnati	10/21
		Richmond	9/30		

"Down the road, weathering steel

The Pennsylvania Turnpike is considered one of the safest and best maintained roads in the nation.

But improvements were badly needed near Pittsburgh to provide better access to the city's airport and I-80.

The Mahoning River Bridge, the largest of the 21 bridges being built on the new Beaver Valley Expressway, is a dual-lane, continuous-span, welded plate structure. The five interior spans are 258 ft with end spans of 182 ft and 228 ft for an overall length of 1,700 ft.

WHY WEATHERING STEEL?

The bridge is built with 4,600 tons of Bethlehem's ASTM A588 weathering steel. According to Kempf, the PTC finds weathering steel a very cost-effective bridge material for a number of reasons.

For example, its high strength permits longer spans, thus reducing the number of piers required. And that leads to foundation cost savings.

OTHER STRONG REASONS.

For another, it can be easily inspected, measured and evaluated. If necessary, it can be readily repaired. It's also highly adaptable to redecking, widening and performing other structural modifications.

Weathering steel also eliminates the need for both initial and maintenance painting. What's more, it's as attractive as it's environmentally sound.

Kempf comments, "The PTC's principal criteria for selecting materials for bridges and other Turnpike applications aren't based on price alone but also include long-term serviceability and durability. And with environmental restrictions being what they are today, it's only prudent to use

will save us more than money."

Frank J. Kempf, Jr., P.E. Bridge Engineer, Pennsylvania Turnpike Commission.

weathering steel wherever possible."

Quite simply, weathering steel is a natural for a broad variety of bridge applications.

Including yours. Especially if you want to save more than money.

TECHNICAL LITERATURE AVAILABLE.

We would like to tell you more about weathering steel for bridge applications. For a copy of our Product Booklet No. 3790, and our latest Technical Bulletin, TB-307 on "Uncoated Weathering Steel Structures," get in touch with your nearest Bethlehem sales office. Or call: Brian Walker at (215)694-5906. Bethlehem Steel Corporation, Construction Marketing Division, Bethlehem, PA 18016-7699.

Owner: Pennsylvania Turnpike Commission, Harrisburg, PA
Fabricator: High Steel Structures, Inc., Lancaster, PA
Weathering Steel Supplier: Bethlehem Steel Corporation, Bethlehem, PA

Bethlehem

NSCC93

1993 NATIONAL STEEL CONSTRUCTION CONFERENCE
ORLANDO CONVENTION/CIVIC CENTER
ORLANDO, FL • MARCH 17-19

HOTEL REGISTRATION

NOTE: MAIL COMPLETE FORM DIRECTLY TO THE ORLANDO HOUSING CENTER.

HOUSING CENTER:

The Orlando Convention & Visitors Bureau Housing Department will coordinate NSCC 1993 hotel reservations. If reserving by telephone, have all the information below, and phone

1-800-258-ROOM (1-800-258-7666)

Office Hours:
Monday-Thursday
8:00 a.m. - 7:00 p.m. EST .
Friday, 8:00 a.m. - 5:00 p.m.

RESERVATIONS:

All rooms in Orlando must be guaranteed with a one night's deposit either by credit card or check. If a credit card number is not used, a deposit check in the amount indicated on your acknowledgement form must be sent directly to the hotel within 14 days of date processed. Send one reservation form per room. Names of occupants must be listed in the spot below. Reservations are made on a first-come, first-served basis.

CUT-OFF DATE:

The cutoff date for hotel reservations is February 15, 1993. After that date reservations will be honored on a space available basis.

CHANGES/ CANCELLATIONS:

All changes and cancellations should be made directly with the Orlando Housing Bureau. Your room confirmation will arrive directly from the Bureau.

NATIONAL STEEL CONSTRUCTION CONFERENCE
THE ONLY "ALL-STEEL" CONFERENCE AND TRADE SHOW IN THE U.S.
MARCH 17-19, 1993
ORANGE COUNTY CONVENTION/CIVIC CENTER
ORLANDO, FLORIDA

CONFERENCE HOTEL:

Clarion Plaza Hotel—$93 per night

The Peabody Orlando—$149 per night

Quality Inn Plaza—$57 per night

(The Clarion Plaza Hotel is the official Conference Hotel. Located 1/2 block from the Convention Center, it serves as the primary hotel for sleeping accommodations. All tours and optional events depart and return to the Clarion Plaza Hotel. The Peabody Orlando is across the street from the Convention Center and The Quality Inn is 2 blocks away. The Peabody and the Quality Inn have limited space. Suites are available upon request at the Clarion and Peabody.

NAME OF PERSON	ARRIVAL DATE	DEPARTURE DATE	TYPE OF ROOM SINGLE	DOUBLE	HOTEL CHOICE FIRST	SECOND	THIRD

CREDIT CARD INFORMATION

CREDIT CARD COMPANY

NUMBER

EXPIRATION DATE

CARD OWNER'S SIGNATURE

SEND CONFIRMATION TO:

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COMPANY

ADDRESS

CITY

STATE/ZIP

OFFICE PHONE

HOME PHONE

FAX

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of Steel Construction
c/o Orlando Housing Bureau
7208 Sand Lake Road, Suite 300
Orlando, FL 32819
Or Phone: 1-800-258-ROOM or
407/363-5800

HOUSING BUREAU DEADLINE DATE:

FEBRUARY 15, 1993

PRELIMINARY SCHEDULE OF EVENTS

MONDAY, MARCH 15

11:00-5:00 p.m. Exhibitor Move-In - continues until 1:00 p.m.

TUESDAY, MARCH 16

Noon-5:00 p.m. Wednesday
AISC Committee on Research Meeting/Luncheon

4:00-8:30 p.m. **Partners in Education/Committee on**

WEDNESDAY, MARCH 17

8:30-Noon **Education Meeting**
Design Responsibility Panel Discussion
A special panel made up of individuals with different industry viewpoints will discuss structural steel design responsibility. All NSCC attendees encouraged to attend.

8:30-10:00 a.m. **AISC Professional Member Forum**
Special session for structural engineers who are AISC Professional Members. AISC programs, plans, and publications will be reviewed.

8:30-1:00 p.m. **ASCE Committee on Steel Building Structures**

8:30-Noon a.m. **Steel Educator Program**
Session on subjects of interest to those teaching structural steel design courses at colleges and universities. Open to all Conference attendees.

9:00-1:00 p.m. **AISC Safety Task Force Committee Meeting/Luncheon**

12:00-3:00 p.m. **Magic of Ming**

1:00-1:15 p.m. **Welcome Remarks: Stephen E. Egger,** AISC Chairman, Egger Steel Company, Sioux Falls, SD
Introductions: Hollis L. (Pat) Hance, Jr., NSCC Co-Chairman
Southern Engineering Co., Charlotte, NC
Robert H. Woolf, NSCC Co-Chairman, Cives Steel Company, Roswell, GA

1:15-3:00 p.m. **General Session: Mill Practices into the 21st Century**
Presiding: Robert H. Woolf
Moderator: Robert Abramson, Interstate Iron Works Corp., Whitehouse, NJ
Panel: Companies represented on the panel include Bethlehem Steel, British Steel, Chaparral Steel, Northwestern Steel & Wire, Nucor-Yamato Steel, and TradeARBED
A panel of experts present the major structural shape producers' views of steel production techniques into the 21st century. There will be particular emphasis on new product development and the impact on the fabricators market for the future.

3:00-5:30 p.m. **Exhibits Open**

5:45-6:30 p.m. **Exhibitor Workshops (A-G)**
These special sessions offer a forum where companies share the latest technological advances in their fields, conduct demonstrations or question-and-answer dialogues, and introduce new or updated equipment and programs.

6:30-8:00 p.m. **AISC Welcome Reception**
All conference attendees and spouses are invited to this party in Exhibit Hall.

THURSDAY, MARCH 18

7:00-8:00 a.m. **All-Speaker Breakfast**

7:00-8:00 a.m. **Southern Association of Steel Fabricators Educator Breakfast**

7:00-8:00 a.m. **Virginia/Carolinas Structural Steel Fabricators Educator Breakfast**

7:30-8:15 a.m. **Exhibitor Workshops (H-N)**

8:30-9:15 a.m. **General Session: "Unique Exposed Steel Frame Creates Architectural Expression for Barcelona Tower"**
Presiding: Hollis L. (Pat) Hance, Jr., NSCC Co-Chairman
Speaker: Robert C. Sinn, Skidmore, Owings & Merrill, Chicago
The Hotel Vila Olimpica Project is a multi-use complex consisting of a luxury hotel/apartment facility, a commercial office building, retail parking, and other amenities. The hotel and apartment portion of the project includes a 43-story tower overlooking the Mediterranean Sea. The primary and most visually prominent structural elements of the hotel tower are the architecturally exposed X-braced structural steel frames located on the building periphery with the curtain wall set back. The exposed, unfireproofed exterior structure was designed using the latest state-of-the-art fire engineering methods developed in Europe and the U.S.

9:00 a.m.-2:00 p.m. **Universal Studios of the Stars**

9:15 - 10:00 a.m. **General Session: Design and Construction of the Cooper River Bridge**
Presiding: Hollis L. (Pat) Hance, Jr., NSCC Co-Chairman
Speaker: Raymond J. McCabe, Howard Needles Tammen & Bergendoff
June 20, 1992, marked the opening of the Cooper River Bridge, the longest bridge in South Carolina. Serving as a vital link in the I-526 Mark Clark Expressway, the construction challenges were met with innovative design elements. The main river span of this 5.1-mile bridge is a modern state-of-the-art parallel chord steel truss with spans of 400 feet, 800 feet and 400 feet.

9:30-3:30 p.m. **Park Avenue Shopping/Boat Ride**

10:00-3:00 p.m. **Exhibits Open**
Lunch will be served from Noon-1:30 p.m. in the Exhibit Hall.

10:45- 12:15 p.m. **Technical Sessions**
(See Technical Session section for description)
2. Managing Subcontract Detailing
3. Manual of Steel Construction: Volume II-Connections
4. Building and Motivating a Productive Workforce
5. Fire Restoration and Protection
6. Fabrication of Architecturally Exposed Structural Tubing
9. Steel Bridge Rehabilitation

12:30-2:00 p.m. **Poster Session (Exhibit Hall)**
An exhibition of technical papers will be displayed throughout the conference. Authors of papers will be available during this time period for discussion of the papers' contents.

1:30-3:00 p.m. **Technical Sessions**
1. Industrial Buildings
4R. Building and Motivating a Productive Workforce
10. Welding Symbols and What They Really Mean
11. Construction Automation in Steel Framing
12. Project Management: Organizing the Job
13. Composite Structures

3:10-4:40 p.m. **Technical Sessions**
7. Current Issues in Steel Building Design
14. The Fabrication Shop and the Environment
15. Seismic Design in Steel
16. Quality Standards vs. Fitness for Purpose

5:15-6:00 p.m. **Exhibitor Workshops**

7:00-11:30 p.m. **Conference Dinner at Church Street Station**

FRIDAY, MARCH 19

7:30-8:15 a.m. **Exhibitor Workshops**

8:30-9:15 a.m. **General Session:**
Moderator: Robert F. Lorenz, AISC Director of Education
Presiding: Robert H. Woolf
T.R. Higgins Lecture — winner to be announced

9:00-3:00 p.m. **Cypress Gardens Luncheon & Tour**

9:00 a.m. **Exhibits Open**
Lunch will be served from 11:30-1:00 p.m. in the Exhibit Hall

9:30-2:30 p.m. **Walt Disney World Village**

10:00- 11:30 a.m. **Technical Sessions**
6R. Fabrication of Architecturally Exposed Structural Tubing
7R. Current Issues in Steel Building Design
8. OSHA's Review of Steel Construction Accidents
9R. Steel Bridge Rehabilitation
11R. Construction Automation in Steel Framing
14R. The Fabrication Shop and the Environment

1:00 p.m. **Exhibits Close**

1:00-2:30 p.m. **Technical Sessions**
1R. Industrial Buildings
3R. Manual of Steel Construction: Volume II-Connections
8R. OSHA's Review of Steel Construction Accidents
12R. Project Management: Organizing the Job
15R. Seismic Design in Steel
16R. Quality Standards vs. Fitness for Purpose

2:40-4:10 p.m. **Technical Sessions**
2R. Managing Subcontract Detailing
5R. Fire Restoration and Protection
10R. Welding Symbols and What They Really Mean
13R. Composite Structures

5:00-9:00 p.m. **Sea World Party**

SATURDAY, MARCH 20

9:00-4:00 p.m. **Kennedy Center Tour**

1993 NATIONAL STEEL CONSTRUCTION CONFERENCE
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Exposed Steel Frame Creates Architectural Excitement

New developments in fire safety analysis allowed the developer of a 45-story Olympic hotel to utilize a steel exoskelaton

By Hal Iyengar, P.E., John Zils, P.E., and Robert Sinn, P.E.

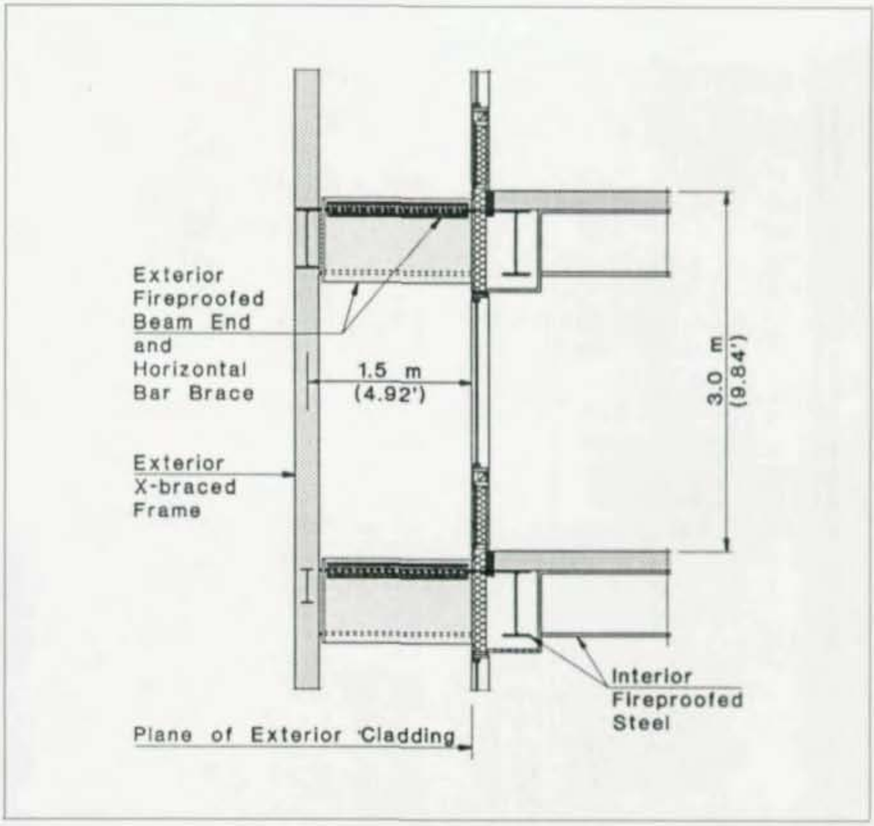
The Hotel de las Artes complex in Barcelona, Spain forms part of an overall seafront development of the Olympic Village area for the 1992 Summer Olympic Games. The complex consists of a 45-story hotel/apartment tower, a six story office block, and low-rise retail and public spaces.

The program for the hotel tower called for 460 rooms arranged primarily on two sides of a 118′ (36 meter) square tower over the lower 32 floors.

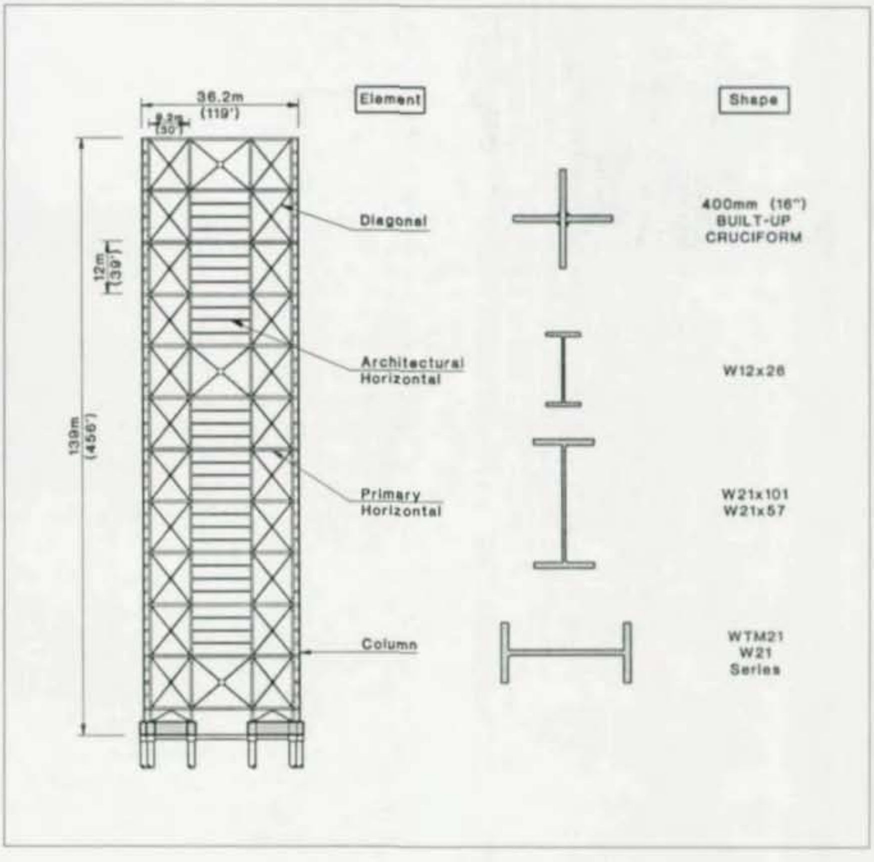
The upper 13 floors contain 32 deluxe apartments arranged generally on the basis of two or three units per floor. The elevator core is located in the center of the floor plate along with interior columns. This tower together with an adjacent office tower forms a twin-tower gateway into Barcelona and the Olympic Village area from the Mediterranean seafront. The design concept was based on an exposed exoskeleton steel frame which envelopes a smooth glass and metal rectilinear prism containing the hotel and apartment floors.

Exoskeleton Design

Architects and engineers have continually searched for structural systems that would make tall buildings economical and efficient. The critical factor in the evaluation of tall building structural systems has historically been the resistance to lateral wind and seismic forces. This process has led to a genre of structural systems which utilize the entire three-dimensional exterior

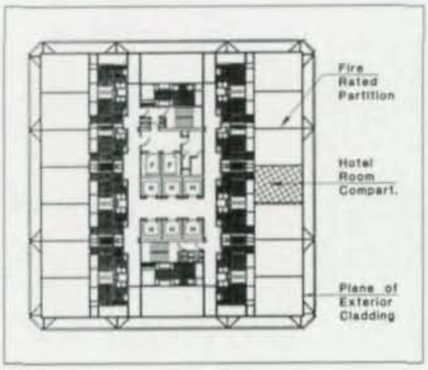


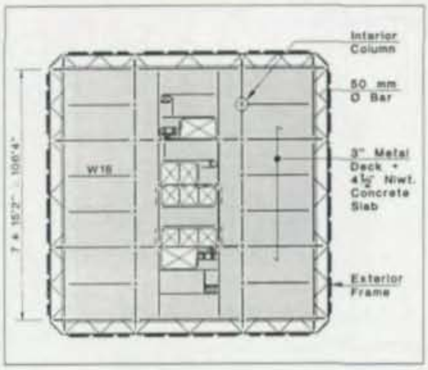
The top righthand drawing shows a section through the exterior wall revealing both the cladding and exposed steel on the Olympic The Hotel de las Artes complex in Barcelona, Spain. The bottom righthand rendering shows the exterior frame component sections. Shown on the opposite page is the completed hotel, while the photo below is a closer view of the exoskelaton form.

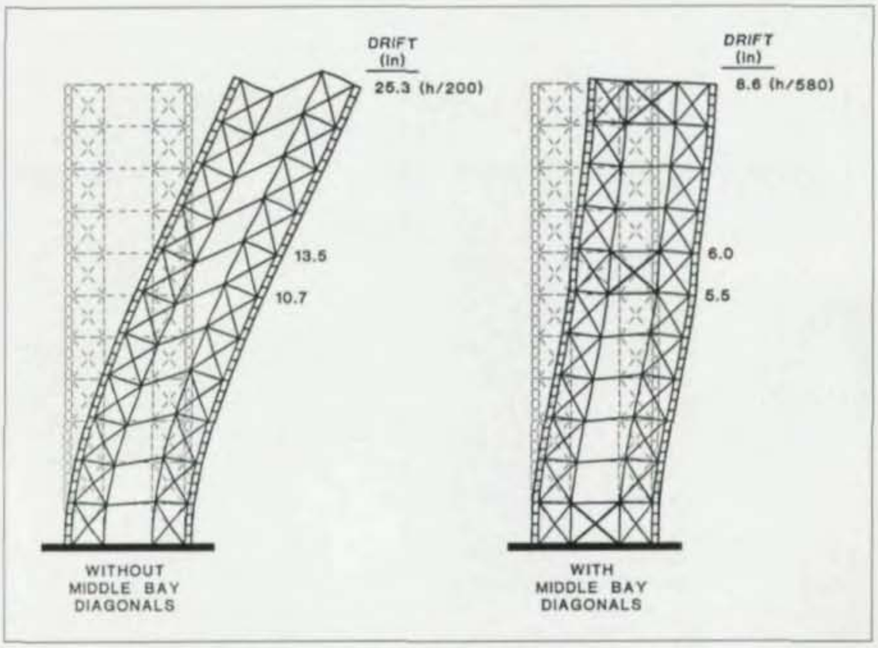


form of the building as a rigid box or "tube", thus providing maximum resistance to lateral loads.

Contrasted to a fully trussed tube such as the Hancock building in Chicago, it is also possible to create a trussed tube in the form of a megaframe. Essentially, this consists of vertical braced frame "legs" in each corner of the building interconnected at several locations by horizontal braced frame intersections. This megaframe form, while







At top are architectural and structural floor plans for the hotel, while pictured above is the structure's deflected shape under wind with and without middle bay diagonals.

not as efficient as a fully trussed tube, still brings on the essential tubular behavior and is suitable for medium height to supertall buildings depending on the number and spacing of the interconnections.

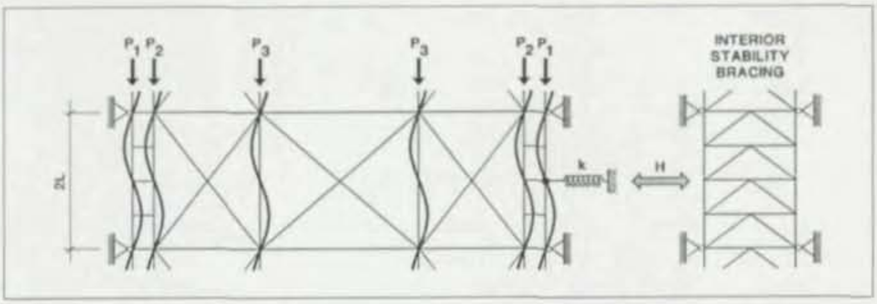
In order to create an exoskeleton form, the braced frame must be disassociated and pulled away from the plane of the window wall. This approach was taken for the Hotel de las Artes tower, which allowed the structure to be displayed against the backdrop of the window wall system. The exoskeleton form provides a basis for a purer structural expression; the clear articulation of the character of the structure in terms of member proportions, shapes, and joinery is made possible by fully exposing these members and their connecting joints.

One drawback to this type of exposed structure has historically been fire and corrosion protection requirements. In most cases, the members needed to be fireproofed with concrete or sprayed-on material and then enclosed in masonry or metal and glass facades. While other fire protection systems, such as flame shielding and liquid cooled members, have been attempted, they tend to camouflage and confuse the frame expression.

However, the development of an analytical fire engineering approach to determine the steel temperatures when exposed to different fire conditions, as well as the determination of the character and nature of the fire, has been the real breakthrough in the design of exposed structural steel systems. The pioneering work by Margaret Law and others has set the stage for such designs and the technical feasibility of exposing structural steel has brought on a new vocabulary in architecturally expressing the structural frame. It is now possible to emulate the natural beauty of exposed steel with all its crisp member proportions as exemplified in the Eiffel Tower and the great Victorian train stations in London and Europe.

The Hotel tower exterior frame is placed approximately five feet away from the window wall surface thus creating the exoskeleton form. The floor framing projects through the window wall and connects to the exterior frame. Bar type horizontal diagonal braces in the plane of each floor complete the engagement of the exterior frame with the floor slabs to establish the required diaphragm action. Each elevation consists of X-braced frames over a 30′ (9.2 meter) width on each corner that are interconnected horizontally at the 2nd, 33rd, and 41st floors by another X-braced frame in the center bay. The frames on each elevation are connected by continuous beams along chamfered corners. These frame interconnections serve to induce three-dimensional behavior under wind forces, thereby establishing the efficiency of an equivalent tubular system.

Two sets of columns were placed on the interior; one set at the edge of the hotel rooms and the other in the core area. Floor beams were placed 15′ (4.6 meters) on center corresponding to the dividing partitions between hotel rooms. The floor beams were designed on a composite basis with a 3" (75 mm) composite metal deck and 2½" (60 mm) of stone concrete topping. A second 2" (50 mm)-thick concrete slab was added to the composite slab system to provide a two-hour fire separation between floors. Bar type diagonals were provided where required to brace the columns and vertical diagonals of the exterior frame as well as to resist diaphragm shears. The floor framing at the apartment levels was similar to that of the hotel floors.



The diagram above shows the stability system four-story concept of design. The photo at right shows the bars and clevises at the gallery level.

The structural steel for the frame was obtained from various European sources. Locally available European standard (IPE, HEA, HEB) steel shapes were used for the lighter floor framing and some interior columns, while the exterior frame rolled shapes were obtained through AISC-member Trade-ARBED. All structural steel in the tower, both interior and exterior, was ASTM A572-Grade 50. Sherardized coated (a British process similar to galvanizing) high-strength bolts for exterior connections were obtained from Great Britain while conventional bolts for interior application were obtained locally.

The project's owner is The Travelstead Group, architects/engineers was Skidmore, Owings, and Merrill, and general contractor was Bovis International.

Frame Behavior And Analysis

Apart from gravity loads, the braced megaframe was required to resist wind and seismic lateral forces. Barcelona is in an area of relatively low seismic activity and as such the seismic lateral forces did not control the design of the tower. Based on applicable Spanish loading codes and wind tunnel testing, two levels of design wind loads were established: one at a 50-year recurrence period which was used for stiffness/sway control; and the other at a 100-year recurrence period used for strength and stability design.

The braced frame system was conceived on the basis of maximizing efficiency under wind load acting as a vertical cantilever, while minimizing the dimension of the bracing required on each facade. The four story high, 30 foot wide bracing module was appropriate for the scale of the 460 foot tall tower. The framed beam connections at the chamfered corners emphasized discreetness of the bracing on each facade while providing structural continuity around the corner. The net result of this system was to provide L-shaped frames at each corner of the building. Preliminary frame analysis under wind load with only these corner frames revealed large lateral sway deformations on the order of 25" (64 cm) and also produced significant uplift forces in the columns and foundations. The addition of braced bays in the middle module of each facade only at three vertical locations, namely the 2nd, 33rd, and 41st levels, significantly improved the stiffness resulting in a sway deformation of 8.6" (22 cm).

The threefold increase in stiffness produced with only 0.25 psf (1.2 kg/square meter) of steel expended for the middle bay bracing diagonals testifies to the efficiency of the system. The tension forces that existed at the middle facade columns in the unlinked condition were eliminated when the links were introduced. In behavioral terms, the isolated L-shaped corner braced pieces were linked together by the middle bay bracing to achieve an equivalent cantilever utilizing the entire three-dimensional form of the tower as a mega-portal frame. The cantilever efficiency, as a proportion of the efficiency developed on the assumption of a fully effective column stack with neutral axis through the center of the building cross-section, was 78 percent, which compares well with the efficiencies of systems such as the John Hancock Center and Sears Tower. The resulting steel quantity for the tower was 17.8 psf (86.9 kg/square meter) applied over the 550,000 sq. ft. (51,100 square meters) gross framed floor area.

The dynamic properties of the tower corresponded to a 5.2 second period for the fundamental sway mode and 2.15 seconds for the torsional mode which resulted in accelerations at the top of the building of 19 mg for a 10 year wind as determined by the wind tunnel study.

The general attachment laterally of the diaphragm and the frame was by means of diagonal bars at the exterior columns at each floor and at the vertices of the vertical X-braces. The 2" (50 mm) diameter horizontal diagonal bar braces were chosen to represent the least visual encumbrance with respect to the clarity of the frame expression. The design procedure with respect to column and frame instability can be summarized as follows:

- All columns taken as isolated individual elements were designed with an effective length of one story as all columns are braced in two directions at the floor lines.
- Eigenvalue studies were undertaken to evaluate the possibility of four story buckling of all columns taken together (one vertical

bracing module).

- Since the slender horizontal bar brace at the diagonal vertex was insufficient to prevent the four story buckling mode, it was decided to add secondary vertical braced frames in the core area to supply the required stiffness and strength to induce a two story group buckling mode.

Exposed steel frames exhibit thermal movements due to seasonal temperature changes which must be addressed both in terms of induced stresses and in terms of relative vertical displacement of the exterior structure with respect to the interior frame. The moderate climate in Barcelona produced a seasonal variation of only 25 degrees C. This allowed the use of simple, conventional partition details which accommodated the relative vertical movement between the interior and exterior structures. Any additional ambient thermal stresses caused by restraints in the system were absorbed in the exposed exterior structure.

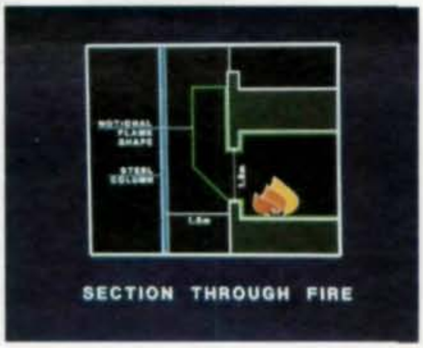
Fire Engineering

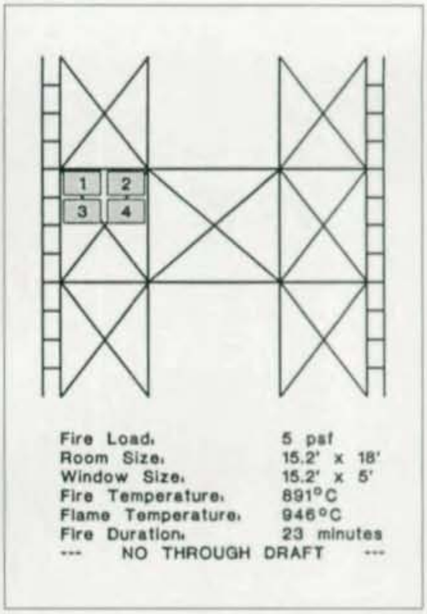
The approach taken for the fire-engineering of the exposed, exterior structural steel frame was based on the latest state-of-the-art methods developed in the United States and Europe. The problem of fire rating of an internal element exposed to fire loading is well documented and is based on standardized tests conducted in a fire chamber according to ASTM E119 or ISO standards.

The case of an external element exposed to fire is much different. External elements will be exposed to the effects of: radiant heat from the fire through the window opening, radiation and convection from flames projecting outward toward the steel, and radiant heat loss to the surrounding atmosphere. The design procedure involves four distinct steps. First, the fire load must be determined based on either a survey of the amount of combustible material available or on a code prescribed fire load magnitude. A fire load of 5 psf (25 kg/square meter) in the hotel rooms and 10 psf (50 kg/square meter) in the apartments was used based on European codes. Second, based on the fire load, the compartment and window opening sizes, and the amount of ventilation present, the character of the fire and the flame profile as well as the duration of the fire are determined based on empirical fire engineering equations. Third, the temperatures of the exposed steel elements opposite the window opening are determined by classical heat-transfer theory and compared against accepted maximum values. Finally, high-temperature structural analysis of the exterior frame is performed to demonstrate structural stability under fire loading conditions.

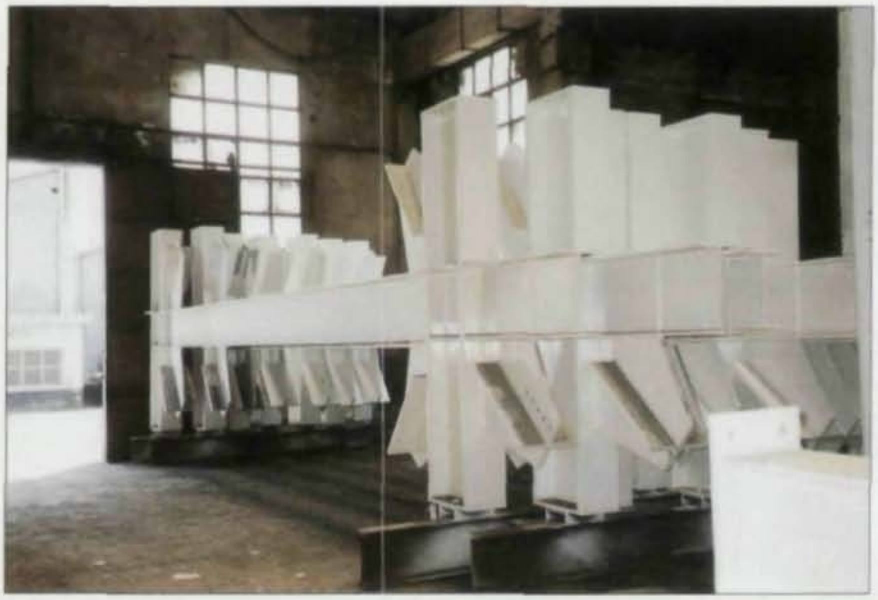
Since the hotel floors are compartmentalized by fire rated partitions between the rooms, the extent of the fire was confined to a single room at a time. The compartmentalization also prevented through-draft ventilation from occurring with the result that the flame profile hugged the window wall. The exposed steel frame was placed 4.92' (1.5 meters) away from the cladding such that the flame does not engulf the exposed steel members and therefore limits their heat gain and resulting steel temperatures.

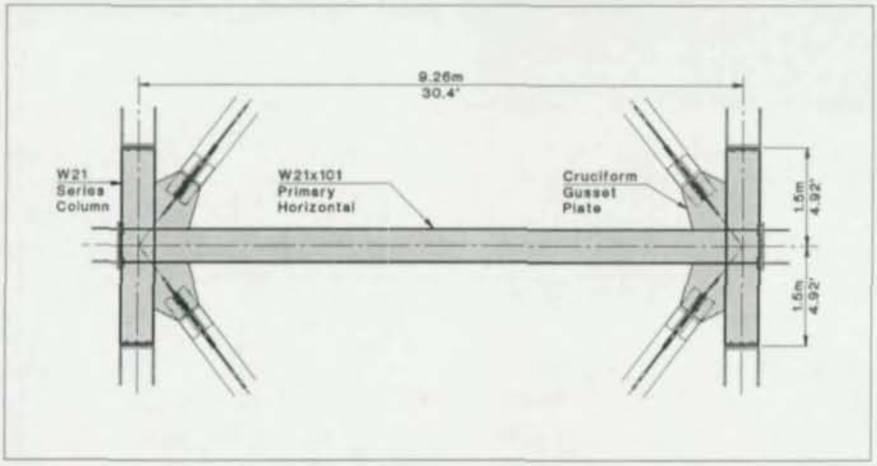
The flame does, however, engulf all the connecting steel between the window wall and the exterior frame and as such these elements consisting of diagonal bar bracing and the floor beam projections were fire protected. The apartment floors had larger compartments and had the possibility of through-draft ventilation. The flame profile under these conditions was broader and it projected out engulfing the steel frame. Analysis indicated that temperatures in excess of the established limits would re-





These three pictures, clockwise, show the fire engineering notional flame profile without throughdraft, fire design diagram, and the actual fire test.





Pictured above left are the erection units loaded onto a truck at the fabrication shop, while below left shows a fabrication detail. Pictured above is an eight-story section of a corner bay.

sult. In lieu of fire protecting the frame in this region, a two-hour fire rated glazed window wall system was used which prevented the flame from extending through the window wall until all the fire load was exhausted.

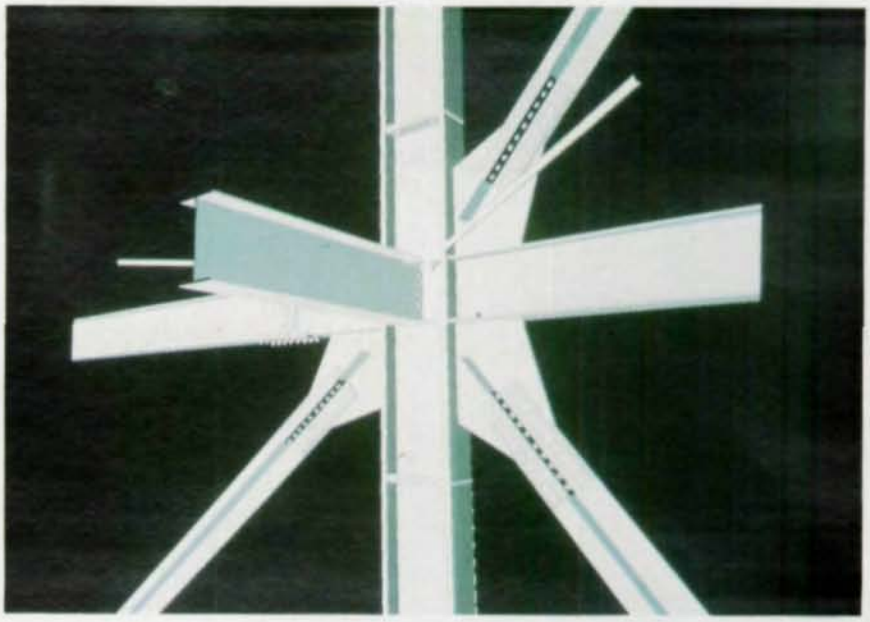
A number of hypothetical design fire events representing different locations of hotel compartments were considered. The characteristics of the fire and compartment are indicated as well. Two types of design fire event were considered in designing that portion of the exterior frame opposite the apartment floors where the window wall assembly is fire-rated. First, the possibility of a localized breakage of the rated glass wall was considered to account for the possibility of accidental damage during the fire and second, the effect of radiation of the fire through the intact curtain wall. Fire engineering analysis indicated that the maximum steel temperatures were 169 degrees C, 386 degrees C, and 383 degrees C for the columns, beams, and diagonals respectively which was well below the accepted maximum temperature of 550 degrees C (1000 degrees F). At this temperature, steel retains 75% of its yield strength and modulus of elasticity and beyond which these properties begin to rapidly deteriorate.

The final step in the fire engineering procedure for the exterior frame was a high temperature structural analysis for the various design fire events. Each design fire required a unique set of material properties based on a reduced modulus of elasticity corresponding to the calculated steel temperatures. Thermal loads were applied to the various members based on the maximum steel temperature opposite the top of the window opening reduced somewhat to account for the temperature gradient along the length of the member away from the heat source. Stability of the exterior frame under fire loading was checked using the standard AISC ASD beam-column interaction equations modified to account for reduced modulus of elasticity and yield strength at elevated temperatures. A minimum 25% additional reserve capacity was included in the member designs for the fire load cases.

It should be noted that most model building codes in the United States and Europe, including the Uniform Building Code, BOCA, and the Southern Building Code, allow fire engineering calculations to determine the fire resistance as an alternate to conventional hourly fire rating.

Since fire engineering is a rational method that uses first principles, it cannot be compared with prescriptive methods generally stated as the number of hours of rating for an assumed standard time-temperature curve. However, as an approximate verification, a fire test was performed at the request of the Barcelona fire department officials to verify that the fire-engineered design of the structure would produce temperatures equivalent to the code requirement of a two-hour fire rating.

The test chamber approximated the conditions of a typical hotel room. Steel sections were placed in front of the test chamber representing the lightest and therefore the most critical exposed steel sections used for the actual frame. More than 100 thermocouples were placed in various positions on the steel pieces. Gas jets located within the compartment were calibrated so as to produce an equivalent of the standard ISO time-temperature curve and were allowed to remain active for three hours. The maximum steel temperatures recorded during the test were well below the 550 degrees C critical temperature and indeed were far below the temperatures calculated using fire engineering principles. The test showed that the nature of a fire with flames emerging through an opening in the exterior wall as in a real fire is generally more critical than any application of the standard ISO fire within the hotel room for this situation.



Exterior Framing

The general nature of structural steel is one of wide flange shapes expressed by their thin outstanding flanges. The frame character is best exhibited by continuity of the flanges at the beam-column joint. The general placement of the wide flange column web parallel to the building plane provided a head on view of the flanges and the beam-column intersections. The diagonal connection with a single gusset plate in the plane of the member webs allowed for continuity of the frame without disruption by the gusset plate.

This basic member orientation was used throughout the structure. The exterior frame erection was facilitated by designing a shop prefabricated unit involving two columns and interconnecting beam, with all welding of the interconnecting joints performed in the shop and later field bolted.

Similarly, corner moment connections were end plate type, field bolted. Column splices were also field bolted butt-plate type connections.

The exterior frame members and connections were chosen based on erectability, accessibility for painting and future maintenance, limiting the potential for corrosion, and aesthetic visual considerations. The sections chosen for the various frame members represent a hierarchical progression based on their relative importance with respect to lateral and gravity load resistance.

The dominant role of the column element in directly resisting gravity loads and overturning moments due to wind is expressed by choosing deep W21-series rolled shapes rather than conventional W12 or W14-series columns. By specifying tailor-made WTM21-series members from TradeARBED for the lower story columns, all exterior columns were rolled shapes of constant depth between the flanges. The second level of frame expression included the 16" (400 mm) deep built-up cruciform diagonal member and W21-series primary horizontals on the four-story vertical module. A cruciform section was chosen for the vertical diagonal as the open shape best suited for painting accessibility, simplicity in connection detailing, and consistency with the flange expression of the other components.

The connection detailing philosophy was formulated based on eliminating site welding of the exposed painted steelwork and directly expressing the bolted character of the connecting joints. All exterior frame details were completely specified on the working drawings including weld sizes, gusset plate dimensions, bolt spacings, and plate thicknesses. All details were computer generated with typical details developed and reviewed with the architectural team via three-dimensional computer renderings.

Rules for the symmetric and organized placement of connection plates, bolt heads, and splices were specified as well. Gusset plates were shaped to more clearly define the underlying flow of forces through the structure. All shop welding of exposed steelwork was continuous and completely wrapped around connection corners and intersections to avoid corrosion.

Painting Systems

The exposed, unfireproofed exterior steel structure received a coating system designed to provide "long life" as defined by British Standard BS 5493 for an "exterior exposed polluted coastal atmosphere" with finish coats. Due to the various conditions of exposure and accessibility, a number of different coating systems were required to achieve the desired level of corrosion protection while at the same time maintaining the aesthetic visual appearance.

The painting system included four types depending on the location and access requirements. The basic sequence of painting systems application involved shop cleaning, priming, and painting of the steel for shipment to the site and application of a second finish coat in the field after erection and final cleaning.

Two types of exterior exposed steel fire protection were utilized for the structural steel members that extended between the exterior window wall and the exoskeleton frame. An epoxy resin, preformed intumescent material was used to provide a two hour protection for all horizontal diagonal struts, clevises, and associated gusset plates. The mesh reinforced intumescent fireproofing thickness required to provide the two hour rating was 9⁄16 inch (14 mm) which was used throughout. The material was preformed to the appropriate shapes and then applied directly to the steel members with adhesive. A flat fireboard material was used for all exposed wide-flange floor framing beam ends. Both exterior fireproofing materials were suitable for exterior exposure.

The Hotel de las Artes tower represents a significant step forward in establishing the validity of fire engineering methods in exposing steel frames in tall buildings. The use of a mega-portal frame system illustrates the adoption of a powerful modular system for buildings in the mid-height range. By marking a major international event in the summer of 1992 and by serving as a catalyst for the reawakening of Barcelona as a major European capital; the Hotel de las Artes tower represents a significant step in the development of tall building structural systems and architectural design.

Hal Iyengar, P.E., is a partner and director of structural engineering, John Zils, P.E., R.A., is an associate partner and senior structural engineer, and Bob Sinn, P.E., is an associate and project structural engineer with Skidmore, Owings, and Merrill. This article was adapted from a paper delivered at the 1993 NSCC.

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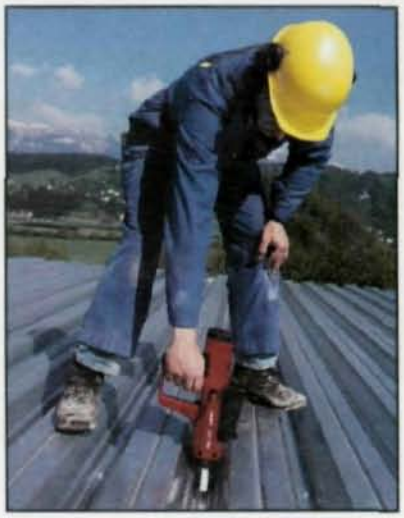

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Designing Architecturally Exposed Steel Tubes

An in-depth examination of bending and connection *requirements* for engineers and fabricators

By Lawrence A. Kloiber, P.E.

Hollow structural sections have structural and aesthetic properties that make them uniquely suited for use on exposed steel structures. The axial, lateral and torsional strength of tubes allow them to be used in long unbraced lengths, carry loads in two planes, and carry eccentrically placed loads. And their clean appearance have made them an increasing favorite with architects.

However, structural engineers and fabricators must pay special attention to two aspects of design when working with tubes: radius bending and connections.

Tube Bending

Tubing can be readily formed to small radius curves because with the closed shape there is excellent torsional resistance and no unsupported edges, which have a tendency to buckle. If proper wall width thickness ratios are maintained, there will be no local buckling the walls when forming.

There are basically three methods of bending tubes—heat cambering, mechanical bending, and *magnetic induction bending.*

Heat cambering is limited to relatively large radius curves such as those used for beams and trusses. It can be done in the fabrication shop using standard heating torches.

Mechanical bending is capable of forming smaller radius curves, such as would be required for arches and domes. The bending is done by applying enough force to permanently deform the tube. Because of the arrangement of the rolls or presses, up to 4' of extra length must be added on each end of the curve to serve as a lead and tail on the piece.

Obviously, during bending the tube walls must be stressed well into the yield region. For a given tube size, the thicker the wall, the less likely it is to buckle at these large strains. Walls of ¼" or less in thickness should be avoided except in tubes 3x3 and smaller. Table 1 shows some typical tubing sizes and the approximate minimum radius curve that can be mechanically formed.

Note that it is possible to bend smaller radius curves on some sizes of rectangular tubing bent in the weak axis direction by using some type of internal stiffener to help prevent buckling.

Magnetic induction bending can produce even smaller radius curves by combining heating with mechanical bending. The tube is first passed through magnetic induction coils that heat the tube enough to substantially lower its yield strength. While hot, the tube *is mechanically bent similar to the* cold bending process. The heated tube not only requires less force to bend, but the material in this condition also will accept greater strains without buckling. As a rule of thumb, tubing can be bent with this process to a radius of approximately five times its section depth.

Because of the equipment required and the cost of the process, magnetic induction bending is the most expensive way of forming.

Fig. 1: Recommended Minimum Circular Curves For Mechanical Bending

Rectangular Tubing Weak Direction	Minimum Radius	Special Radius (Using Internal Stiffening)
TS 4 x 2 x 5/16	3'-0" IR	
TS 6 x 2 x 3/8	4'-0" IR	3'-0" IR
TS 6 x 4 x 3/8	5'-0" IR	4'-0" IR
TS 6 x 4 x 1/2	4'-0" IR	3'-0" IR
TS 8 x 2 x 3/8	6'-0" IR	3'-3" IR
TS 8 x 4 x 3/8	8'-0" IR	5'-0" IR
TS 10 x 2 x 3/8	7'-0" IR	3'-6" IR
TS 10 x 4 x 1/2	10'-0" IR	4'-6" IR
TS 10 x 6 x 1/2	18'-0" IR	9'-0" IR
TS 12 x 2 x 3/8	8'-0" IR	4'-6" IR
TS 12 x 4 x 1/2	12'-0" IR	5'-0" IR
TS 12 x 6 x 1/2	23'-0" IR	9'-6" IR

Rectangular Tubing Strong Direction	Minimum Radius
TS 4 x 2 x 5/16	2'-6" IR
TS 6 x 2 x 5/16	4'-6" IR
TS 6 x 4 x 3/8	4'-6" IR
TS 8 x 4 x 3/8	8'-0" IR
TS 8 x 6 x 1/2	6'-0" IR
TS 10 x 4 x 3/8	9'-6" IR
TS 12 x 4 x 3/8	12'-6" IR

Square Tubing	Minimum Radius
TS 2-1/2 x 2-1/2 x 1/4	1'-6" IR
TS 4 x 4 x 5/16	4'-0" IR
TS 4 x 4 x 1/2	2'-6" IR
TS 6 x 6 x 3/8	6'-6" IR
TS 6 x 6 x 1/2	4'-6" IR
TS 8 x 8 x 3/8	13'-0" IR
TS 8 x 8 x 1/2	8'-0" IR
TS 10 x 10 x 5/8	4'-6" IR

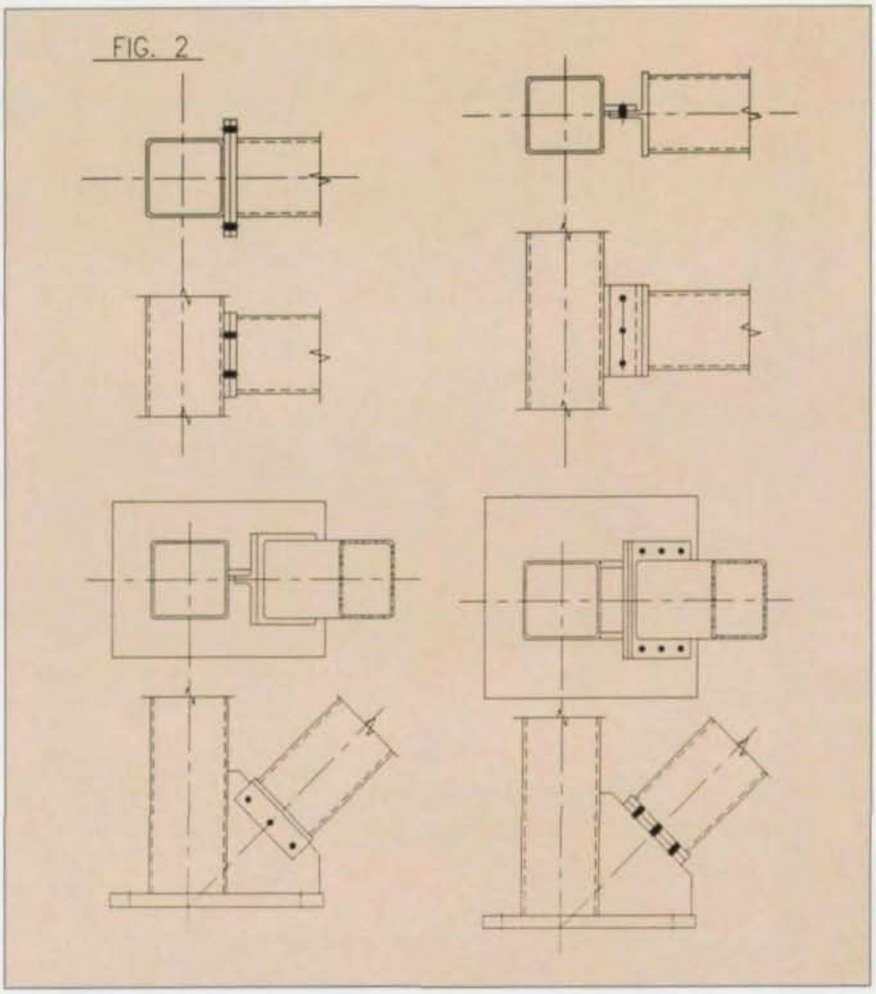
Pipe	Minimum Radius
4" Standard	2'-6" CLR
6" Standard	4'-0" CLR
8" Standard	6'-6" CLR
10" Standard	12'-0" CLR
12" Standard	24'-0" CLR

However, it is sometimes the only way to achieve the desired curve. Also, both mechanical and magnetic induction bending are usually done by specialty subcontractors, so freight can also be a significant cost.

Connections

Hollow structural sections present special connection problems. The standard bolted connections that work so well on wide flange shapes are not readily adaptable to tubes. Bolted tube to wide flange connections can be readily made by welding the connection material to the tube and bolting it to the wide flange. End plates, single plate shear connections, WT connections, and even some types of double angles can be used.

Bolted tube-to-tube connections are much more cumbersome. Through bolting is not very practical because the long grip lengths required are expensive and the bolts cannot be tensioned. Except for light connections, it can be difficult to stick the bolts in the field because of the necessity to line up two separate faces. Most bolted tube-to-tube connections require connection material to be welded to each tube and then the connec-



tion material is bolted together (see Figure 2).

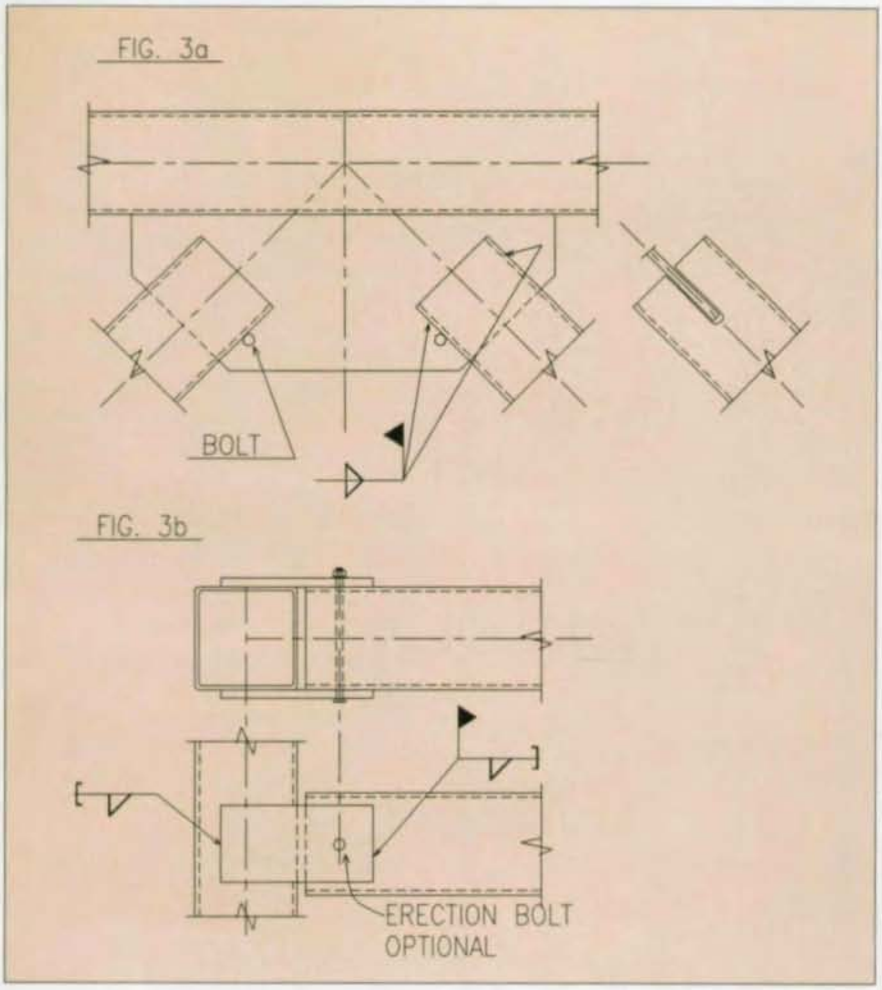
Simple all welded tube-to-tube connections can easily be made using various types of gusset plates. A slotted tube can be slipped over a gusset plate that is welded to the face of another tube. After aligning, the slotted members can be easily fillet welded to the gusset plate.

The slot allows enough adjustment for easy fitup. A simple erection aid can be provided, as shown in Figure 3, by placing a bolt in the gusset for the tube to rest on until welded.

Two gusset plates that lap the sides of the supported tube also work well. Connections similar to Figure 3 provide good shear capacity as well as torsional strength. If any of these gusset plate connections are used in exterior exposures, it is important to provide some method of sealing the tubes.

Direct welded tube-to-tube connections often are preferred for architecturally exposed tube structures. Unfortunately, design drawings often simply show these members with their centroids intersecting at a common point and then call for complete joint penetration (CJP) welds. CJP welds for tubing where the back side of the weld is not accessible must either use internal weld backers or comply with the special requirements of Chapter 10 of the AWS Structural Welding Code D1.1. Backers can easily be provided in butt and "T" joints, but attaching backers is much more difficult with the "Y" and "K" joints.

When sizing fillet and PJP welds for "T", "Y" and "K" joints for tubing, the designer should be familiar with the requirements of Section 10.5.1.3 of AWS D1.1. It is not adequate to simply size welds for the load in the member. Due to the difference in stiffness across the wall of the member normal to the load,



the load transfer is highly non-uniform. Welds must be sized to prevent "unzipping" or progressive failure because of so-called "hot spots."

When designing structures using direct tube-to-tube welds, the layout and sizing of the members should be planned to simplify fitting and welding.

Overlapping joints should be avoided and to a lesser extent matched member sizes should be avoided (see Figure 4). Overlapping joints require double miter cutting and are generally more difficult to fit and weld. The member that is lapped must be welded before the overlapping member is installed. On most trusses and frames, this will either require out-of-position welding or extra handling. Overlapping joints should be used only where there are heavy loads in the branch members and it is desirable to directly transfer some of this load between branches.

When planning trusses, overlapped joints can be avoided by: keeping the intersection angle shallow; using as deep a chord section as possible; or by permitting some eccentricity in the joint.

Stepped joints are preferred over matched joints because they can be easily fit and fillet welded. Stepped joints should be sized so the toe of the required fillet weld is as close as possible to the tangent point of the corner radius to provide the best load transfer.

Architects like the appearance of the flush sides in a matched joint. In a matched "T" joint, if the branch is cut square, the side wall of the branch and the radius of the main member.

When fabricating large trusses or frames with field welded tube-to-tube connections, the field connection should be at the member intersection. Sometimes the designer or the erector wants to sim-

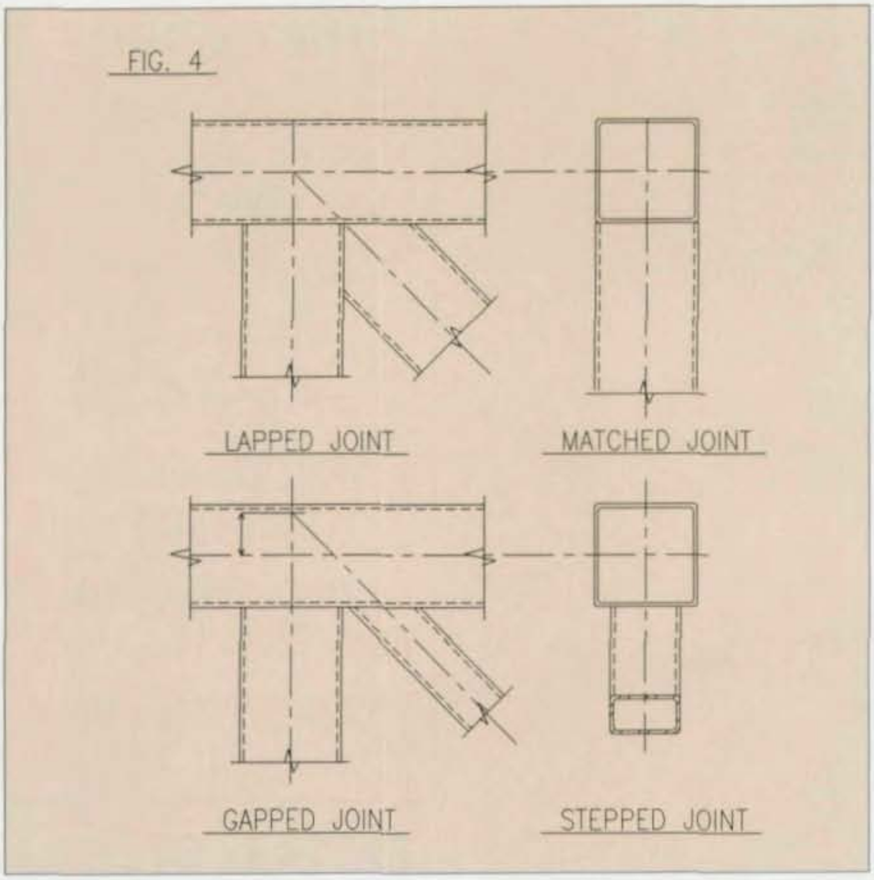
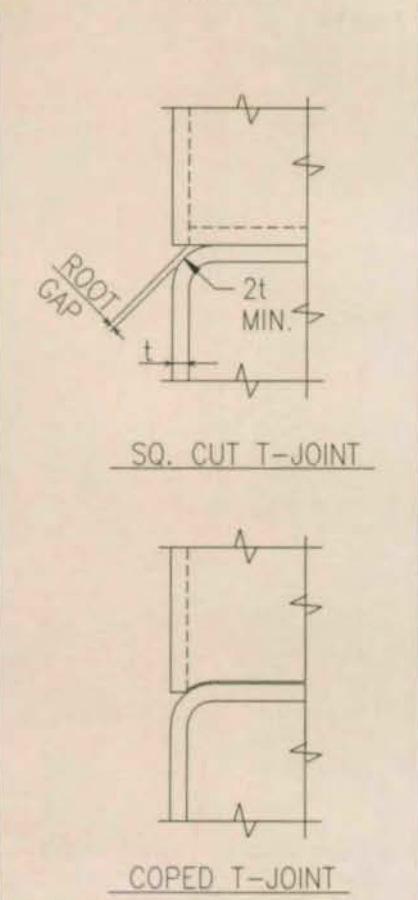


FIG. 4

plify the field connections by shop welding stubs to the main member and field splicing the branch members. Not only is this more expensive because of the extra splices, but the field splices are visually intrusive. Even if the fabricator can align and fit the stubs perfectly, there will be some movement due to weld shrinkage, which will result in an offset field splice or a dog leg in the branch.

Erection costs of field welded tube-to-tube connections can be reduced by providing simple erection aids. This is one place where engineers must use their ingenuity. The detail shown in Figure 6 is one we have used on several projects with sky lights framed with tubing. A similar connection could be used for beams framing to columns by using a clip angle. The hole in the clip should be large enough to accommodate the weld flash at the base of the threaded stud and still provide erection clearance. Figure 7 shows a detail that can be used where the entire tube is exposed.

Fig. 5



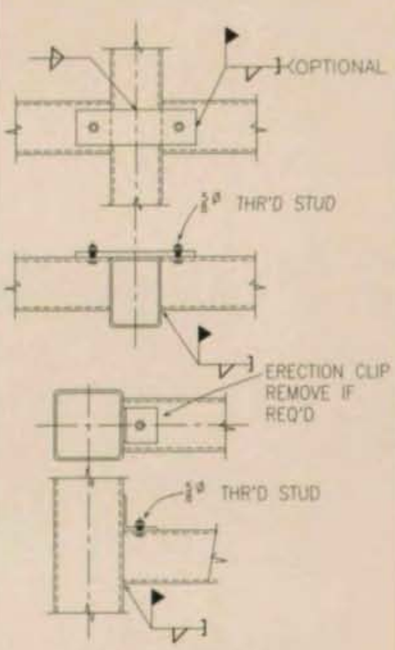
This is similar to the Saxe clips that were used in early welded structures.

Welding And Grinding

The previously mentioned aids are designed to be left in place. Other aids can be designed with instructions for the erector to remove and grind the area after the tube is welded.

The AISC Code of Standard Practice offers good basic guidance on welding architecturally exposed structural steel: ""Reasonable smooth uniform as-welded surfaces are acceptable on all welds exposed to view. Butt and plug welds do not project more than 1/16" above the exposed surface. No finishing nor grinding is required except where clearance or fit of other components may necessitate or when specifically required by contract documents."

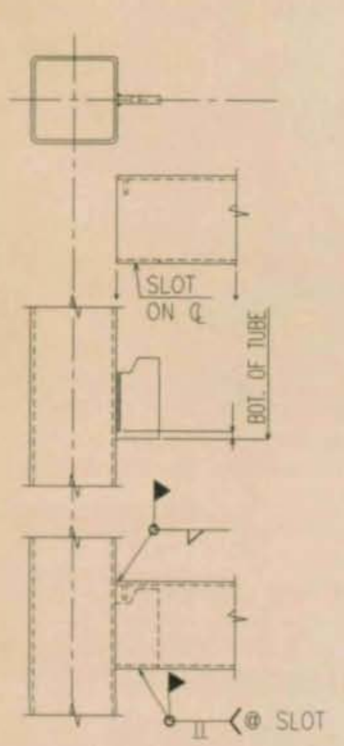
Figures 6a & 6b



Unfortunately, many architects think they can get an improved appearance by writing specifications calling for grinding all welds. This is not only costly, but it may also result in poor appearance and improper weld size or profile. Certain welds, such as butt welds, can be easily ground if appearance is especially critical. However, even when grinding butt welds it is impossible to get a perfectly flat, smooth surface. There is usually always some misalignment between pieces, and weld metal is always higher in strength than the base metal. When grinding, there is a tendency to either over or under grind parts of the joint. The best you can expect is a smooth transition.

Fillet welds are very difficult to grind (as shown in Figure 8). The convex weld bead is positioned at 45 degrees to the intersecting joint faces, making it impossible to place the face of the grinder on the weld. The grinding has to be done with the edge of the grinding wheel or disk. Any attempt to continuously grind a fillet weld in this manner

Fig. 7



usually results in gouges and varying weld profiles. It is better to use a welding process and procedure that will result in the workmanlike reasonably smooth uniform weld described in the AISC Code of Standard Practice.

Good workmanship as described in AWS D1.1 requires that all deviations from the required weld profile, such as craters or excessive undercut, be repaired. Grinding should be limited to faring in these repairs and any other profile variations, such as overlaps.

Multi-pass fillet welds such as shown in Figure 9 present a special problem. Even with good workmanship, the individual passes will be readily visible. On larger size fillet welds, it is possible to make some improvement with grinding, but it is impractical to try to eliminate the lines between passes. If the architect wants a smooth appearance, then epoxy resin fillers should be used to dress the welds. These fillers can be sanded to provide a smooth transition. Also, it

FIG. 8

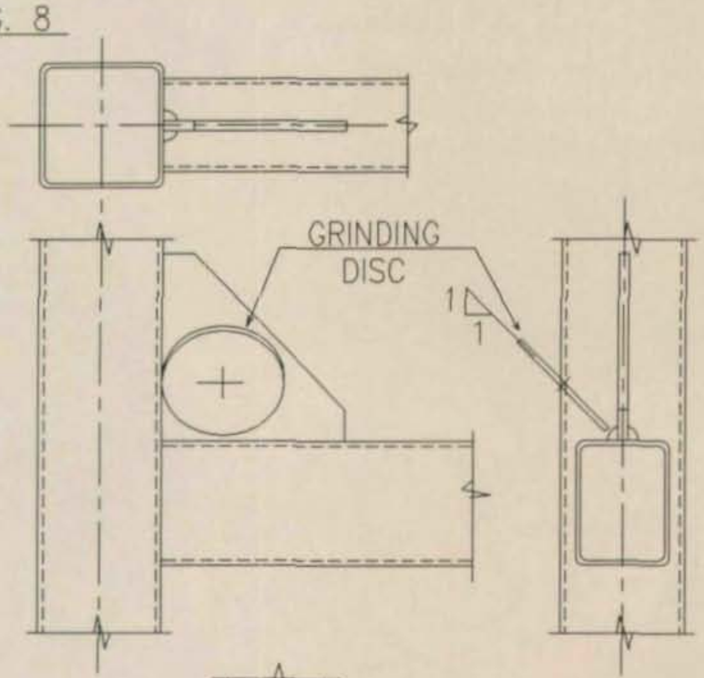


FIG. 9

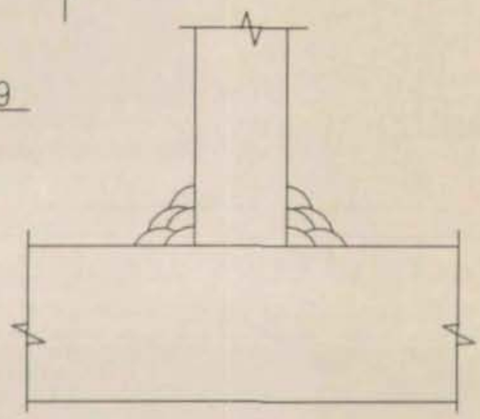
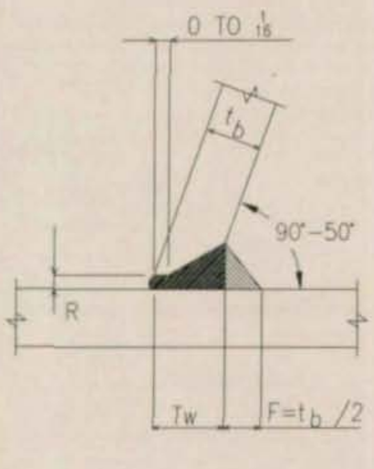


Fig. 10



may be possible to avoid the appearance at a multi-pass fillet by combining a PJP weld with a single pass fillet to get added strength.

Groove welds in ""T" joints can be ground, but ""V" and ""K" joints have limited accessibility. When grinding these types of welds, care should be taken to make sure the proper throat is maintained. Reinforcing fillets are not required for standard pre-qualified ""T" type groove welds in statically loaded structures, but it is good practice to provide smooth transition with a small fillet weld. The special AWS pre-qualified CJP details for tubular ""T", ""Y" and ""K" joist shown in Figure 10 must have specific weld geometry that includes the minimum reinforcement requirements shown.

Visual Acceptance Criteria

The decision on what is an acceptable final product is highly subjective. It is complicated by the difference between inspecting a piece close at hand in the fabricating shop and viewing it in place. The desired quality level also can be affected by how readily the connections can be viewed.

We recently went to considerable effort for the connection details for architecturally exposed tubing only to find catwalks and ductwork everywhere, effectively hiding many of the connections.

An experienced engineer once told me that architecturally exposed steel should be inspected from the same distance as it can be viewed after erection.

It's also important that expectations be realistic. We had a recent project where we had to cut and taper tubes for columns and beams in a large entrance canopy and monumental stairway. The architect expected the weld seem along the taper to be completely invisible. And even after we used epoxy resin fillers and special sanding, the architect was not satisfied. We later found out that the architect originally wanted to enclose those members with aluminum covers, but the construction manager eliminated them as too expensive. Yet the architect wanted the same finish with tubing, and that just isn't possible.

One partial solution is to use mockups to show what the finished product will look like. These mockups allow all parties to agree on a standard prior to construction.

Lawrence A. Kloiber, P.E., is president of LeJeune Steel Co. Several consultants, suppliers and specialty subcontractors provided information for this article, including the Max Weiss Co., Naptech, Inc., and the Welded Tube Co. of America. This article was adapted from a paper presented a t the 1993 National Steel Construction Conference.

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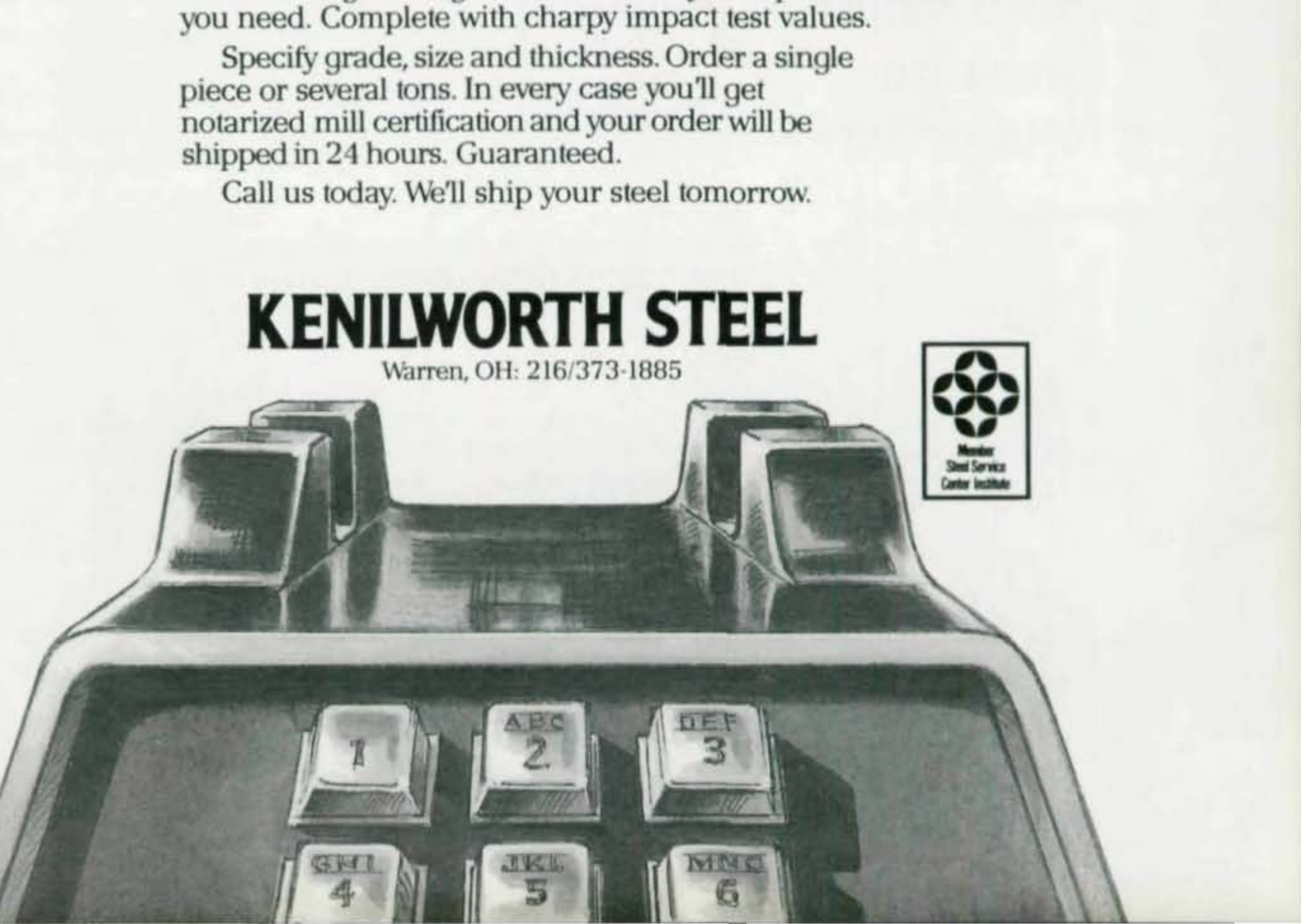

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Jointless Redecking Of Simple-Span Stringers

Elimination of unnecessary roadway expansion joints can often be accomplished without major structural rehabilitation

By Philip C. Pierce, P.E.

At the age of 40, the Tioga River crossing in Steuben County, NY, was showing signs of deterioration, though it still was capable of the same H20 truck loading capacity as when it was first opened to traffic. To ensure the bridge's future functionality, the State of New York committed to an extensive rehabilitation program.

A retrofit project eliminated expansion joints in part by splicing the bridge's stringers.

Existing Conditions

The structure was built in 1949 and is composed of four 100′ spans of simply supported steel wide-flange stringers. The cross section consists of five stringers spaced at 6′-9″ supporting a 24′-wide roadway with 2′-6″ brush curb and metal railing on both sides.

Although the stringers are simply supported, the deck was cast as a semi-jointless system. The deck slab is supported directly on top of the abutment backwalls and was cast with a cold joint and non-continuous reinforcement at the piers. A detail of copper flashing and asphaltic material over the pier joint in combination with a 3″ asphalt overlay sealed the joint; however, normal deterioration over 40 years—combined with the lack of continuous reinforcement—led to joint leakage and accompanying corrosion of the steel support components.

The existing stringer bearing details also were unusual. Both bearings at the center pier were fixed steel pedestal style. The bearing at the side piers and abutments were steel rocker style. A steel strap was welded on the top of the bottom flanges at the piers that apparently was intended to provide stability during erection and during the service life of the structure (the end spans are supported by rockers at each end).

By the late 1980s, some of the steel straps had fractured in the gap between the adjacent stringers. The resistive forces at the abutments from the approach pavement pressure and wingwalls and the frictional resistance at the interface of deck slab and abutment backwall had apparently restrained the structure from unstable longitudinal movements.

Rehabilitation Plans

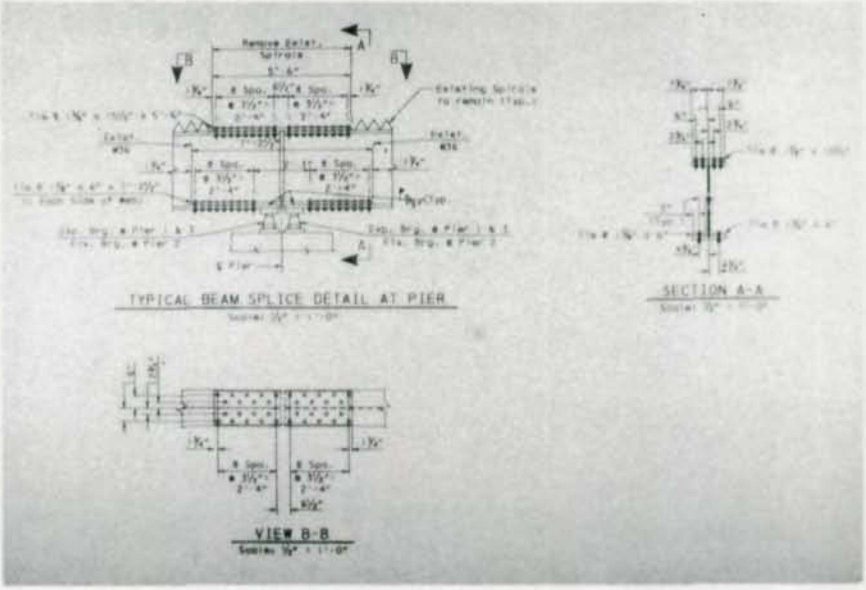
Although the existing structure was composed of multi-span simply supported steel stringers, NYSDOT desired the elimination of roadway expansion joints. This led to the consideration of splicing the stringers at the piers. The concept of making simple span prestressed concrete beams continuous for live load was judged to be directly adaptable to the simple steel stringers at this site. The bearing arrangements could be maintained without modification and the desired HS20 strength could be obtained without strengthening the stringers. And the deck detail at the abutments could be replaced with a similar arrangement, thereby avoiding expensive modifications.

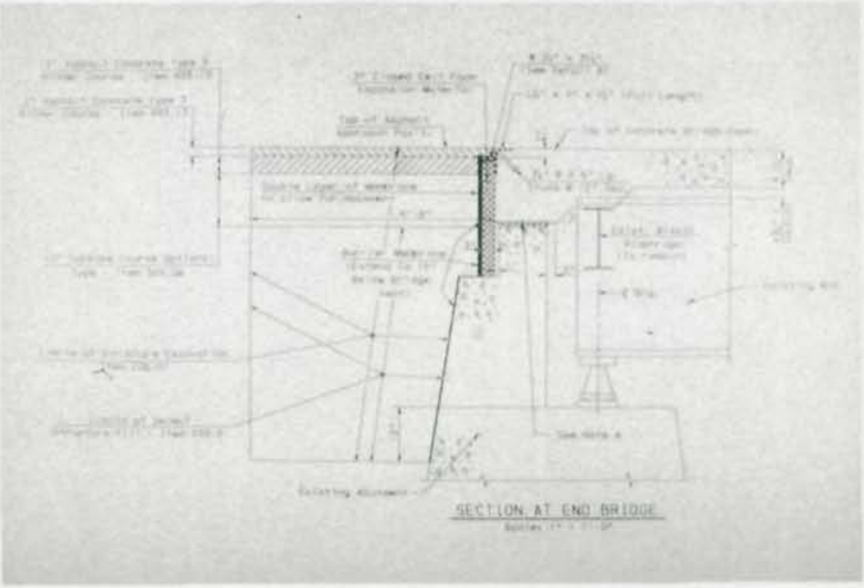
No formal cost analysis was performed to justify the elimination of the deck joints. In the words of one project participant, "It was done because it made sense."

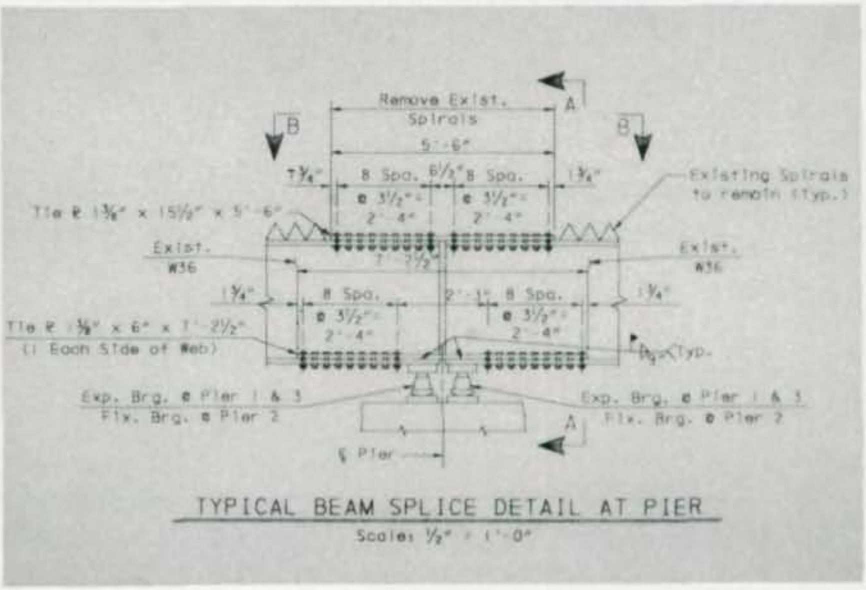
However, it was clear that eliminating deck joints did make good economic sense. Eliminating three expansion joints resulted in: reduced fabrication and erection costs; reducing bridge railing costs; reduced stringer connection costs from multiple rocker bearings; and savings from not having to interrupt the slab reinforcing and concrete placement. Note, though, that there was the additional cost of the stringer flange continuity connections.

In addition to first costs, eliminating the deck joints also provides substantial life cycle cost advantages. The elimination of deck joints will eliminate deterioration of the stringer ends, diaphragms and bearings due to the potential leakage of seal joints. Further, the tops of the piers are protected from the corrosive nature of roadway salts in the event of seal leakage.

Engineering services were provided by McFarland-Johnson. The structure rehabilitation contract was awarded in June 1990 for







TYPICAL BEAM SPLICE DETAIL AT PIER
Scale: 1/2" = 1'-0"

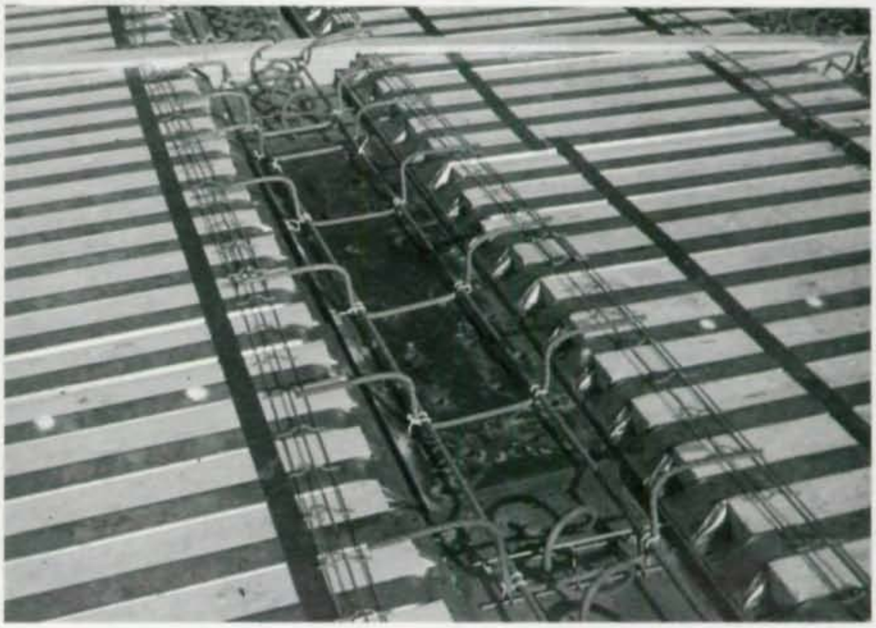
Construction photos of the bridge with the old deck removed show that foam was used for the bulkheads of the cold joints. Also, note that the new top splice has no bolts on one side to avoid continuity connection.

$649,152 to A.L. Blades & Sons. In addition to the redecking and stringer splices, the construction contract included installation of two NYSDOT standard two rail steel bridge railing and complete repainting. The bid also included $10,000 for ground and waterway protection and $43,000 for new approach guide railing. Steuben County Dept. of Public Works performed construction observation. The project progressed without significant problems and was opened to traffic on November 6, 1990.

Splicing Concepts

Since the new deck weight was lighter than the old deck, the continuity connection could have been designed and detailed to provide continuity for the change in deck weight. Alternately, the connection could have been detailed to defer continuity until after the deck replacement. After consideration, the latter alternative was chosen, primarily due to the difficulty in assuring what residual dead load would remain after the deck replacement. Further, the splice plates would have been significantly larger if they had been designed to support dead load continuity. Therefore, the splice was designed for continuity of live loads and future dead loads.

The details of the connection include a flange splice plate on top of the stringer ends and twin plates on the top of the bottom flanges. The splice plates are bolted to the existing stringer; however, due to interference with the top shoe of the bearings, a length of field fillet weld is utilized over the bearings to seal the edges against moisture penetration and to restrain the plate against buckling. The use of the fillet weld causes recognition of the fatigue characteristics of the connection, thereby leading to the size of the plate being governed by fatigue allowables. The continued use of the steel bearings permitted the elimination of the web splice plates.



With HS20 as the minimum desired live load capacity, the splice plates were designed for the negative moment associated with four span continuous behavior of HS20 live load with impact. The associated fatigue moments also were considered, and, as mentioned above, governed the size of the plates. It should be noted that truck loading was used for design in lieu of lane loading due to the rural location of the bridge.

Future Rating Considerations

The splice was designed to provide in excess of HS20 capacity based on fatigue considerations. Hence, ratings based on non-fatigue allowable will be in excess of

A closeup photo of the bridge during construction shows the new bottom plates.

HS20.

The existing stringers have capacity in excess of HS20 based on simple span behavior. Since the capacities exceeded the goal of HS20 live load, the rating can be conservatively taken as the lower of that based on simple span positive moment behavior or splice negative moment behavior.

If a higher capacity is desired, or if section loss occurs, it would be possible to consider ratings based on continuous behavior.

Installation Sequence

The existing bearing details and the desire for dead load simple span behavior led to the following recommended installation sequence:

1. Remove 10′ of deck at the piers (5′ on either side); retain shear connectors on beams.
2. Remove existing bottom flange steel straps; grind flush; install new bottom splice plates via bolting with bolts only snug tight in oversize holes; work to progress on one beam at a time.
3. Position new top splice plates to avoid later installation interferences with rebar mats. Drill and install bolts on one side only to avoid dead load continuity.
4. Remove remainder of deck; retain existing shear connectors.

5. Position deck forms, rebars, and anchor bolts for bridge railing.

6. Place new deck concrete in positive moment areas to within 5' of pier.

7. Drill/ream holes in the remaining side of the top splice, install bolts, and tighten all bolts in top and bottom flange splices.

8. Place closure pour of deck concrete in the 10' void above the piers.

9. Install bridge railing.

The contractor chose to repaint the structural steel after the deck removal and prior to installation of the new deck forms. The bolting sequence at the pier required touch-up painting of bolt heads and nuts. Elimination of touch-up of the bottom bolts may have been possible if all of the bottom bolts were installed at full tension prior to painting and deck concrete placement. Though such installation would cause minor bending of the bottom splice plates during initial concrete, it still might be judged acceptable.

Abutment Details

The existing slab was cast on top of the abutment backwall stem. It was considered appropriate to continue that type of slab support. A detail of the area is depicted in Figure 3, including an embedded armor angle and closed cell expansion joint material with membrane waterproofing intended to minimize moisture penetration through the slab/abutment stem sliding joint.

Adaptability

McFarland-Johnson was selected to provide engineering services for a similar bridge following the completion of the Tioga River crossing. The second structure supports the bridge crossing of Depot Street over the Canisteo River in the village of Canisteo. The bridge is composed of three simply supported spans of 63' each. Unlike the Tioga River Bridge, the Depot Street Bridge was built with conventional fixed and expansion bearing arrangements. Severe deterioration of the bearings led to the decision to replace them with NYSDOT standard neoprene bearings. The use of neoprene bearings required the installation of web splice plates in addition to the flange splice plates. This project is scheduled for construction in 1992.

The design of the rehabilitated Tioga River crossing did not require sophisticated analysis or design techniques. Also, this general concept has been utilized by others and is readily applicable to many older bridges.

Philip C. Pierce, P.E., has been involved with bridge and structural engineering for more than 20 years. He is currently the manager of the structures department at McFarland-Johnson, a transportation engineering firm headquartered in Binghamton, NY. This article is adapted from a paper presented at the 1993 National Steel Construction Conference.

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Reducing Parking Structure Costs

Parking structures framed with composite girders supporting a double tee deck system can reduce costs by 12% to 15%

By Robert M. Barnoff, P.E., Ph.D. and Edwin L. Mead, P.E.

During the past three decades, parking structure design has been dominated by precast concrete frames with double-tee deck beams and by steel frames with a cast-in-place post-tensioned concrete deck. Both systems had advantages—and serious flaws.

Shown above is the installation of steel side forms for the custom form of a composite system. Note a steel beam is already in position to receive the concrete flange.

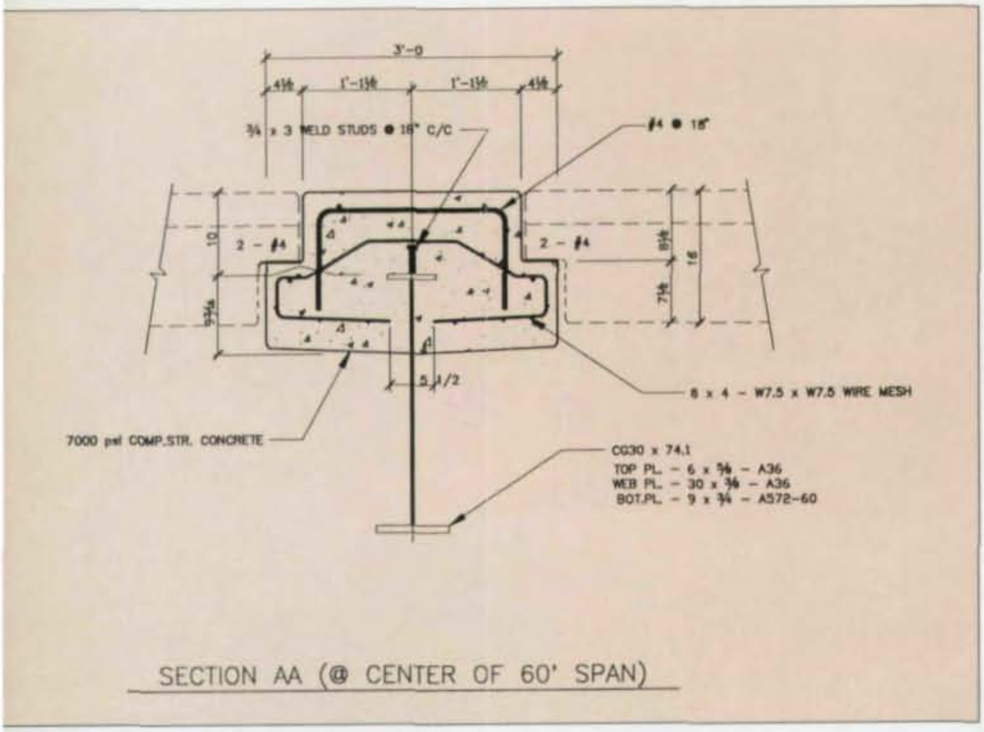
Steel-framed parking structures have smaller dead loads, lower shipping weight for structural components, and more rapid erection. Also, the flexibility of steel frames allow them to more easily adapt to temperature changes and lateral forces without causing excessive cracking in structural or non-structural components. However, steel parking structures may have higher labor costs for the cast-in-place concrete deck compared with the precast deck used on concrete parking decks.

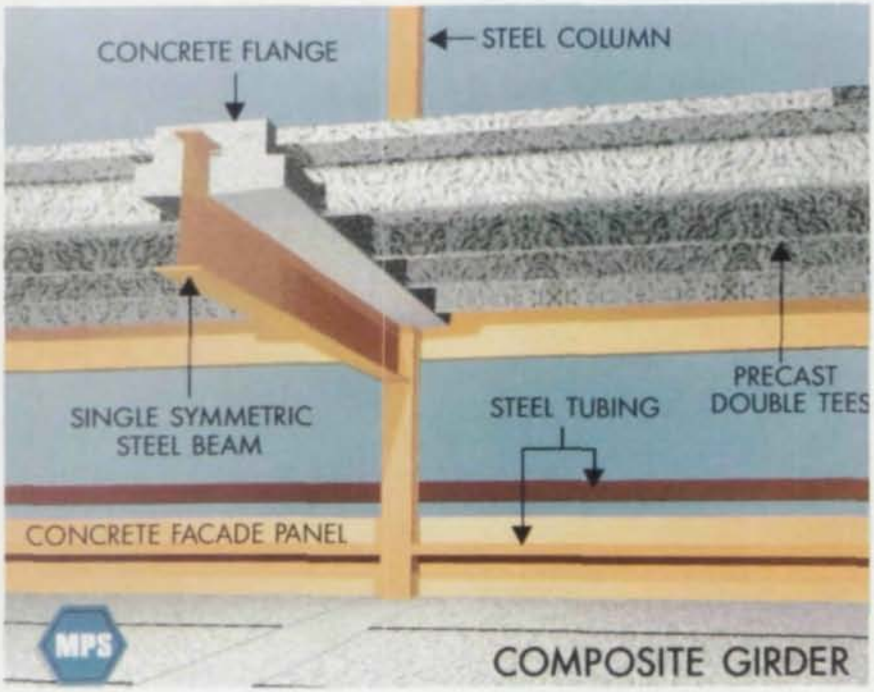
But while precast systems have lower deck labor costs, they also are heavy and require large foundations. Also, shipping costs may be high, especially in areas far from an existing precast plant.

Marrying Steel And Concrete

Recently, Mulach Parking Structures Corp. has developed and tested a system to combine the best features of both materials. The design features single symmetric hybrid composite girders framed into steel columns. These girders have spans varying from 54' to 62', depending on the building's footprint. Spacing of the girders vary from 20' to 40'. The girders are fabricated from three plates using continuous welds to connect the flanges to the web. End plate connections are used to connect the girders to the columns.

A unique feature of the composite girders is the encasement of the top flange in high strength concrete. This concrete also forms a portion of the parking deck and provides support for the precast concrete double-tee beams that span in the longitudinal direction between the girders.





Pictured above is a schematic diagram and a rendering of a composite girder.

Since most of the flexural compressive stresses in the composite girder can be accommodated by the concrete, the top flange of the single symmetric steel girder is smaller than the bottom flange. Further economies and reduction in steel weight are obtained by using A36 for the top flange and web and ASTM A572 Grade 60 or 65 for the bottom flange. Note, however, that in seismic zones, A572 Grade 50 steel is used for the bottom flange in accordance with the LRFD Seismic Provisions.

Lateral Loads

Lateral forces due to winds or seismic activity are resisted by frame action provided by the end plate moment connections in the transverse direction. Precast concrete panels that act as shear walls provide restraint in the longitudinal direction. If required, additional restraint can be provided by bracing in the interior longitudinal column lines. An alternate method for providing restraint in the longitudinal direction is to frame two structural steel tubes between the columns in the longitudinal direction to act as longitudinal bracing and to serve a dual function as a barrier.

The composite girder is usually cast at the job site in reusable steel forms. However, when the construction site has limited space, the girders may be cast off site and transported to the site by truck. Most parking structures have a clear span of approximately 60′ and composite girders with this span have an average weight of 20 tons.

An important detail for parking garages is adequate drainage for the deck ramps and level turn around areas. Two drainage swales are provided for the ramps by sloping the top of the concrete flanges. In the turnaround areas, the depth of the swales varied from maximum at the first interior composite girder to zero at the spandrel girder.

Materials And Design

Concrete decks are usually the weak link in parking structure design and the most cost effective—and in many instances, the most successful—remedy is the use of high quality concrete.

The concrete used in the precast composite girder, which was cast on site, has a design compressive strength of 7,000 psi and is obtained from commercial batch plants. Silica fume additive is used in the mix to increase strength and reduce permeability of the concrete.

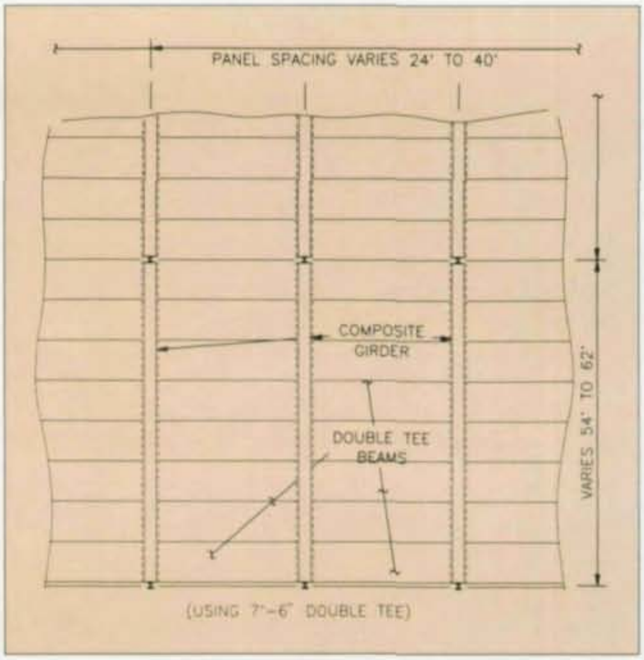
High water reducing admixture is usually introduced at the job site to improve workability of the mix. A water-cement ratio of 0.4 is used with this mix design, and entrained air admixture is used to provide resistance to freeze-thaw deterioration. Best results for this mix are obtained when crushed limestone is available for use as the coarse aggregate and 660 lbs of high early strength cement is used in each cubic yard of concrete. Design of the concrete mix, and placement and curing of the concrete, is done in accordance with American Concrete Institute, "Building Code Requirements for Reinforced Concrete" (ACI 318-89).

Longitudinal reinforcement in the concrete flange of the composite girder consists of deformed round steel bars. Steel in these bars conforms to ASTM standard A615-90, Grade 60. Transverse steel may be either deformed bars or welded wire steel fabric (ASTM A185-90a). All reinforcement is detailed, fabricated and placed in accordance with ACI 318-89.

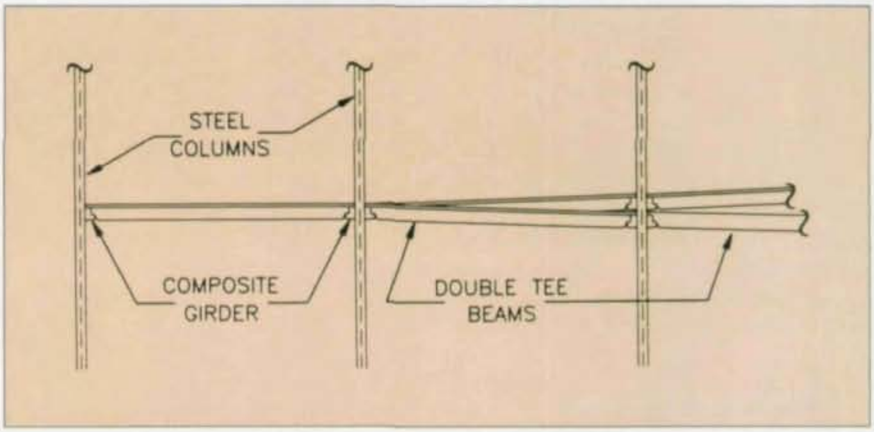
Concrete in the double-tee deck beams has an ultimate compressive strength of 5,000 psi.

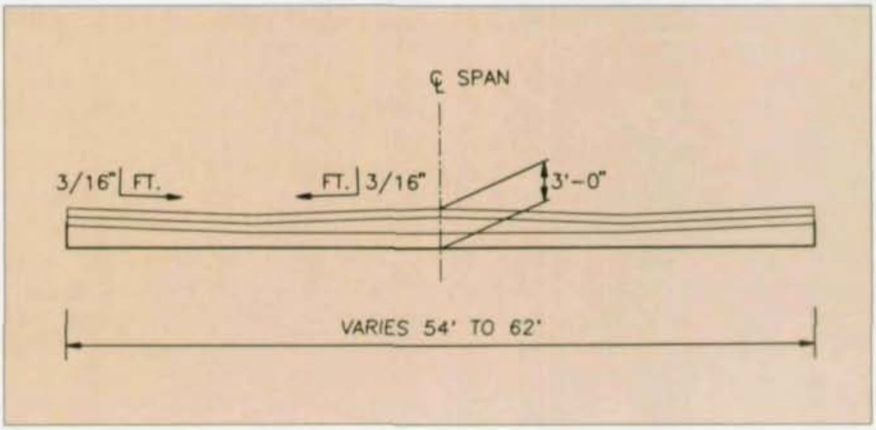
The steel girders are fabricated in accordance with the AISC LRFD Specification, with A572-60 or ASTM A572-65 used for the bottom flanges and A36 for the top flanges and webs. Steel columns, filler beams, bracing and miscellaneous steel are fabricated from A36 or A572-50. High strength A325 bolts are used.

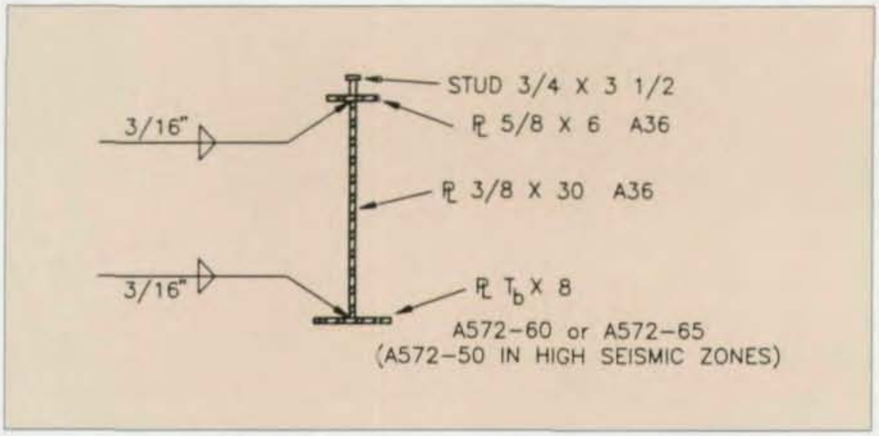
Since the girders are precast in supported steel forms on site, they are designed in accordance with shored construction requirements. This procedure, along with the use



Pictured, from top to bottom, are: a partial framing plan for a typical parking structure; a partial structural framing plan; and typical details of a single symmetric beam. Pictured on the following page is a composite girder elevation.







of single symmetric steel beams with the top flanges encased in high strength concrete results in considerable savings of structural steel. Fabrication costs are reduced by the use of end plate connections.

All structural steel is shot blasted to an SSPC-6 finish prior to applying a protective coating. An epoxy-polyamide coating of four to six mil thickness is applied to all surfaces in the shop with the exception of the top flange of the composite girders. After erection in the field, all exposed steel surfaces are coated with two to five mils of a high-build acrylic polyurethane enamel.

Lateral Forces

Wind loads and seismic forces acting in the plane of the girders are accommodated by using moment resisting connections between the composite girders and the steel columns. The design may conform to AISC Type PR (partially restrained) construction, or Type FR (fully restrained) construction.

Type PR construction requires the girders to be designed as simple beams and connections are designed to resist moments at the joints caused by lateral loads. Type FR construction requires beam to column connections to be designed as rigid connections.

An alternate to these two construction procedures is to consider partial restraint at the girder ends provided by the connections required to resist lateral loads. This restraint results in a significant reduction of the positive moment at the mid span of the girders with further savings in the weight of structural steel.

Composite action in the girders is developed by adhesive bond stresses between the embedded top flange and concrete. Building codes and specifications provide design criteria for fully encased beams, but are silent on beams partially embedded in the concrete.

However, researchers have recommended a bond stress of 300 psi for the design of beams with embedded flanges. Using this 300 psi bond stress, it was determined that shear connectors were not required.

But, since the conditions that exist in the composite girder are not covered by specifications, a decision was made to use 25% of the total number of shear connectors that would be required if no adhesive bond stress was present.

Test Results

In order to verify the design assumptions of this system, a 60% scale model was constructed and tested. Data from load tests indicated that full composite action was obtained between the steel beams and the high strength concrete flange. Stresses in the steel and concrete obtained from measured strains were slightly lower than theoretical stressed computed by an elastic analysis. This was due to the conservative assumption used in the elastic analysis that flexural tension stresses do not exist in any part of the concrete flange.

Vertical displacements at the center line of the span due to applied loads compared favorably with theoretical displacements obtained using the full uncracked section of the concrete to obtain the transformed moment of inertia of the girder.

A cost analysis between the new system and existing structures showed substantial savings.

A recently completed parking structure utilizing a steel frame with castellated steel beams supporting a composite concrete slab post tensioned in two directions used approximately 6.5 lbs. of structural steel per sq. ft. of parking deck—and was previously the lightest system available. However, the use of a composite girder system and double tee deck beams would result in a structural frame using only 5 lbs. of steel per sq. ft. of parking deck—a 23% weight savings.

In addition, the use of the double tee deck system eliminates the need for labor intensive cast-in-place concrete construction. In fact, the only cast-in-place concrete on the structure is in the deck at the small blockouts around the columns.

It is estimated that using the composite system will result in a savings of 12% to 15% in the total construction cost.

While maintenance costs were not compared, it should be noted that deck repairs for the composite girder system can be performed easily and rapidly by removing and replacing the double tee beams.

Robert M Barnoff, P.E., Ph.D., is a consulting structural and foundation engineer in State College, PA. Prior to establishing his consulting practice, he was a professor and head of the Department of Civil Engineering at Pennsylvania State University. Edwin L. Mead, P.E., is vice president of research and development for Mulach Parking Structures Corp., Bridgeville, PA. This article is based on a paper presented at the 1993 National Steel Construction Conference.

NATIONAL STEEL CONSTRUCTION CONFERENCE

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Steel Construction Accidents

Incorrect design or construction procedures can have fatal consequences

By Mohammad Ayub, P.E.

Each year, newspapers report fatal construction accidents. But for design and construction professionals, these accounts are of only minor interest because they only report what happened, not why.

However, an examination of several steel construction accidents reveals a common thread: incorrect design or construction procedure. While this article will only discuss two accidents, information on several more accidents is contained in "Case Studies Of Steel Construction Accidents" in the *1993 Proceedings of the National Steel Construction Conference.*

Case Study: Four-Story Computer Facility

Description of accident: Several beams collapsed during the erection of structural steel for a computer facility and three ironworkers on the fourth level of the structure were killed.

The framing for the 320′ x 320′ structure consisted of steel columns and composite steel girders with infill beams. The floor consisted of 2½" concrete over 2" deep composite deck. Typical bay size was 40′ x 40′ and the floor-to-floor height was 16′.

At the time of the accident, construction activity was limited to a small area in the northwest corner of the building. Steel erection was proceeding in the area bounded by column lines 1 and 4 and column line A and C up to the roof level. The third floor structural beams, the fourth level beams and the roof level beams already were erected. The steel deck floor was placed up to the third level. And in the remainder of the area, preparations were underway to erect additional steel.

The beam-to-beam connections and beam-to-column connections at different levels were completed to varying degrees. Up to the third level, the shear connections were completed for beam-to-beam and beam-to-column framing. The fourth level beams and the roof level beams were held in place only by the erection bolts, with the number of erection bolts varying from one to five depending on location. Welding of the flanges of the girders to the column was not yet completed, so there were no moment connections. *Therefore, in the case of any bending moment, the erection bolts would be required to resist the additional force arising from the moment.*

Discussion: Shop drawings indicated that all bolts to be used in beam-to-beam connections be 7/8" diameter A325 bolts. For the cantilevered beam-to-column connections, 1" diameter A490 bolts were specified. When the cantilever beams were erected on the roof and fourth level, they were held in place by 7/8" diameter erection bolts in 1 1/16" diameter holes.

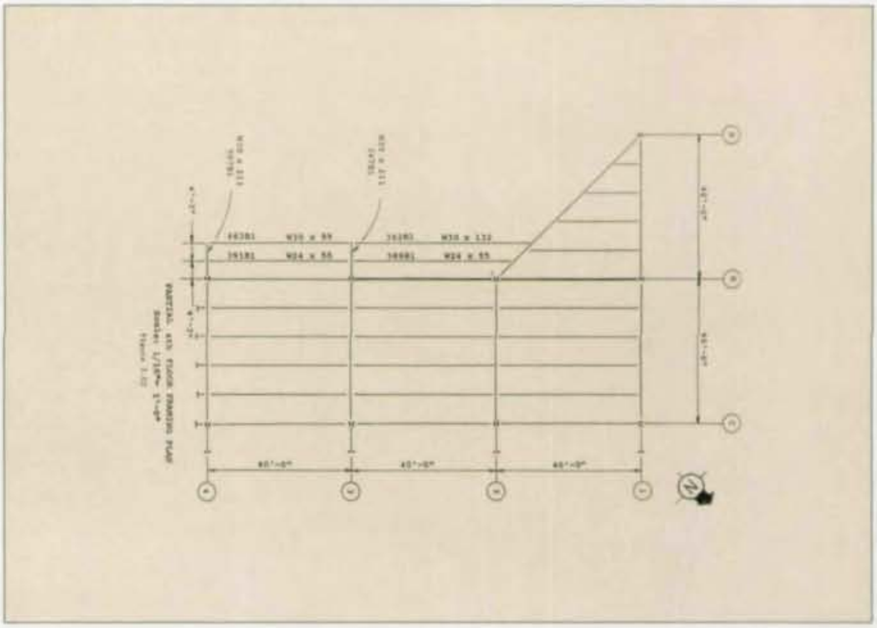
Immediately prior to the accident, a crew of four ironworkers was engaged in levelling the cantilevered beam framing to column line B-3 on the fourth level. The levelling procedure was necessary because permanent shear connections were being made at the joint where the cantilevered beam framed to the column. The procedure was to use a wire rope come-along of two-ton rated capacity with the top end fastened to the roof cantilever beam directly above the fourth level cantilever beam and the bottom attached to the cantilever beam being levelled.

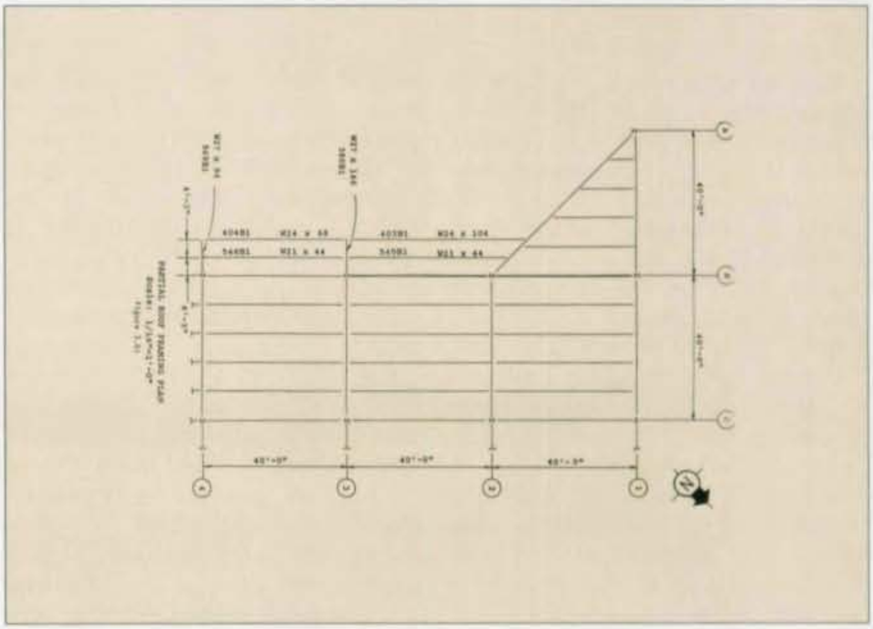
The intent of the design was to support the cantilever beams on column lines 3 and 4 by providing moment connections at the column joints. The moment due to the cantilever action would be transferred to the columns through full penetration welds of the flanges and the beam shear due to gravity load would be resisted by 1" diameter A490 bolts.

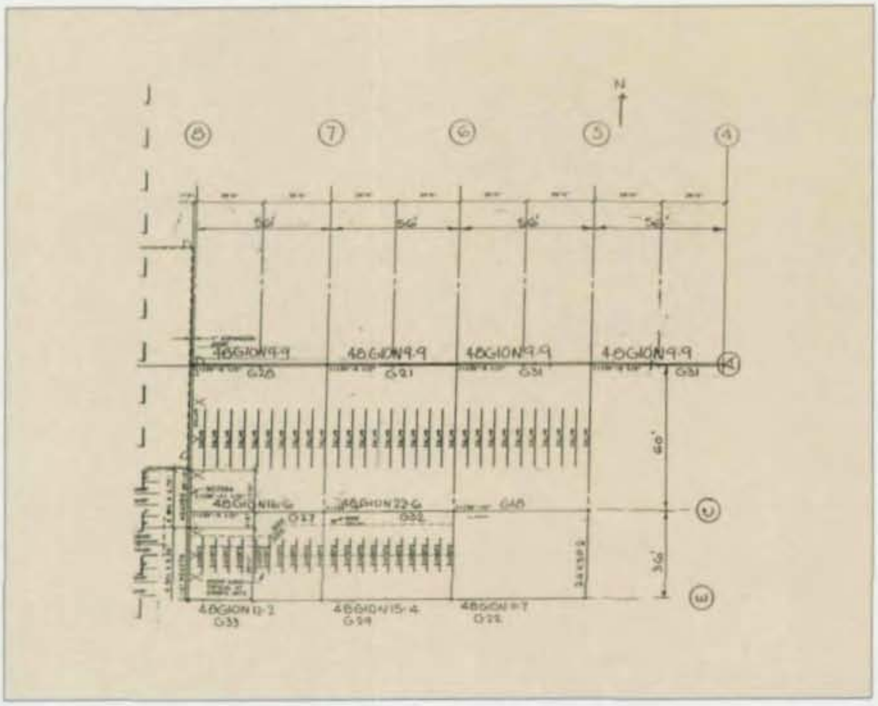
The steel erection company erected the cantilever beams and simple span beams. The simple span beams were connected to the cantilever beam by ⅞" diameter erection bolts. After the beams were erected, they were left in a self-supporting state. The cantilever connections to the column were subjected to both the gravity load of the cantilevered beam plus the dead load of the simple span beams connected to it and the moment due to the eccentricity of the gravity load. *The ⅞" diameter erection bolts were therefore subjected to the shear force due to the total gravity load and additional shear due to the moment at the connection.*

Loads in the fasteners prior to the erection procedure were as follows:

- The three ⅞" diameter A325 erection bolts holding the roof cantilever beam at column line B-3 were placed in the second, fourth and sixth holes from the top of the column shear plate. The dead load of the perimeter beams and the filler beams produced a reaction of 7.5K and a moment of 45K-ft. at the connection to the columns. Due to these loads, the resultant shear in the top and bottom erection bolt is 45K acting in single shear. Without the capacity resistance factor and load factors, the limit state shear value of ⅞" diameter bolts with threads excluded from the shear plane was 43.3K. The actual test result of ⅞" diameter bolt in 1.062" diameter hole had an ultimate capacity of 43.9K and an overstress of 1%. However, with a load factor of 1.4 and resistance factor of 0.65, as was required by AISC, the force in the bolt was exceeded by a factor of 2.18.
- Five erection bolts in the first, second, fourth, sixth and seventh holes from the top were used for the roof cantilever beam at column line B-4. There was a reaction of 3.24 K and a moment of 18.33K-ft. The resultant shear in the extreme top and bottom bolts was 8.52K. Based on a load factor of 1.4 and a resistance factor of 0.65, there was a factor of safety of 2.35.
- Five erection bolts in the first, second, third, fifth and seventh holes in the shear plate were used for the fourth level cantilevered beam on column line B-3. A reaction of 9.73K and a moment of 58.6K-ft. was computed due to the dead load of the beams and the eccentricity of the loads. The maximum shear in the farthest bolt from the neutral axis was 34.5K. The ultimate shear capacity without any load factor nor any resistance factor was 43.3K. Therefore, the factor of safety against ultimate load was only 1.25. However, if the load factor of 1.4 and a resistance factor of 0.65 were taken as required by AISC, the bolt was overstressed by a factor of 1.71.







• Five erection bolts in the first, second, fourth, sixth and seventh holes in the shear plate were used for the fourth level cantilever beam on column line B-4. There was a reaction of 5.0K and a moment of 27.54K-ft. due to the self weight of the beams. The top and bottom bolts were subjected to a shear force of 12.78K, which was well within the ultimate shear strength of 43.3K. Taking the load factor of 1.4 and resistance factor of 0.65, the factor of safety was only 1.6.

When the come-along was tensioned (for computations, it was assumed to be fastened 12" from the center line of the perimeter beams along the cantilever beam) it would impose an additional load of 8.8K from the fourth level to the roof cantilever beam. Due to the this additional load, the roof cantilever beam on column B-3 would be subjected to an additional reaction of 8.8K and an additional moment of 58.7K-ft. The cantilever beam-column joint at the roof level at column B-3 would therefore be subjected to a total reaction of 16.25 and a moment of 104K-ft. This would produce a shear force of 104K and the actual force would exceed the ultimate strength by a ratio of 2.4. If the load factor and resistance factors were used, then the force in the bolt would be exceeded by a ratio of 5.1 with the most stressed fasteners being located in the roof and fourth level cantilever beams.

Conclusion: The cause of the collapse was the overstressing of the erection bolts of the roof cantilever beam on column line B-3 and was precipitated when tension was gradually applied to the come-along.

The top end of the come-along was fastened to the roof cantilever beam, which resulted in additional load on the connection at column line B-3 at the roof level. This procedure was adopted without any engineering calculations to verify the adequacy of the existing connection. *The method of erection for the cantilever beams should have been evaluated for the number, size and location of erection bolts at each cantilever connection to the column before erection was undertaken.*

Case Study: One-Story Warehouse

Description of Accident: Structural steel framing for a one-story warehouse was being erected when a portion collapsed. Structural framing bounded by column lines 4 through 8 and A through E collapsed minutes after a fourth stack of roof decking material was placed at the top of longspan roof joists. There were two iron workers on the roof at the time; one plunged to the ground along with the collapsing frame and was killed, while the other fell along with the joist girder and sustained injuries.

The structure consisted of structural steel framing with 12"x12"x3⁄8" structural tube columns and joist girders spanning in the east-west direction and longspan and bar joists spanning in the north-south direction. The typical bay was 56' in the E-W direction and varied from 36' to 60' in the N-S direction. The columns were supported on concrete pedestals and foundation walls with anchor bolts. All concrete pedestals were supported by individual footings.

Construction activity was bounded by column grid lines A through E in the N-S direction and column grid lines 5 through 8 in E-W direction. In addition, columns at grid location 4A also were erected. The roof framing members were all placed in position except for the roof joists in the bay bounded by column lines 5 and 6 and column lines C and E. The joist girders along column lines A, C and E were in place in the E-W direction. The long span joists 36LH09 spanning in the N-S direction between column lines A and C were placed from column lines 5 through 8. Between column lines E and C, bar joists 24K7 also were placed from column lines 6 through 8. The longspan joists and bar joists also were positioned on the column lines, as required by contract documents.

Typically, the structural tube columns had shop welded base plates and were supported on concrete pedestals/foundation walls. At the center of the underside of the base plates were shims to fill the space between the top of the concrete and bottom of the plate. The base plates were connected to the embedded 1½" diameter anchor bolts with nuts. No levelling nuts were provided.

Roof decking materials, consisting of wide steel decks 28'-2" long with each pile weighing 4,931 lbs., were placed on the roof along column line A. After placement of the fourth pile between column lines 5 and 6, one iron worker proceeded to walk toward column 7C, while the other was positioned at the mid-position on column line A between column lines 5 and 6.

Discussion: The lateral stability of the steel frame for the completed structure in the N-S direction was based on the permanent structural crossbracings provided in grid lines 1 and 8 and the roof steel deck diaphragm. The lateral stability of the frame in the E-W direction was based on the rigid connections of the joist girders at each column line created by providing welded connections at the top and bottom chords of the joist girder. At the time of the accident, the progress of the construction was limited to four bays in the E-W direction and two bays in the N-S direction. Therefore, the lateral stability of the erected frame would depend upon the rigidity of the connections of the erected structure or upon temporary bracings.

The AISC Code of Standard Practice in Section 7.9.1 states that temporary supports, such as temporary guys and braces, will be determined, furnished and installed by the erector.

In the N-S direction, which was the direction of the collapse, the longspan joist between column lines A & C and the bar joist between lines C & E were supported, respectively, by 13/16"x2" and 9/16"x2" slotted holes at their bearing seats on the grid lines. The connections at the column lines were made with one 3/4" diameter A325 bolt and one 1/2" diameter bolt for the longspan and bar joists, respectively. The bottom chords of the joists were not connected to the column.

Eyewitness accounts reported that the bolts were handwrench tightened. The filler longspan joists and bar joists were simply placed over the top of the joist girders with neither bolted nor welded connections. The bottom chords of the filler joist also were not connected to the joist girders. Therefore, the longspan joists and bar joists on the grid lines in the N-S direction did not provide any rigidity. The stability of the frame could then only depend upon the rigidity of the column base connections to the footings or base walls.

AISC publication D-801 "Column Base Plates', has given guidelines for the tensile capacity of anchor bolts and the minimum edge distance for hooked bolts. For A307 bolts, the minimum embedment length is 12 times the diameter and the minimum edge distance is five times the diameter but not less than 4".

The joist girder spanning in the E-W direction between column lines 6 & 7 on column line A was analyzed to determine its load carrying capacity due to the application of the construction load on the day of the accident. It was determined that the compressive stress in the top chord was 4.65 ksi. Since the critical buckling stress, based on the unsupported length of 56' was determined to be 2.69 ksi, *failure of the joist girder under the application of the construction load would be expected*. In fact, the joist girder manufacturer in its general notes for the erection had stated that "no loads shall be placed on the joist girder until the joists bearing on the girder are in place and welded to the girder" to provide the lateral restraint to the girder top flange.

Conclusion: The cause of the collapse was the instability of the frame caused by the inadequate temporary connection of the members and placement of construction materials over the roof members. Temporary bracing of the structural steel framing as required by the project specification and recognized by the industry as the generally accepted means to brace the steel structure during construction were not provided by the steel erection company.

Mohammad Ayub, P.E., is chief of the Division of Engineering in the Office of Construction and Engineering for the Occupational Safety and Health Administration, Department of Labor.

1993 National Steel Construction Conference Exhibitors

(As of January 31, 1993)

Exhibitor	Booth
American Galvanizers Association	221
Am. Inst. for Hollow Struct. Sections	815/817/819/821
American Institute of Steel Construction, Inc.	TBA
The American Punch Co.	610
Arkansas Steel Processing, Inc.	404
Behringer Saws, Inc.	828/830/832
Nicholas J. Bouras	207/306
CNA Insurance Co.	TBA
CANAM Steel Corp.	605/704
Daito Seiki Co., Ltd.	421/423/520/522
Design Data	210/212/214/216
Dogwood Technologies	314/316
ELCO Industries, Inc.	204
Elite Equipment, Inc.	405/407/504/506
ITW Buildex	111
Kaltenbach	828/830/832
LeJeune Bolt Co.	TBA
The Lincoln Electric Co.	217
Lohr Structural Fasteners	205/304
MetroSoft	207
Mid-South Bolt & Screw	104
Modern Steel Construction	TBA
NAPTech, Inc.	307
National Institute of Steel Detailing	TBA
NSS Industries	106
Nucor Corporation	804/806/808/810/812
Peddinghaus Corp.	311/313/315/317/410/412/414/418
Portland Bolt	321
Ram Analysis	220
Research Engineers	511
St. Louis Screw & Bolt Co.	521
Service Supply Co.	420
Southern Coatings, Inc.	406
Steel Solutions, Inc.	515/517
SteelCad International	505/507/604/606
Structural Software Co.	108
Structural Steel Systems, Ltd.	513
Towe Iron Works	312
TradeARBED	305
J&M Turner, Inc.	305
Voss Engineering	121
Welded Tube Co. of America	815/817/819/821
Westbrook Engineering Co.	105
Zentech	TBA

American Galvanizers Association
Booth #221

AGA has published a number of new technical brochures including "Recommended Details For Galvanized Structures." Also available is a 1993 publications index. Additional information is available through free AGA seminars.

For more information contact: American Galvanizers Association, 12200 E. Iliff Ave., Suite 204, Aurora, CO 80014 (303) 750-2900; FAX (303) 750-2909.

American Institute For Hollow Structural Sections
Booth #815/817/819/821

The AIHSS is a non-profit technical organization committed to advance and improve the use of hollow structural sections/structural steel tubing in building, bridge and special structure applications. The institute serves as an information resource center for the design profession and construction industry and publishes design aids and holds seminars.

For more information contact: Frederick J. Palmer, AIHSS, 929 McLaughlin Run Road, Suite 8, Pittsburgh, PA 15017 (412) 221-8880.

American Punch Co.
Booth #610

A complete new line of punches, dies, and shear blades for Angleline, Beamline and Portable punches is available from the American Punch Co. Standard punches with matching dies are offered in round sizes from 1/8" to 1½" diameter in 1/32" increments.

For more information contact: The American Punch Co., 685 South Green Road, Cleveland, OH 44121 (800) 243-1492; FAX (216) 381-1037.

Arkansas Steel Processing
Booth #404

This full service structural steel processor is located adjacent to the Nucor-Yamato wide flange beam mill near Blytheville, AR. ASP offers an array of processing services for steel fabricators, including splitting tees, cambering beams, cutting to length, drilling and coping beams, and attaching bolted connection angles.

For more information contact: Arkansas Steel Processing, 17W697 Butterfield Road, Oakbrook Terrace, IL 60181 (708) 495-8600; FAX (708) 620-6394.

Nicholas J. Bouras
207/306

United Steel Deck has enhanced its ability to produce custom deck systems by networking with the affiliated companies of Nicholas J. Bouras, Inc. Special finishes, such as plasticol, or materials such as stainless steel, are being used to produce custom decks and panels that solve durability and environmental problems caused by some industrial atmospheres. Special finishes combined with the roll forming and bending capabilities of United Steel Deck, Inc., can provide solutions to most decking needs.

Contact: United Steel Deck, Inc. (Nicholas J. Bouras, Inc.), 475 Springfield Ave., Summit, NJ 07902-0662 (908) 277-1617; fax (908) 277-1619.

Canam Steel Corp.
Booth #605/704

Canam Steel provides quality open web joists, girders and trusses to fabricators from coast-to-coast from its five North American plants. Development of its own state-of-the-art design and estimation software programs enables Canam to assist engineers and customers in providing the most efficient and economical roof and floor system.

For a free 76-page catalog, contact: Sales Manager, Canam Steel Corp., 2000 West Main St., Washington, MO 63090 (314) 239-6716.

Design Data
Booth #210/212/214/216

The SDS/2 Steel Fabrication System developed by Design Data allows steel fabricators to save both time and money. The system allows users to integrate detailing with estimating, production control, automated shop equipment and engineering analysis and design. This integration eliminates the redundant entry of data and ensures data consistency allowing fabricators to achieve productivity levels never before possible. Version 5.4 features enhanced capabilities and will be demonstrated during a technical product presentation at the NSCC show, as will the company's third-party engineering interface, DesignLINK.

For information or to schedule a demonstration call Design Data at (800) 443-0782 or (402) 476-8278.

Dogwood Technologies
Booth #314/316

Procedural Detailing System (PDS) is an automated structural steel detailing system operating under UNIX. Structural members are entered on a text input screen and are automatically generated. PDS details beams, columns, horizontal and vertical bracing, girts, purlins, lintels, door frames, and pan and grated stairs with breakovers and landings. A bill of material also is automatically created. An interactive CAD graphics model and CNC download are available.

For more information, contact: Dogwood Technologies, 420 Bearden Dr., Suite 100, Knoxville, TN 37919 (800) 467-0096.

Elite Equipment
Booth #405/407/504/506

The company will be displaying several new pieces of equipment at this year's NSCC show, including the new CM-2044 Beam Profile Cutting Machine, CM-7/75 Web Punch, C-16 Column Rotator, and MD60-TV Measuring System with Custom Material Handling System. Information also will be available on the company's new line of Tank Turning Rolls Welding Positioners and Sub Arc Welding Systems, as well as a list of used steel fabricating equipment for sale.

Contact: Elite Equipment Inc., P.O. Box 3417, Tustin, CA 92681 (714) 569-1050; FAX (714) 569-1009.

ITW Buildex
Booth #111

At this years NSCC Show, the company will be demonstrating the new Autotraxx Fastening System for metal decking. This system is designed for attaching deck to structural or deck to deck (stitch). Fasteners for this system are 10-14 x ⅞" or 12-14 x ⅞" ICH Traxx 1 for deck to deck attachments and 12-24 x ⅞" ICH Traxx/4 or 12-24 x 1¼" ICH Traxx/5 for deck to structural steel (up to ½").

Contact ITW Buildex, 1349 West Bryn Mawr Ave., Itasca, IL 60143 (708) 595-3500; FAX (708) 595-3549.

LeJeune Bolt Co.
TBA

LeJeune Bolt Co. will feature LeJeune Tension Control bolts at this year's NSCC show. Installed with a special non-impacting wrench, the bolts exhibit irreversible tip shear-off, indicating a specific tension (clamp load) has been accomplished. Rotation should always occur between nut and washer. The nut is driven by a clockwise force as an equal counterforce is applied to the bolt tip, assuring that all bolts are pulled into direct tension. Neither wrench nor operator affects the installation characteristics or installed tension value. Available in ASTM A325 Type I & III and A490 grade, these assemblies can be mechanically galvanized to ASTM B695. Diameters are ⅝", ¾", ⅞", 1", and 1⅛".

For more information contact: LeJeune Bolt Co., 8330 West 220th St., Lakeville, MN 55044-9106 (612) 462-5521; FAX (612) 469-5893.

The Lincoln Electric Co.
Booth #217

The Lincoln Electric Co. will feature the Invertec V300 PRO, Pro-Cut 40 and inverter Racks at the NSCC show. The portable Invertec V300 PRO is a 300 amp arc welding power source utilizing single phase or three phase input power to produce either constant current or constant voltage outputs. The Pro-Cut 40 is a constant current, single range continuous plasma cutting power source designed to be lightweight and easily portable.

For more information contact: The Lincoln Electric Co., 22801 St. Clair Ave., Cleveland, OH 44117-1199 (216) 481-8100.

Lohr Structural Fasteners
Booth #205

Lohr's line of domestic high-strength Tension Control Fasteners and Hex Head Fasteners are part of a total assembly concept, which is designed to solve the problem with current specifications not requiring fasteners to be lubricated and tested as matched assemblies to assure they will work together properly when shipped. Lohr's fasteners are domestically manufactured, lubricated and tested as pre-assembled sets, to assure that when they arrive at the project they can be properly tensioned.

Contact: Lohr Structural Fasteners, P.O. Box 1387, Humble, TX 77347 (800) 782-4544.

Metrosoft
Booth #207

Metrosoft will be demonstrating its latest release of ROBOT V6, a structural analysis and design software package. The totally integrated system provides full graphical input and result processing and handles simple as well as complex problems with ease.

Contact: Metrosoft, Inc., 332 Paterson Ave., East Rutherford, NJ 07073 (201) 438-4915.

NAPTech
Booth #307

NAPTech is the largest Induction Heat Bender in North America. Bending capabilities range from 2" to 66" O.D. in diameter and up to 4" in thickness. The company bends all shapes—including squares, rectangles, and beams—without wrinkles or distortions. Radii of 3x diameter can be achieved.

For more information contact: NAPTech, Inc., 1009 Whetherly Way, Alpharetta, GA 30202 (404) 644-4464; FAX (404) 475-7898.

National Institute of Steel Detailing
TBA

NISD will feature a video presentation at the NSCC. Also available are a number of brochures describing the many programs developed by the Institute.

For more information contact: National Institute of Steel Detailing, Inc., 1799 Portola Ave., Suite 3, Livermore, CA 94550-1633 (510) 443-3363.

Nucor Corporation
Booth #804/806/808/810/812

Three divisions of the Nucor Corporation will be exhibiting at the NSCC show. Nucor Fastener will display its full line of structural nuts, bolts and washers, mechanically galvanized for maximum corrosion resistance. Nucor-Yamato Steel will exhibit the range of structural steel beams and members it produces. The Vulcraft Division will display it capability to produce long span joists and steel floor and roof deck.

Contact: Nucor Corp., P.O. Box 6100, St. Joe, IN 46785 (219) 337-5611; FAX (219) 337-5394.

Peddinghaus
Booth #311/313/315/317/410/412/414/418

Peddinghaus Corp. will exhibit the latest technologies for the automated processing of structural steel at the National Steel Construction Conference in Las Vegas. Application problem solving for the structural fabricator will be featured, with special emphasis on control/software enhancements for existing equipment. Innovative methods of fabricating beams, columns, angle channel and plate will be shown. The principles of automated punching, drilling, sawing, burning and plasma cutting will be explained.

Contact: Lyle Menke, Peddinghaus Corp., 300 N. Washington Ave., Bradley, IL 60915 (815) 937-3800.

Portland Bolt
Booth #321

Portland bolt is a domestic manufacturer of headed and threaded fastening hardware up to 5½" diameter and 40′ in length. The material is produced and certified to the latest ASTM and AISC specifications. Testing is done in-house. The company specializes in carbon, alloy, non-ferrous fasteners and hot-dipped galvanized coatings. A recent expansion into a 55,000-sq.-ft. facility has increased production and reduced lead times and costs.

For more information contact: Portland Bolt & Manufacturing Co., Inc., P.O. Box 2866, Portland, OR 97208 (800) 547-6758.

Ram Analysis
Booth #220

The newly released RAMSTEEL Version 3.0 offers several new features for designing fully integrated floor framing and building design software. In addition to composite and non-composite rolled and built-up shapes, the program is now capable of selecting and designating steel joist based on S.J.I. joist designations or user defined tables. K-, LH- and DLH-Series or custom joist may be selected and joist girder designations will automatically be determined. Load diagrams may be output as well as shear, moment and deflection diagrams. Material takeoffs may be obtained for joist as well as beams, girders and columns. More than 30 additional modeling, performance and design enhancements have been added including: fenced in area zoom; base plate design; and support for the RAMSTEEL column module. The program's powerful graphical modeling capabilities create a model of the entire structure from which the distribution of loads, live load reductions and member interactions are automatically determined per local building code and optimally sized per specified design code (ASD, LRFD, SJI).

For more information contact: Gus Bergsma, Ram Analysis, 5315 Avenida Encinas, Suite M, Carlsbad, CA 92008 (800) 726-7789; FAX (619) 431-5214.

Research Engineers
Booth #511

STAAD-III/ISDS is an integrated software system for structural analysis, design and drafting of steel, concrete, timber, and aluminum structures per American and International codes. Research Engineers will be demonstrating the program at this year's NSCC show, along with AutoSTAAD/MAX, an AutoCAD-based integrated software system for structural analysis, design, drafting and detailing. The company's project management software—AutoProject—runs within AutoCAD's superior graphics environment. All AutoProject output, including the Network, Bar Diagram, Resource/Cost Histogram, Performance Curves, and customized reports are generated as AutoCAD drawings, which allows the use of all standard AutoCAD commands and functions.

For more information contact: Research Engineers, Inc., 1570 N. Batavia, Orange, CA 92667 (714) 974-2500; FAX (714) 974-4771.

St. Louis Screw & Bolt
Booth #521

This 106-year-old industrial fastener manufacturer specializes in the steel construction industry with an emphasis on bridges. St. Louis Screw & Bolt maintains a huge production range of structural bolts, manufacturing the entire range from ½" diameter through 1½" diameter, with no limits on length. They are made and inventoried as both Type I and Type III, as well as weathering steel.

For more information, contact: St. Louis Screw & Bolt Co., 6900 North Broadway, St. Louis, MO 63147 (800) 237-7059; FAX (314) 389-7510.

Service Fastener Center
Booth #420

Service Supply Co., Inc., is a leading distributor of all types of fasteners for the steel construction industry. The company has 44 service centers across the country. On exhibit at the NSCC will be the company's complete line of structural products, including tension control bolts manufactured by Infasco.

For more information contact: Service Supply Co., Inc., 603 E. Washington St., Indianapolis, IN 46206 (317) 638-2424; FAX (317) 634-9087.

Southern Coatings, Inc.
Booth #406

Southern Coatings is a leader in providing environmentally conscious primers and topcoats for the steel fabrication and joist manufacturing industries. The Enviro-Guard line represents lead- and chromate-free primers and coatings that offer superior protection against rust and corrosion on steel. Complete information on the Enviro-Guard VOC compliant primers as well as Chemtec 606 Weather Base Epoxy Zinc Rich Primer, Chemtec 608 Inorganic Zinc Rich Primer and Dura-Pox 646 Epoxy Mastic High Build system will be available at the NSCC show.

For more information contact: Southern Coatings, Inc., P.O. Box 160, Sumter, SC 29151 (800) 766-7070.

Steelcad International
Booth 505/507/604/606

Steelcad International will be demonstrating the latest releases of 10 new programs covering the full spectrum of the steel fabrication industry. The new capabilities include automatic downloading from engineering design programs. The programs integrate shop drawings, material lists, CNC files, erection drawings, and all production control on a PC-based system.

For a free information packet contact: Steelcad International, 2265 Lee Road, Suite 201, Winter Park, FL 32789 (800) 456-7875.

Steel Solutions
Booth #515/517

Steel Solutions will be demonstrating its STEEL 2000 automated fabrication management system at this year's NSCC. The system includes programs for estimating, material control, production, service center and accounting. Featured this year will be enhancements to the automated production control and service center modules. The automated production control module allows the fabricator control of all cutting lists and CNC machinery directly from one source of information. This eliminates the redundancy of re-inputting the piece mark information for CNC tool programming. In addition, the program allows the shop floor to communicate directly with the piece mark status database for instantaneous production status recording. And the productivity of every workstation can be monitored on a real time basis.

Contact: Richard Inserra, Steel Solutions Inc., 2260 Flowood Dr., P.O. Box 1128, Jackson, MS 39215-1128 (601) 932-2760; FAX (601) 939-9359.

Structural Software Co.
Booth #108

A new Production Control program from Structural Software features a comprehensive piece and labor tracking system that follows every item in a job through every phase of fabrication. The system also generates detailed tracking reports and percentage completion bar graphs on the status of individual pieces or of entire jobs and allows the user to see what shop work remains to finish a given piece mark, sequence or job. Shipping tickets, loading reports and other status reports show through what shop stations a piece has passed, what work was done, who did the work, and the time it took. Sorted lists show individual piece weights, assembly weights and even the weight of all the steel on a drawing. Other add-on programs, such as program control, Nucor-Yamato Steel package, PC/PO Link and Inventory Control, also offer money-saving potential.

For more information contact: Structural Software Co., 5012 Plantation Road N.E., P.O. Box 19220, Roanoke, VA 24019-1022 (703) 362-9118.

Structural Steel Systems
Booth #513

This subsidiary of Peddinghaus is dedicated to the promotion and sale of pre-owned structural steel and plate fabricating equipment. Structural Steel Systems offers a complete line of services to the equipment buyer and seller, including service, financing, leasing, re-manufacturing, CNC controls retrofitting, warranty contracts and brokerage service.

For more information contact: Structural Steel Systems, Ltd., Rd. #1, Box 125, Hellertown, PA 18055 (215) 838-7338.

TradeARBED
Booth #305

This major steel manufacturer offers a variety of shapes and sizes. Tailor-made wide flange shapes up to 44" deep and 920 lbs./ft. are available for bridges, long spans and columns in high-rise buildings. HISTAR Quality Steels offer high strength, low carbon, good toughness and excellent weldability and are available in 50, 60, 65 and 70 ksi yields. HISTAR is excellent for H bearing piles, gravity columns and trusses. In addition, the company offers steel sheet piling and HZ Steel Wall systems.

For a free catalog, contact: TradeARBED, Inc., 825 Third St., New York, NY 10022.

J&M Turner
Booth #107

Direct Tension Indicators (DTIs) from J&M Turner are used to assure proper bolt tensions according to the RCSC Specifications for Structural Joints using ASTM A325 or A490 bolts. DTIs are now made under an improved production quality assurance program that requires lot testing at every stage of production. In addition, design improvements assure proper performance under field conditions. Also, each DTI is marked with a lot number to allow traceability back to the steel heat lot number.

For more information contact: J&M Turner, 101 Crofton Dr., Pittsburgh, PA 15238 (412) 967-9302.

Voss Engineering
Booth 121

Voss Engineering will feature PTFE expansion bearings at this year's NSCC show. The bearings utilize Fiberlast and Sorbtex elastomeric pads as support for the PTFE element. These elastomeric materials are designed to handle non-uniform loading conditions and vibration control. The company also will display Neosorb AASHTO-grade Neoprene pads and slide bearings made with glass-filled PTFE. The company also will sponsor a seminar on the basic principles of elastomeric bearing pads and PTFE expansion bearings.

For more information contact: Voss Engineering, Inc., 6965 North Hamlin Ave., Lincolnwood, IL 60645-2598 (708) 673-8900; FAX (708) 673-1408.

Welded Tube Co.
Booth #815/817/819/821

Welded Tube Co. of America is the largest domestic manufacturer of quality welded structural and mechanical tube and pipe. Produced in sizes 1" square through 16" square up to 5⁄8" gauge. The company also produces KleenKote, a cleaned and coated tube or pipe. KleenKote is produced from an in-line process where Welded Tube Co. mechanically cleans, degreases, and pre-primer coats the tubing or pipe during manufacturing. Some benefits of KleenKote Tubing are reduced cleaning and preparation, enhanced welding, and prolonged storage life.

For more information, contact: Welded Tube Co. of America, 1855 East 122nd St., Chicago, IL 80633 (800) 733-5683.

Westbrook Engineering Co.
Booth #105

In business since 1964, Westbrook Engineering Co. sells new and used beam and plate fabricating equipment. Lines include shears, brakes, saws, drills, rolls, angle lines, beam, punch and drill lines.

For more information contact: Westbrook Engineering Co., 23501 Mound Road, Warren, MI 48091 (313) 759-3100; FAX (313) 759-3106.

Yamazen Inc.
Booth #421/423/520/522

Daito Seiki Co. will introduce their line of CNC drilling machines to the U.S. market at this year's NSCC show. On exhibit will be their DNF 1000 CNC Structural Drilling Machine, which has the capacity of handling 40" wide by 16" high material. As with all of the company's drilling machines, it features a fixed workpiece, traveling drill design. Each drill head is independently programmed to move on three axes along the flanges or the web. This allows the flange drills to move down the beam and drill all the flange holes in the pattern without waiting for the web drilling to be completed, increasing both speed and accuracy.

For more information contact: Yamazen Inc., 735 East Remington Road, Schaumburg, IL 60173-5610 (708) 882-8800; FAX (708) 882-4270.

AISC 1993 Prize Bridge Competition

Rules

Eligibility
To be eligible, a bridge must be built of fabricated structural steel, must be located within the United States (defined as the 50 states, the District of Columbia, and all U.S. territories), and must have been completed and opened to traffic between *May 1, 1988* and *April 30, 1993*.

Judging Criteria
Judging will be based upon aesthetics, economics, design and engineering solutions. Quality of presentations, though not a criterion, is important.

Award Categories
Entries may be judged in one or more categories, but can receive only one award.

Long Span One or more spans more than 400 ft. in length.

Medium Span, High Clearance Vertical clearance of 35 ft. or more with longest span between 125 and 400 ft.

Medium Span, Low Clearance Vertical clearance less than 35 ft. with longest span between 125 and 400 ft.

Short Span No single span greater than 125 ft. in length.

Grade Separation Basic purpose is grade separation.

Elevated Highway or Viaduct Five or more spans, crossing one or more traffic lanes.

Movable Span Having a movable span.

Railroad Principal purpose of carrying a railroad, may be combination, but non-movable.

Special Purpose Bridge not identifiable in one of the above categories, including pedestrian, pipeline and airplane.

Reconstructed Having undergone major rebuilding.

Entry Requirements
All entries must contain an entry form, photographs and a written description of the project. A separate binder must be submitted for each entry. No entry free is required; submission materials will not be returned. The use of any entry's submitted data, detail and/or photographs by AISC shall be unrestricted. **Note:** Projects not receiving an award still may be used in *Modern Steel Construction* magazine or other AISC marketing materials.

1. *Entry form:* The complete and accurate entry form and one copy must be enclosed.

2. *Photographs:* A minimum of four professional quality 8x10 color prints of various views showing the entire bridge, including abutments as well as selected details, are required. 35 mm slides are strongly recommended. Photographs will not be returned.

3. *Description:* Explanation of design concept, problems and solutions, aesthetic studies, project economics and any unique or innovative aspect of the project. Include no larger than 11x17 drawings showing elevation, framing system and typical details.

Method of Presentation
Each entry should be submitted in an 8½" x 11" binder, containing transparent window sleeves for displaying inserts back to back. The entry form included in the brochure must be easily removable, so that the identification of the entry can be concealed during judging.

Awards
The winners will be notified shortly after the mid-August judging. Public announcements of the winners will be made in the November issue of *Modern Steel Construction* magazine. Award presentations will be made to the winning designers at the National Symposium on Steel Bridge Construction, November 11, 1993, in Atlanta, GA.

Deadline for Submission
Entries must be postmarked on or before ***June 18, 1993***, and addressed to: American Institute of Steel Construction, Inc., Attn: Awards Committee, One East Wacker Drive, Suite 3100, Chicago, IL 60601-2001. For further information, call 312/670-5432.

AISC 1993 Prize Bridge Competition

Entry Form

Entry Date__________

Name of Bridge__________ Completion Date__________

Location__________ Date opened to traffic__________

Category in which entered__________ Approx. total cost__________

Span lengths__________ Roadway widths__________ Steel wt./sq. ft. of deck__________

Vertical clearance__________ Steel tonnage__________ Painted: Yes__________ No__________

Structural system(s) (describe briefly here)__________

Innovative Concepts__________

Descriptive data: Attach separate sheets (see entry requirements)

No. of photographs enclosed: Color prints__________ 35 mm slides__________

Design Firm:__________ Phone

Address:__________ Street City and State Zip

Person to contact:__________ Title

Consulting Firm (if any):__________ Phone

Address:__________ Street City and State Zip

Person to contact:__________ Title

General Contracting Firm:__________ Phone

Address:__________ Street City and State Zip

Person to contact:__________ Title

Steel Fabricating Firm:__________ Phone

Address:__________ Street City and State Zip

Person to contact:__________ Title

Steel Erecting Firm:__________ Phone

Address:__________ Street City and State Zip

Person to contact:__________ Title

Owner:__________ Phone

Address:__________ Street City and State Zip

Person to contact:__________ Title

This entry submitted by:

Name:__________ Title

Firm:__________ Phone

Address:__________ Street City and State Zip

(ADDITIONAL ENTRIES MAY BE SUBMITTED ON COPIES OF THIS FORM)

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DoAll Vertical Band Saw TF24SA Tilt 24 x 24
Lamina Mag Drill 1½ CAP

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The only technical magazine in the United States devoted exclusively to the design of steel structures, the AISC Engineering Journal provides structural engineers, architects, fabricators and educators with the latest information on steel design, research, and construction.

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P.O. Box 660, Lake City, TN 37769
615-426-9634 FAX 615-426-6454

Ram Analysis Steel Design Software

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