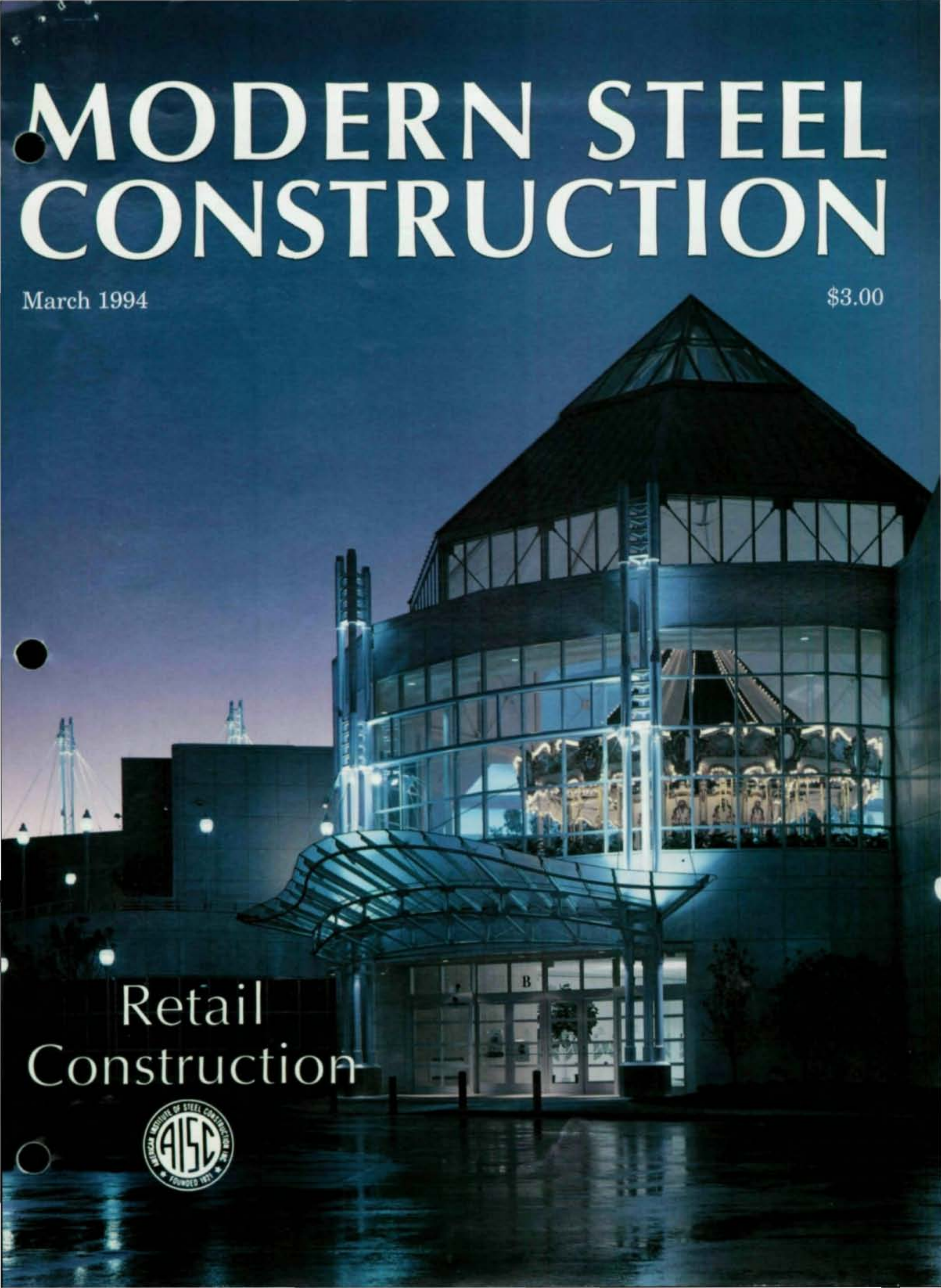
MODERN STEEL CONSTRUCTION

March 1994

$3.00



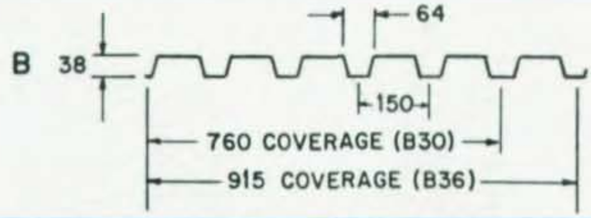
Retail Construction

DESIGN DECK DATA SHEET #20

UNITED STEEL DECK, INC.

METRIC IS COMING!

Soon it will be required that all federal jobs use international units. **UNITED STEEL DECK, INC.** will publish a complete metric deck catalog early in 1994. If you want a copy of this new publication send us your request and we will mail a copy as soon as it is available. In the meantime we hope you find this load table for B (wide rib) roof deck useful.



GAGE	THICKNESS mm	I mm^4/mm	S_p mm^3/mm	S_n mm^3/mm
22	0.749	232	10.2	10.7
20	0.909	301	13.4	14.0
18	1.204	423	18.3	19.4
16	1.519	546	23.7	24.[illegible]

TYPE B, BI, BA, BIA

Uniform Total Load (Dead + Live), kPa

Span Type	Gage	Span, mm 1500	1650	1800	1950	2100	2250	2400	2550	2700	2850
Single	22	*5.0*	*3.8*	*3.1*	*2.5*						
	20	*6.3*	*4.8*	*3.8*	*3.1*	*2.6*	*2.2*				
	18	*8.7*	*6.6*	*5.2*	*4.2*	*3.5*	*2.9*	*2.1*	*1.9*		
	16	*11.0*	*8.4*	*6.6*	*5.3*	*4.3*	*3.6*	*3.1*	*2.6*	*2.3*	*2.0*
Double	22	5.2	4.3	3.6	3.1	2.7					
	20	6.7	5.6	4.7	4.0	3.5	3.0	2.7	2.4		
	18	9.3	7.7	6.5	5.5	4.8	4.2	3.7	3.3	2.9	2.6
	16	11.6	9.6	8.1	6.9	6.0	5.2	4.6	4.1	3.6	3.3
Triple	22	6.4	5.3	4.5	3.8	3.3	2.9				
	20	8.3	6.9	5.8	5.0	4.3	*3.7*	*3.1*	*2.7*		
	18	11.5	9.6	8.1	6.9	6.0	5.0	*4.2*	*3.6*	*3.1*	*2.7*
	16	14.4	12.0	10.1	8.6	7.5	6.4	*5.3*	*4.5*	*3.9*	*3.4*

Loads shown in *italics* are controlled by L/240 deflection. Dead load is assumed to be 0.48 kPa.

NICHOLAS J. BOURAS, INC.
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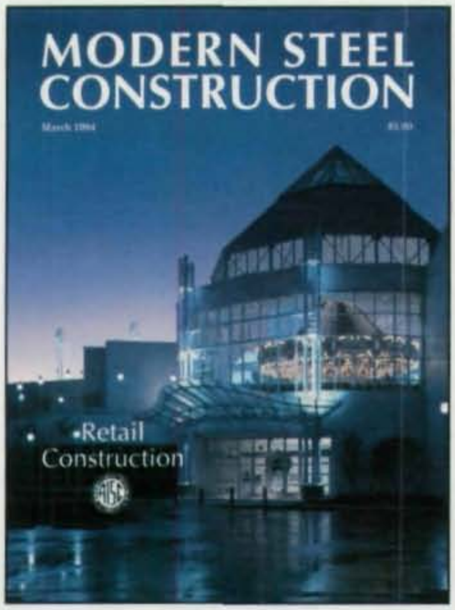
MODERN STEEL CONSTRUCTION

Volume 34, Number 3 March 1994



The architecture is a main attraction at the new North Point Mall including a large carousel near the main entrance. Photo by Ron Rizzo / Creative Sources.

FEATURES

DEPARTMENTS

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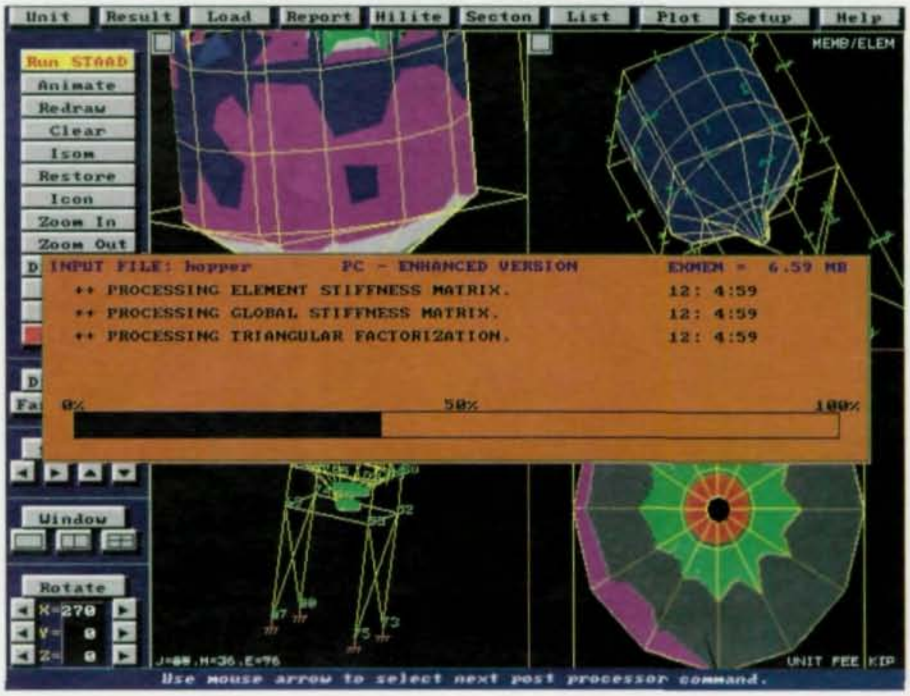

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Modern Steel Design

During the past few months, AISC has been buzzing with activity. Late last year featured the introduction of a new phone/fax service that allows design and construction professionals to quickly and easily request information on AISC publications and software (just call 800-644-2400). Next came the release of a Parametric Bay Studies computer program that provides relative cost information on more than 2,400 composite beam and girder designs on one IBM-compatible 3.5-inch diskette (to order a copy of the $50 program, call 312/670-2400). And this month marks the announcement of a new edition of the Manual of Steel Construction—Load and Resistance Factor Design.

When the first LRFD Specification was introduced in 1986, few designers took much notice—despite the fact that in many instances using LRFD can reduce steel costs by as much as 10% and despite the fact that, quite simply, LRFD is a more rational design method than ASD. Increasingly, however, designers are beginning to utilize LRFD. And more importantly, the engineering community continues to recognize that LRFD is the future of steel design. Today, all major structural engineering programs at U.S. colleges and Universities teach LRFD, and many all but ignore ASD.

Due to the larger amount of information provided in the new Manual, the Second Edition has been split into two volumes.

Volume I (structural members, specifications and codes) contains the shape and member design information that has been the hallmark of AISC manuals since 1927. The new Specification incorporates all of the changes garnered from seven years of practical experience working with LRFD including, of course, the 1993 AISC Specification for Structural Steel Buildings. In addition, there are two substantial improvements over the First Edition. "Essentials of LRFD" is an introduction to and simplified adaptation of the Specification. And, by popular demand, the Manual now includes Uniform Load Tables for LRFD.

Volume II (connections) contains—and more importantly, updates—all the design information previously presented in the "connections" portion of the First Edition, along with the information presented in several other AISC publications, including "LRFD of Simple Shear Connections" and "Manual of Steel Construction-Volume II Connections (ASD/LRFD)." The volume contains a wealth of information on simple shear connections, PR moment connections, FR moment connections, diagonal bracing connections, and beam bearing plates and column base plates.

Together, the two Volumes offer more than 2,000 pages of technical data. Each volume costs $72 ($54 for AISC members). The two-volume set sells for $132 ($99 for AISC members). An order form is printed on page 8 of this issue, or you can call (312) 670-2400 ext. 433.

Steel stands for the future. **SM**

Steel Interchange

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

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

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

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

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

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

Some Building Codes permit a reduction in NDT for welders with less than a 5% rejection rate. How big a sample is used to get the 5%?

The 1991 Uniform Building Code section 2711(k) states that all complete penetration groove welds shall be tested 100% by ultrasonic or radiography. However, with approval of the building official and the engineer responsible, the testing can be reduced to 25% of those welds. This reduction is based on a 5% or less reject rate for the welder. The reject rate is based on the test results of 40 or more welds.

Formula:
reject rate = (number of rejects) / (number of welds)

Example: Welder has completed 25 girders with top and bottom flanges groove welded to column. Welder has 2 rejects.

reject rate = 2 rejects / 50 welds
= 4%

Reject rate is less than 5%, thereby permitting the testing of only $^1/_4$ of that individuals welds. Three rejects out of 50 welds would have required the 100% testing to continue until the ratio improved.

Adrian L. Sherrill
Twining Labs
Long Beach, CA

When was the Vierendeel truss first utilized, why was it named, and for what contributions to structural engineering was he/she recognized?

We have always been interested in the Vierendeel type truss but have not used them except to a very limited degree.

Before the advent of the personal computer, analysis involved too may time intensive hand calculations.

I remembered one of my old college textbooks by L.E. Grinter (*Theory of Modern Steel Structures, Volume II*) where he states:

"A. Vierendeel, formerly a professor at the University of Louvain, Belgium. Vierendeel or open-web trusses have been recently introduced into America..."

This was a 1937 Macmillan Company publication and the method of analysis proposed was far from accurate but served the purpose. Deflections could be considerably more than expected.

Lloyd W. Abbott, P.E.
Lloyd W. Abbott, Consulting Engineer
Tulsa, OK

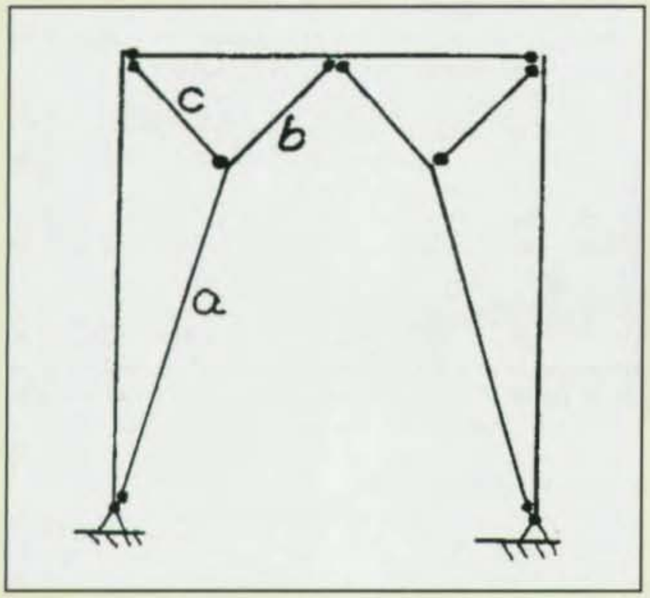
Can an existing steel beam and concrete slab be made to work together in composite action by adding studs to the steel through cored boles? are there any special considerations?

The answer to this question is an emphatic yes. One of the most economical methods to provide for the increased live load capacity of an existing steel beam, that supports a concrete floor system, is to attach headed studs to the top flange so that the slab is engaged for composite action of the entire cross section.

Typically, holes are cored along the centerline of the beam at a predetermined spacing. The holes must have a diameter sufficient enough to allow for the field installation of the studs and placement of the surrounding grout. Proper preparation of the hole prior to grouting and the use of a high-strength, non-shrink cementitious product to refill the hole is essential to the longterm serviceability of this type of strengthening program.

The analysis of this type of beam is no different than that used for new unshored composite construction. Care should be taken however, when calculating the initial stress levels of the bare steel section to include all existing superimposed dead loads that might be present within the contributory area of the beam. Depending on the size of the steel member, the existing framing conditions and the additional live load requirements, it is sometimes necessary to install a cover plate on the bottom flange of the beam in order to satisfy the requirements of the new loading criteria.

D. Matthew Stuart, P.E.
Mobile, AL



Due to some clearance requirement, a frame has the configuration shown. For out-of-plane buckling, what will be the unbraced lengths for members a, b, and c for different conditions?

The stability problem posed by the kinked brace can be simplified if the entire joint, including member c, is fixed. The out-of-plane bending stiffness can be analyzed as a plane tripod by distributing the out-of-plane load to three supports and calculating the deflection at the joint, providing the effective measure of stiffness. Note that by fixing the end connection of c, the reliance of torsion in a and b in resisting the out-of-plane load is eliminated, i.e. the torsion has largely become a bending moment in c.

John Vasko, P.E.
Foster Wheeler Energy Corp.
Clinton, N.J.

New Questions

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

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

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

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

James C. Peterson, P.E.
McLaren Peterson Associates, Inc.
Seattle, WA

Under the ASD design specification, how is the maximum unbraced length (L_c) of a structural tee beam to be determined if the tee stem is in compression? How is the allowable flexural stress to be calculated if the unbraced length exceeds this limit?

Paul DeArment, P.E.
Howard C. Dutzi & Associates, Inc.
Colorado Springs, CO

Serviceability is a particular concern for crane systems in industrial buildings but is not clearly covered in the standard code literature. What are deflection limits for crane runway systems?

LRFD Revisited

Steel design will be advanced to a new level this month when AISC introduces the 2nd Edition Load and Resistance Factor Design *Manual of Steel Construction*. The new Manual improves on the First Edition, both in content and organization (see ad on page 8 for ordering information).

New Specification

The 1993 AISC Specification for Structural Steel Buildings is the latest and most complete structural steel design specification ever produced by AISC, according to Charles Carter, AISC staff engineer-structures. "Key chapters have been augmented to give more in-depth and broader coverage of the design strength of structural steel members and connections," he stated. "You will find this revision to be a complete and comprehensive resource; not just an update—a thorough improvement."

Major changes include:

Alternative fillet weld design strength procedures now account for the increased strength of transversely loaded fillet welds, resulting in an increase in design strength of up to 50%.

Reorganization and expansion of Chapters F (Beams and Other Flexural Members) and K (Concentrated Forces, Ponding, and Fatigue).

The web crippling design provisions have been updated to reflect the latest research.

Recommendations for the use of heavy rolled shapes and welded members made up of thick plates previously released as a supplement to the Specification have been incorporated into this revision.

Specification provisions for slender web girders and unsymmetrical members have been updated and improved.

A new, more general and accurate C_b equation for beam design has been incorporated.

Provisions for the design of slip-critical joints at factored loads have been added.

Information on the stability of unbraced frames has been reorganized and expanded.

Specification provisions for built-up compression members have been revised and improved.

Easy Reference

The newest versions of several specifications and codes traditionally found in the Manual are included:

The 1992 AISC *Code of Standard Practice for Steel Buildings and Bridges*

The 1988 LRFD *Specification for Structural Joints Using ASTM A325 or A490 Bolts*

The 1992 AISC *Seismic Provisions for Structural Steel Buildings*

Additionally, a new specification, the 1993 AISC *Specification for LRFD of Single-Angle Members*, makes its debut in this Manual.

For ease of use, the almost 2,000-page Manual has been produced as a two-volume set. Volume I contains all the shape information, member design information, and specifications and codes with which the steel industry has become familiar throughout the more-than-70-year history of AISC. Volume II contains all connection design information. Both volumes are thumb-indexed for handy reference.

Integration Of Resources

Volume II-Connections seamlessly integrates and updates several formerly separate AISC publications on connection design: the "Connections" part of previous Manuals, LRFD of Simple Shear Connections, and the *Manual of Steel Construction, Volume II—Connections*, ASD 9th Ed./LRFD 1st Ed., and Engineering for Steel Construction-A Sourcebook on Connections.

Included in this complete and concise reference is information on: simple shear connections, partially restrained (PR) and fully restrained (FR) moment connections, shear and moment splices, diagonal bracing connections, beam bearing plates, and column base plates. It also includes numerous design aids and illustrative examples. Improvements include:

Improved and expanded design aids for shear end-plate and single-plate connections as well as a new, more rational approach to tee connection design.

New tables for eccentrically loaded bolt and weld groups with values tabulated at load angles of 0°, 15°, 30°, 45°, 60°, and 75°; additionally, the eccentrically loaded weld group tables have been updated to incorporate the new alternative fillet weld design strength provision resulting in typical strength increases of 10% to 30%.

New design aids for bolted connections include slip-critical joint design strengths for comparison with factored loads.

Improved block shear rupture design aids.

Expanded coverage of partially restrained (PR) moment connections.

Expanded, more complete, and more logical treatment of fully restrained (FR) moment connections as well as transverse stiffener and doubler plate design.

Further refinement of treatment of diagonal bracing connections and the Uniform Force Method.

Current Shapes

The most current design aids,

CONTINUED ON PAGE 14


Creating a masterpiece requires more talent than just welding commodity products!
At Canam Steel Corporation, everything is possible, even carving a masterpiece. This commitment to supply joists, joist girders and specialty products to projects like B.M.W. Corporation or Gulfport Grand Casino requires more talent than just welding commodity products. Our engineering team can propose different solutions to reduce your erection costs.
By acting as a partner or by performing subcontract work, Canam Steel Corporation can assist you at any stage of a project. We have the capability and experience to do everything to guarantee you complete satisfaction and on-time delivery. That's why Canam Steel Corporation is the Fabricators' fabricator.
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dimensions, properties and shape availability have been incorporated into the new Manual. All the traditional design aids (and the many new ones) have been updated to reflect today's shape series with nominal member depths through 44".

User Friendly

A new section, Part 2-Essentials of LRFD, serves as an introduction to and simplified adaptation of the 1993 AISC Specification for Structural Steel Buildings. The addition of this information makes it easier than ever before for experienced engineers who are extensively familiar with previous AISC Manuals and Specifications to elevate themselves to the next level of excellence in structural steel design. Also, for those engineers unfamiliar with LRFD or steel design in general, this section serves as a thorough and understandable primer on LRFD.

Uniform Load Tables

In response to numerous requests, for the first time, Uniform Load Tables are included with the LRFD Manual.

Steel Resource Directory

Included in Volume II is a listing of nearly 90 private and non-government related organizations, federal and state government and related agencies, and foreign organizations. Many organizations have provided statements which identify their purpose, services, and products to speed your search for answers. Addresses, telephone numbers and fax numbers are listed for all organizations.

Why LRFD?

LRFD directly accounts for the variability of both the strength of the steel structure and the loads on the steel structure. Thus, LRFD results in a nearly uniform factor of safety in every structure. ASD cannot account for loading variability and uniform structural reliability is therefore an impossibility with the older ASD procedures. Therefore, LRFD provides for uniform reliability, economy and a logical, straightfoward design.

STEEL CALENDAR

An introduction to the new 1993 LRFD Specification will highlight a new four-part seminar series from AISC Marketing, Inc. *Innovative Practices In Structural Steel* also will provide information on state-of-the-art structural steel design software, the latest NEHRP Seismic Regulations, and a review of semi-rigid composite connections.

The seven-hour, four-part seminar costs $90 ($75 for AISC members), including dinner. The lecture has a CEU value of 0.4.For more information, call 312/670-2400.

1994 Seminar Dates & Locations

WEST	
San Diego	3/17
Irvine	4/21
Sacramento	6/15
San Francisco	6/16
Los Angeles	6/23
Seattle	9/27
Salt Lake City	9/29
Phoenix	10/20
Portland, OR	11/15
Las Vegas	11/17

MIDWEST	
Chicago	3/8
St. Louis	3/24
Des Moines	5/4
Milwaukee	5/31
Minneapolis	6/2
Detroit	10/11
Indianapolis	10/13

NORTHEAST	
New York	4/14
Albany	9/13
Rochester	9/14

SOUTHWEST	
Kansas City	3/10
San Antonio	3/15
Dallas	4/19
Houston	6/21
Oklahoma City	9/8

SOUTH	
Greenville	3/1
Charlotte	3/2
Raleigh	3/3
Birmingham	5/3
Atlanta	9/20
Richmond	9/22
Memphis	10/18
Miami	11/1
Orlando	11/3

MID-ATLANTIC	
Baltimore	4/5
Washington, DC	4/7
Pittsburgh	4/12
Edison, NJ	10/4
Philadelphia	10/6
Cleveland	10/25
Columbus	10/26
Cincinnati	10/27

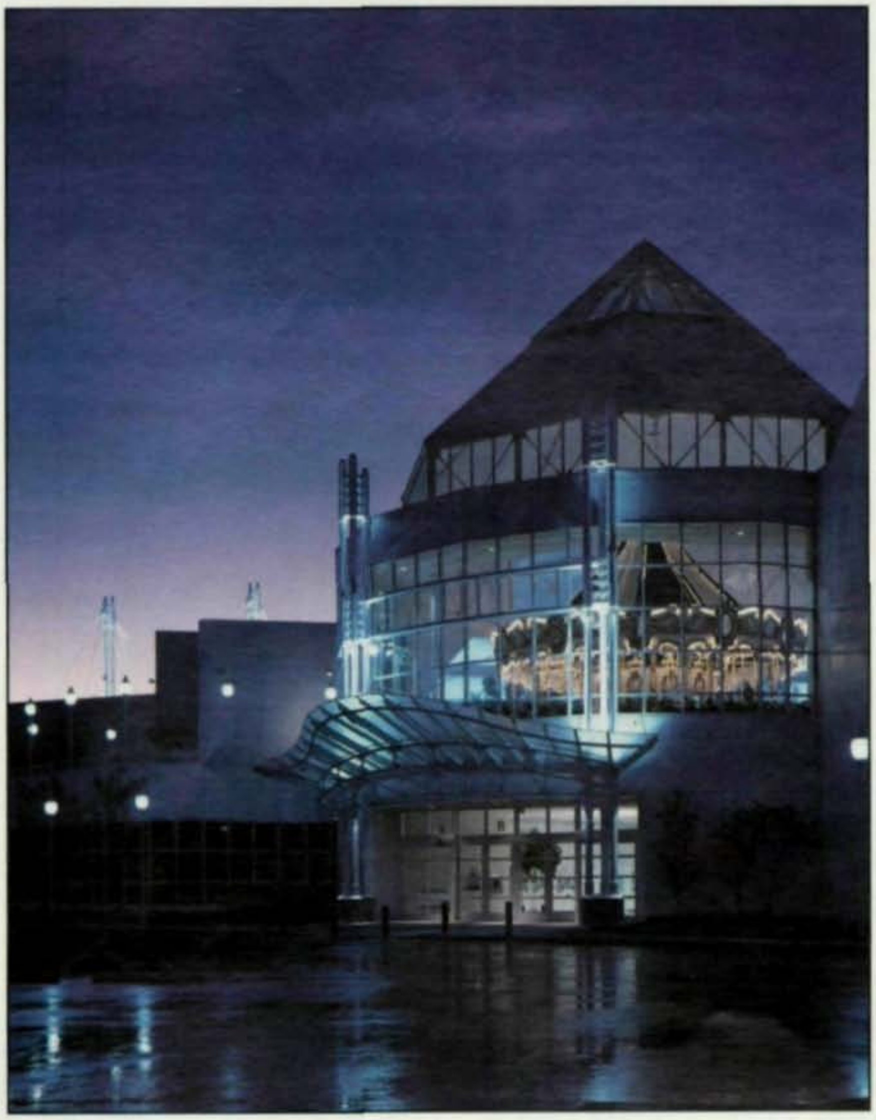
Other upcoming events related to the steel industry include:

• March 15 in New York City—Presentation on **"Research and Practice in Semi-Rigid, Composite Connections."** Sponsored by Steel Institute of New York and AISC. For more information, call: 212/697-5553.

• March 15 in Boston—**T.R. Higgins lecture on partially-restrained connections**. For more information, call: Irma Gabra at 617/951-6464.

•May 18-20 in Pittsburgh—**National Steel Construction Conference**. More than 20 seminars, technical sessions and workshops plus a 100-booth exhibition of steel-related products. For more information, contact: AISC at 312/670-2400.

Selling Architecture



The developers of a mall in suburban Atlanta are counting on the center's stylish design to help entice customers

Photos opposite and top by Ron Rizzo / Creative Sources; photos above and above left courtesy of ELS / Elbasani & Logan Architects.

To experience the full impact of the wondrous design of the North Point Mall in Alpharetta, GA, go there after the sun has set. The first thing to catch your eye as you approach the shopping center 20 miles north of Atlanta is a series of tall masts, each artfully lit, rising from the center of the structure. Next, your eye travels to the glowing glass pyramids spaced about the top of the mall, and then on to light emerging from the angled skylights running along the top of the center's spine. And finally, as you near the main entrance to the center, is a large glass wall revealing a spinning carousel filled with happy children and perhaps some equally happy parents.

"We wanted to create a dramatic roofscape to attract attention," explained Edward Noland, project designer with ELS/Elbasani & Logan Architects, Berkeley, CA. "As you approach the center, you can see that there's activity inside."

ELS is a 55-person firm founded in 1967. They're best known for their retail work (ranging from the fabulous Grand Avenue Mall in Milwaukee to a current project in Singapore) and many of their projects are noteworthy for the designer's attention to detail. North Point Mall is no exception.

The precise design begins with the curved glass canopies above the mall's entrances. "They were designed by Jeffrey Zieba (a project designer at ELS) to be reminiscent of a Paris metro stop," according to Noland. "They not only mark the entrance to the building but also serve as a rain shelter for people waiting for a bus or to be picked up." The structure under the entirely glass-covered canopies is framed with radiating sloping trusses. Truss top chords are 4-in.-diameter pipe and bottom chords are 3-in.-diameter pipe. All web members consist of $^5/_8$-in.- and 1-in.-diameter rods. The entry canopies are hung by $^3/_4$-in.-diameter rods from free standing "column clusters" made up of structural tubes.

The attention to details continue with the design of the center's configuration. "Part of the basic layout was dictated by what the developer (Homart Development Co., Chicago) likes," Noland said. "There's a tremendous concern for sightlines. You want to be able to see as many storefronts as possible, but you don't want just a straight corridor. That would be overwhelming since the mall is six blocks long." ELS' solution was to slightly skew the main corridors around a series of interior courts, each topped with a glass pyramid. "As you begin to turn a corner, you still see a continuous progression of shops," he said.

The mall has two levels, and several techniques were used to entice customers to the second level. The primary technique was to slope the parking lot, so that two of the mall's entrances are on the main floor, while the other two are on the upper floor. "We also tried to make all of the vertical transportation elements interesting—something that people would want to ride on," Noland said. For example, the elevators have glass walls on three sides and a glass roof. The structure holding up the rails is made of tube sections and a pulley and counterweights were installed to create a moveable sculpture. The sculptural aspect was further enhanced by hiring an artist to design different shaped counterweights for each elevator. In addition, the interior workings of the escalators were left visible. To further entice customers to the second level, the food court was located there.

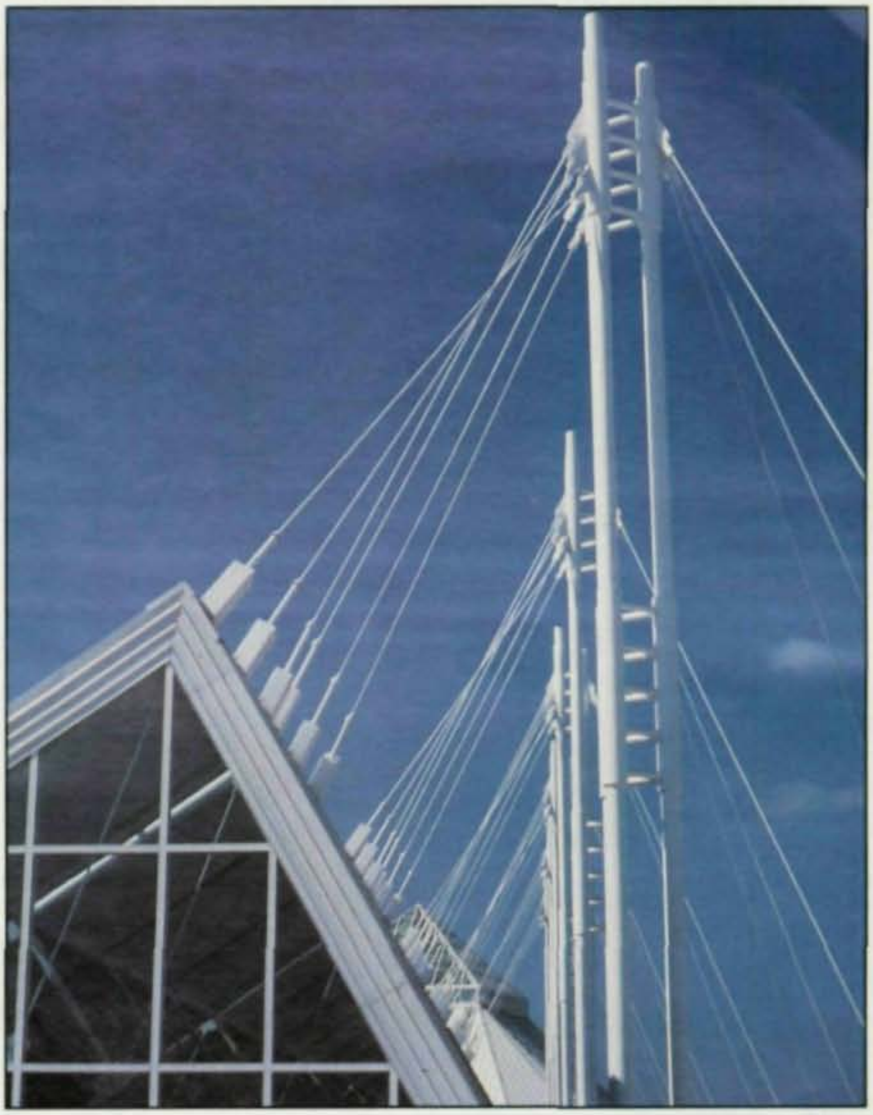
Exposed Structure

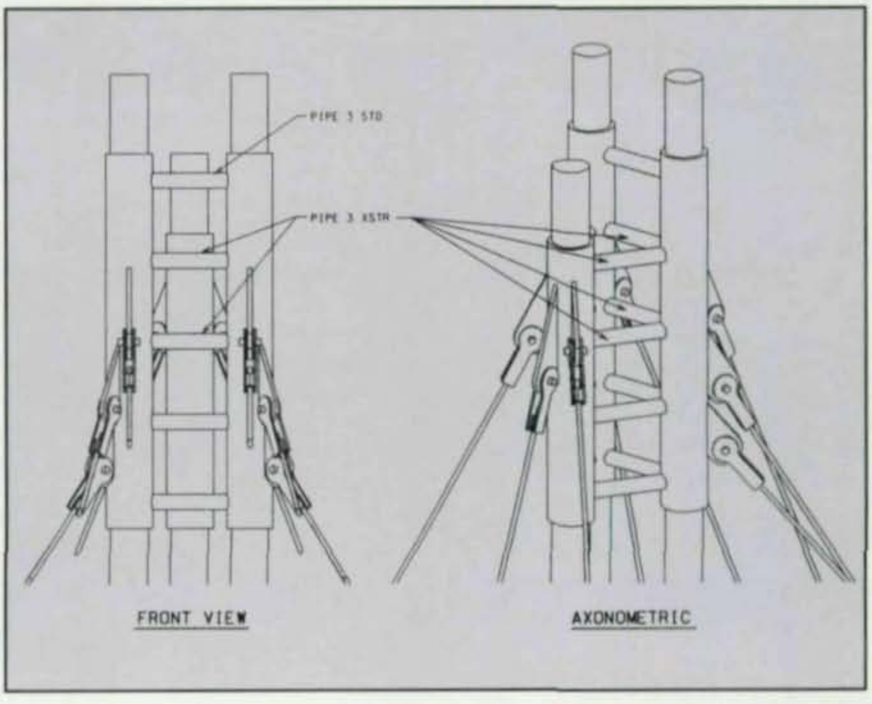
The inside of the structure is as visually exciting as the outside, in part because the structure was left exposed, but also because the glass clerestories afford a close-up view of the tall masts that attract so much attention from the outside.

The masts are 10-in. diameter tubes supported by high-strength Dywidag threadbare cables, according to Anne Piazza, P.E., a principal with L.A. Fuess Partners Engineers, Dallas. "The cables splay diagonally in both directions from the top of column clusters that reach 83-ft. in height at the tip and are 45-ft on center. Horizontal forces imposed by the cables at the top of the column clusters are resisted by cables that fan out to the flat roof structure. Custom made clevises were fabricated to connect the cables to the column clusters."

Exposing the structure was part of the designer's plan to at once relate back to the laciness of 19th century and to look ahead to the 1996 Olympics in Atlanta, according to Barry Elbasani, FAIA, a principal with ELS. "The design is in the spirit of what Atlanta will become and we wanted to contrast the old with the new. And we decided to use the structure to present the message."

Added Noland: "Instead of covering up the structure, we celebrated it. We wanted to keep the structure light in appearance and to create a romantic feel. We wanted to create a lacy, 19th

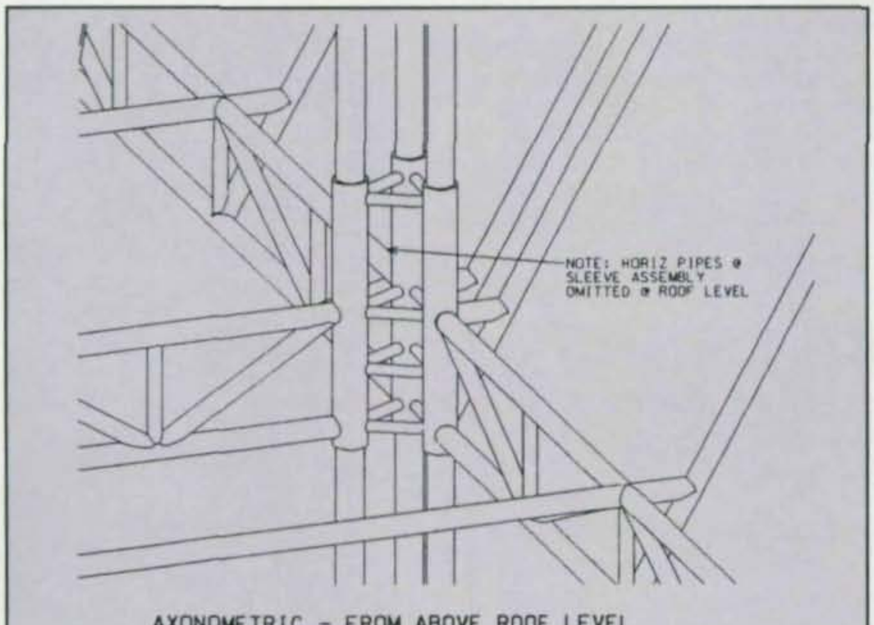




century-looking roof system, like a 19th century railroad station. But we wanted to do it with modern materials."

To capture that look, the designer used a king post truss system. "The typical arcade section consists of a steeply pitched asymmetrical gabled roof," Piazza said. "The southern side of the ridge is framed with double king post trusses at 15-ft. on center that support metal studs and roof deck." The X-bridging between the trusses consists of $^3/_4$-in. diameter rods and matches the trusses bottom chord and web members giving the otherwise planar trusses a three dimensional component. The northern side of the ridge is framed with 6-in. diameter pipe at 15-ft. on center that supports a continuous skylight.

Spaced along the arcade are a series of courts, the largest of which is the 100-ft. square center court, which is topped by a stepped pyramid that reaches a height of 95-ft. above the ground floor and supported on twelve "column clusters" that consist of three 8-in.-diameter pipes intermittently connected by 3-in.-diameter pipes. "The structure is comprised of an intricate combination of pipe truss tension and compression rings, with king post trusses on the sloped sur-



AXONOMETRIC - FROM ABOVE ROOF LEVEL

faces," Piazza said. ""The top portion of the pyramid has skylights, but the majority of the roof surface is metal decking." Because the structure is completely exposed on the inside, 3-in. metal roof deck was used to allow wide truss spacing, and pipe sections were used as top chord members for uniformity of design throughout the facility. The 3D analysis and design was accomplished using Intergraph's Micasplus software package.

"To make the structure seem as lightweight as possible, we rotated the pyramid 45 degrees, which resulted in the four triangular corners being left open," Noland explained. "We put clerestory windows in those areas so that when the sun shines through there is an illusion created of no roof." The light, almost transparent appearance was enhanced by the use of small diameter rods for all tension members and small pipe sections for the relatively few members in compression. Rod members are connected with clevises at each end. Two smaller but similar court roof structures are located near each end of the mall.

The center of the second floor arcade is left open to expose a view of both the upper level shopping and the roof skylights to shoppers on the first floor. Eight bridges span the 30-ft. between the north and south sides of the upper level, with the bridges' design reflecting the design of the arcade roof. "Double king-post trusses on each side of the bridges keep the profile light with 6-in.-deep wide flange top chords," Piazza said.

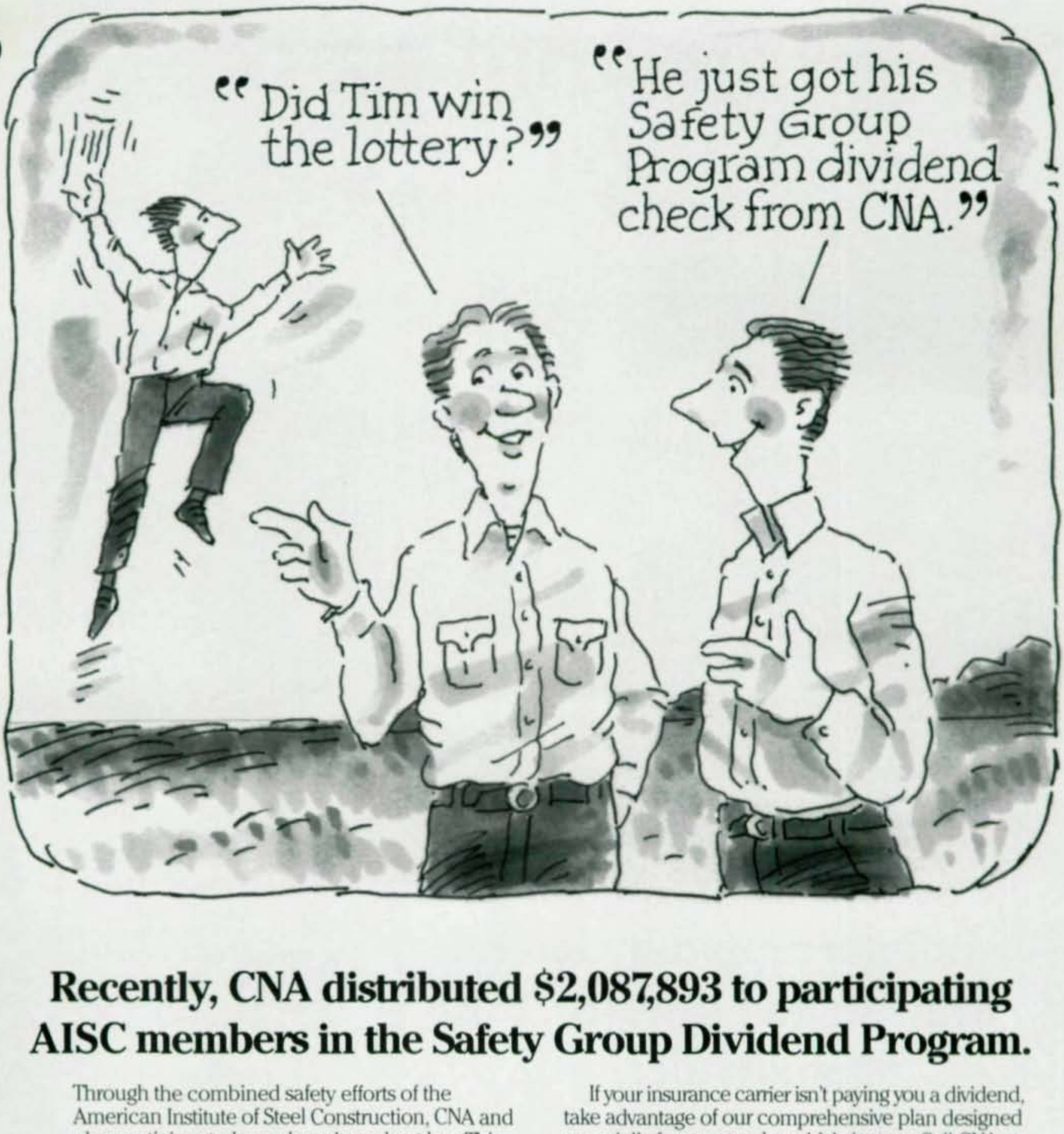
Lateral Loads

The structural design criteria for Alpharetta include zone 2 earthquake and 80 mph wind loads. Lateral bracing consists primarily of diagonal and chevron pipe braces located in perimeter and tenant separation walls.

Upper level tenant areas are framed with conventional lightweight concrete slab on steel formwork, supported by steel bar joists, joist girders and wide flange girders. The flat roofs over the tenant areas are framed with bar joists and joist girders with 22 gauge roof deck. Typical bays were 30-ft. by 30-ft.

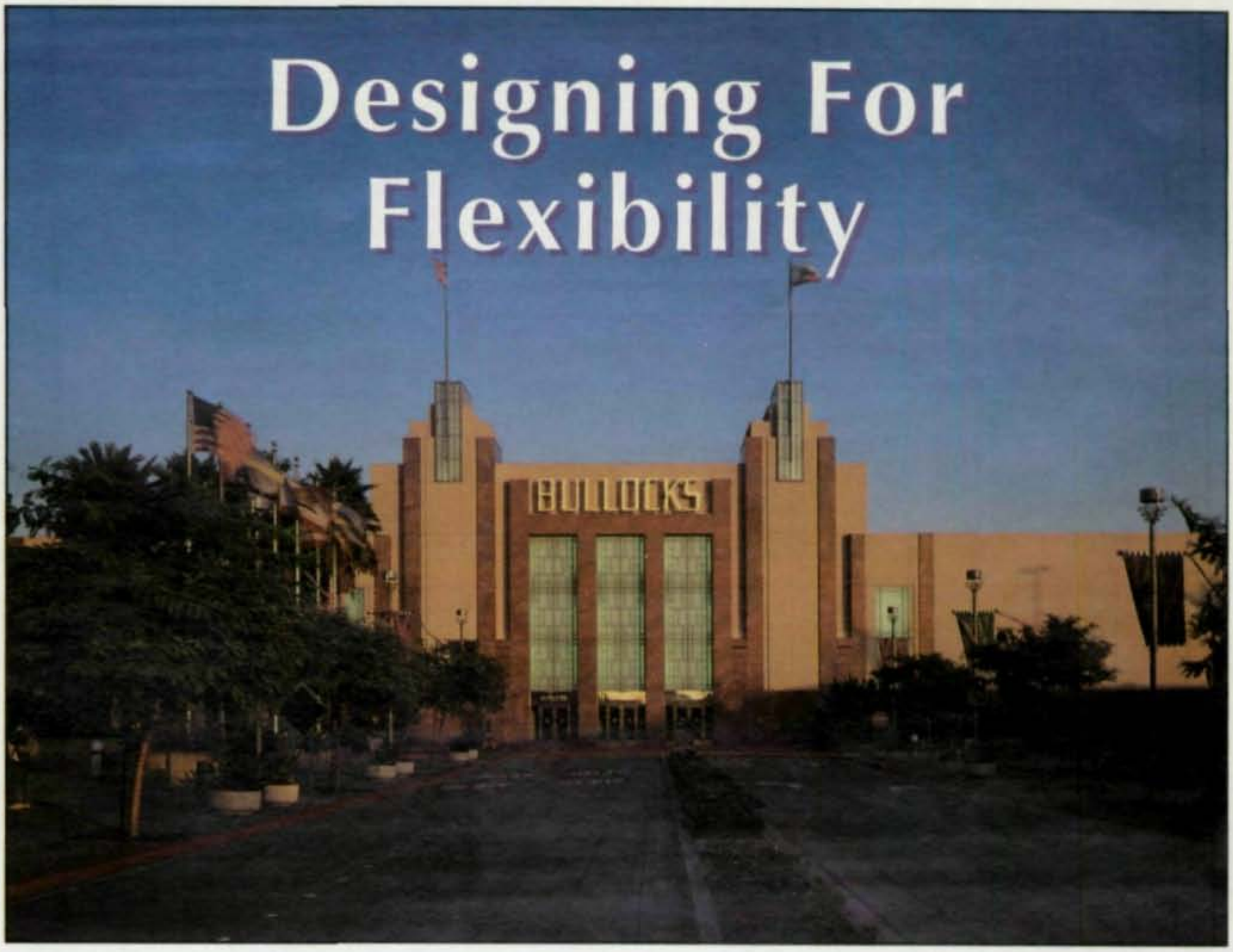
General contractor on the project was Hardin Construction Group, Atlanta. Structural steel fabricators on the project were AISC-members Universal Steel, Inc. and Steel Service Corporation. Erectors were Habersham Erectors and Dixie Erectors. The shopping mall includes 600,000 sq. ft. of gross building area on two levels, plus five department store anchors, bringing the total to approximately 1,400,000 sq. ft.

The finishing touch on the mall is undoubtedly the carousel located in the center court. "Shopping center developers are starting to include entertainment as a draw," Noland explained. "If the architecture of the center is exciting, it sticks in people's minds and makes them want to come back."



"Did Tim win the lottery?"
"He just got his Safety Group Program dividend check from CNA."

Designing For Flexibility



Moment connections provided both open spaces uninterrupted by lateral bracing and the structural stability desired in a seismic zone

By Mario LaGuardia, AIA, and Nabih Youssef, S.E.

The challenge of designing a seismic structure for a large department store is to come up with a flexible framing configuration that is conducive to the ever-changing needs of retail planning and interior design.

Paramount in retailing is dramatic open interior spaces that accommodate exciting visual display and merchandising. However, bracing systems typically used for structures in seismic areas impede the opportunity for customers to perceive open space and therefore may disrupt the balance between visual display and structurally sound design.

Framing and seismic requirements directly impacted the overall design of the

288,000-sq.-ft. Bullock's department store in Burbank, CA, the first Bullock's to be designed and built under R.H. Macy & Co.'s corporate flag. The structure is steel framed throughout with composite steel beams and girders used for gravity framing and a lateral system consisting of ductile moment resisting frames.

To meet the construction budget and time parameters set forth under Macy guidelines, it was imperative to specify the framing requirements early in the design process. Much of the overall architecture and interior design planning rested on this critical decision. It's specification would direct the design development, as well as help to control costs and lead-time for anticipated pre-purchased items, the bulk of which is interiors related.

Central to Macy's design brief was the inclusion of their escalator core and skylight, a trademark in their mall anchor department stores. The building's structure would have to accommodate this central interior theme.

The traditional framing systems for similar retail structures typically uses diagonal chevron bracing. Since it provides an economical and easily designed framing system. Although chevron systems can reliably protect life safety, they have been known to buckle during major earthquakes. Their greater stiffness magnifies building accelerations, which can result in increased damage to non-structural building components, such as exterior masonry walls and cladding, ceilings and partitions. The potential to business disruptions caused by damage to store contents, including merchandise, displays and goods stacked or shelved in stockrooms certainly had to be considered in the design of this structure.

Moment Frame Provides Maximum Flexibility

Principal structural engineers, Nabih Youssef & Associates, proposed using steel

(Finished photos, opposite and following pages, by Steve McClelland. Construction photography courtesy of Brennan Beer Gorman Architects.)

moment frames as the primary seismic load resisting system, making a departure from the typical lateral design of large, multi-story retail stores normally found in Southern California.

A moment frame system's greater flexibility helps control the seismic accelerations to which the building is subjected. Furthermore, the moment frame has a more stable behavior in the post-elastic range and therefore does not experience degrading behavior due to brace buckling, as in stiffer systems. In addition, in a store with a footprint as large as Bullock's (254-ft. by 354-ft., with floor spacing of $17^1/_2$-ft. for the first two floors and 20-ft. between the third floor and roof) lateral resisting elements in the interior of the store were called for, and chevrons would have presented formidable space planning constraints. Conversely, moment frames presented almost no space planning problems, other than a slightly larger than normal column and beam size.

Careful optimization of the design, taking full advantage of state-of-the-art structural engineering software, resulted in a structural design less costly than a comparable traditionally designed building. Both static and dynamic models of the store's lateral system were developed using in-house software for preliminary two-dimensional models and more advanced three-dimensional models were created using ETABS from Computers & Structures, Inc. In addition, much of the detailed design of the lateral system also was accomplished by in-house computer programs. Finally, the system, using only conventional horizontal and vertical framing

members, sped erection, since cumbersome diagonal members were eliminated, a plus in a project of this size.

Sophisticated software also was utilized to provide maximum economy in the design of the floor framing system. Floor vibration concerns were frequently the governing criteria for typical floor area member's design, but keeping the design consistent with the needs of a retail occupancy yielded significant cost savings. Furthermore, since this project had very specific construction schedule and design goals, structural design preceded space planning, so the exact location of high load occupancies, such as stockrooms, was not completely settled.

To avoid penalizing the building's structural design, full use of Load and Resistance Factor Design (LRFD) provisions was made, resulting in final floor member sizes being nearly the same in potential stockroom locations and typical floor space. The RAMSTEEL program from Ram Analysis was used extensively in optimizing the floor framing design, since checking routines for various code provisions, including LRFD, are built into the program. In addition, the program offers a vibration analysis feature.

These procedures for optimizing the design of the gravity and lateral systems led to an economical steel weight of 8.7 psf, compared to an average of 9.5 to 10 psf for similar projects. Because of mechanical ductwork restrictions, most of the girders were limited to W18 sections. However, at the interior moment connections, W24x131 columns and W27x94 girders were specified. Perimeter columns were W30 sections and typical gravity columns were W8x58 and W10x54. Fabricator was AISC-member Riverside Steel.

Project Coordination

Bullock's was designed as the third anchor tenant of Media City Center, the retail component of a major downtown redevelopment program developed by The Alexander Haagen Co., Inc., in partnership with the City of Burbank Community Redevelopment Agency.

Surrounded on two sides, it is adjacent to the mall and a parking garage. Careful coordination between the other building consultants was necessary to minimize building separations, while remaining compliant with seismic requirements for building separations. General contractor on the project was HCB Contractors.

As the structural design of Bullock's was beginning, construction documents for the adjacent parking garage were under production. To reduce costs, it was decided to share a foundation between the two buildings.

One of the challenges on the job was to develop a method to allow stiff elements, such as the exterior cladding, to accommodate the building deformation potential of a relatively flexible moment frame system.

According to Christopher Richter, S.E., NY&A project structural engineer, "the exterior cladding, framed with a combination of steel studs and reinforced masonry, was anchored with out-of-plane support while isolating it from the in-plane lateral movements of the structure."

Architectural Considerations

Brennan Beer Gorman/ Architects' design partner, David Beer, saw the design of Bullock's evolve as a tribute to Hollywood's most romantic and elaborate stage sets of the 1920s and 30s. Echoing the monumental nature of that era, the Art Deco facade provides a sense of playfulness in its use of colors, textures and light.

The symmetrically designed and massive 375-ft.-wide facade is offset by four architecturally expressed steel frame towers rising to a maximum of 16-ft. above the roof. Accented by multi-tiered lighting piers inset into vertical reveals, Bullock's presents a striking nighttime image. The structure is clad in polished brown and rose granite and stucco, accented by vertical metallic and glass grills.

The main exterior lighting element is indirect recessed lighting behind the tinted caladon green glass and bronze decorative grilles, back-lit for greater drama.

Macy's observed that Bullock's proximity to Hollywood would create a ready-made market of television costume designers, production stylists and studio

executives. Therefore, the interior design had to be as dramatic as the exterior. Macy's required an open design that would permit a panoramic view to all departments without visual obstruction.

Wide open spaces, abundant natural light emanating from a soaring octagonal skylight above the escalator core (a Macy's trademark design statement), complemented by custom-designed interior lighting fixtures, interplay with the store's visual design and merchandising approach, enhancing the shopping experience of its customers.

The moment frame provided the perfect solution to Macy's interior design criteria, allowing for the huge interiors and clear sightlines throughout. Brennan Beer Gorman/Architects was able to standardize interior modules to the 28-ft. bays situated around the alternating column sizes. These are the most efficient bay sizes in retail and work well within a reasonable module for floor-to-floor height, as well as effective merchandising display.

Mario LaGuardia, AIA, is a partner with Brennan Beer Gorman/Architects in New York and Nabih Youssef, S.E., is president of Nabih Youssef & Associates in Los Angeles.

1994 NATIONAL STEEL CONSTRUCTION CONFERENCE
DAVID L. LAWRENCE CONVENTION CENTER
PITTSBURGH, PA • MAY 18-20

REGISTRATION FORM

REGISTRATION FEE: (please circle appropriate fees)

AISC Member Fee:	**$290.00**
(Includes AISC Active, Associate & Professional Members)	
Non-Member Fee:	**$340.00**
Educator Fee:	**$100.00**
(Employed full-time at accredited architectural or engineering college or university.)	
Student Fee:	**$ 75.00**
(Letter from faculty advisor or equivalent required.)	
Exhibitor, in booth:	**No charge**
Added Exhibitor:	**$85.00**
Spouses' Fee:	**$100.00***

*Includes: Wed. evening reception; exhibit floor pass; 1 spouse tour each day—Thurs., Fri.

PARTIAL REGISTRATION FEES

(You also may pre-register for one day or half-day.)

Circle your choice below:

Half Day Sessions: (Lunch **not** included)

Wednesday afternoon:	$ 55.00
Thursday morning:	$ 70.00
Thursday afternoon:	$ 70.00
Friday morning:	$ 70.00

One Day Sessions:

Thursday (includes lunch)	$160.00
Friday (includes lunch)	$160.00
Exhibitor Floor Pass:	$ 5.00

(included in full, partial, and spouse registrations)

TOTAL OF PARTIAL REGISTRATION FEES: ______

REGISTRATION FEES INCLUDE: General Sessions; workshops; seminars; exhibition; coffee breaks; luncheons Thursday and Friday; Cocktail Reception Wednesday evening; and a printed, bound copy of the NSCC Proceedings. Exhibitors are entitled to one registration for each 10 ft. x 10 ft. exhibit space reserved. "Added Exhibitor" fee is payable ONLY if in excess of one person per 10 ft. x 10 ft. exhibit space.

REGISTRATION CANCELLATION POLICY: For cancellations received on or before May 5, 1994, 100% of pre-paid registration fees will be refunded; after May 5, 50% will be refunded. Those canceling after May 5 will receive a copy of the Conference Proceedings. Refunds will be sent after the NSCC.

REGISTRATION FOR OPTIONAL AND SPOUSE EVENTS

Event No.	Tickets	Total Price
OPTIONAL EVENTS		
#1—Conference Dinner (Thurs., 7:00 p.m.)	______	@ $39.00 $______
SPOUSE EVENTS		
#A—Pittsburgh's Treasures Tour (Thurs., 9:00 a.m.)	______	@ $45.00 $______
#B—Amish Country (Thurs., 9:00 a.m.)	______	@ $48.00 $______
#C—Fallingwater Luncheon Tour (Fri., 8:30 a.m.)	______	@ $44.00 $______
#D—A Shopper's Dream (Fri., 9:30 a.m.)	______	@ $32.00 $______
#E—AISC Cocktail Reception (Wed., 6:30 p.m.—included with spouse or regular registration)	______	@ $20.00 $______
#F—Exhibit Floor Pass per day (Wed., Thurs., Fri.—included with spouse or regular registration)	______	@ $ 5.00 $______

TOTAL OPTIONAL/SPOUSE EVENT FEE $______

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PRELIMINARY SCHEDULE OF EVENTS

MONDAY, MAY 16

7:00 a.m.- 5:00 p.m. Exhibitor move in continues until 1:00 p.m. Wednesday

TUESDAY, MAY 17

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

WEDNESDAY, MAY 18

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

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

8:30 a.m.- noon **ASCE Steel Buildings Committee**

8:30 a.m.- 1:00 p.m. **AISC Safety Task Force Committee**

1:00-1:15 p.m. **Welcome**
Frank B. Wylie III
AISC Chairman, Grace and Wylie Fabricators, Inc., Brentwood, TN

1:15-2:15 **General Session** (0.10 CEU)
Long Span Roof Structures
Moderator: TBA
Speaker: Dan Cuoco, Thornton Tomasetti, New York
Structures which have large open spaces need roofs that will span great distances. These long span roof structures are often dramatic architectural works as well as exciting engineering achievements. Dan Cuoco and Thornton Tomasetti have been involved in many impressive long span roof structures. Their experience will be presented along with some important structures.

2:15-3:00 **General Session** (0.075 CEU)
T. R. Higgins Lecture
Winner to be announced

3:00-5:30 **Exhibits Open**
5:45-6:30 **Exhibitor Workshops**

6:30-8:00 **AISC Welcome Reception**

THURSDAY, MAY 19

7:00-8:00 a.m. **Speaker Breakfast**
7:30-8:15 **Exhibitor Workshops**

8:30-10:00 **General Session** (0.15 CEU)
STEEL SURVIVES: World Trade Center Explosion
Moderator: Robert G. Abramson, Interstate Iron Works, Corp., Whitehouse, NJ
Speakers: Leslie E. Robertson, *Leslie E. Robertson Associates, New York*
Jack Daly, Karl Koch Erecting Co., Inc.
A detailed discussion by the structural engineer and steel contractor on the structural effects of the World Trade Center bombing. Highlights of Robertson's discussion will be the remakable performance of steel components of the building and technical recommendations on future design considerations. Highlights of Daly's presentation include the aftermath of the explosion and the reconstruction work to repair damage to the building. An extraordinary team effort was required through the close work of engineering in the clearing and reconstruction phase of this emergency work.

10:00-3:00 **Exhibits open**

10:00-10:45 **Coffee Break in Exhibit Hall**

10:45 a.m.- 12:15p.m. **Technical Sessions** (0.15 CEU each)
3. Lean Engineering
4. Electronic Data Transfer
5. Building Innovations
6. Quality Certification: Directions for the 90's
9. Research into Practice
16. Software Requirements

12:15-1:30 **Lunch in Exhibit Hall**

1:30-3:00 **Technical Sessions** (0.15 CEU each)
- **1.** 2nd Edition LRFD Manuals—Volumes I and II
- **7.** Steel Interchange - Connections
- **8.** The Total Team: Let's Get it Together
- **10.** TQM Personnel Issues
- **11.** International Developments
- **12.** Safety = Dollars

3:10-4:40 **Technical Sessions** (0.15 CEU each)
- **2.** Effective Use of High-Strength Steel in Building Construction
- **13.** Building Retrofit
- **14.** Marketing—Getting Your Share In Good Times and Bad
- **15.** Experience from Wind Damage & Design Load Requirements

4:00-10:00 p.m. **ASCE Committee on Structural Connections**

5:00-5:45 **Exhibitor Workshops**

7:00-11:30 **Conference Dinner** (optional)

FRIDAY, MAY 20

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

8:30-9:15 **General Session** (0.075 CEU)
Bridge Construction—Myths & Realities of Life Cycle Costs
Moderator: Franklin B. Davis, Precise Fabricating Corp., Georgetown, MA
Speaker: Robert Nickerson, NBE, Ltd., Hampstead, MD

9:00-1:00 **Exhibits open**

9:15-10:00 **Coffee Break in Exhibit Hall**

10:00-11:30 **Technical Sessions** (0.15 CEU each)
- **4R.** Electronic Data Transfer
- **6R.** Quality Certification: Directions for the 90's
- **7R.** Steel Interchange—Connections
- **9R.** Research into Practice
- **11R.** International Developments
- **14R.** Marketing—Getting Your Share In Good Times and Bad

11:30-1:00 p.m. **Lunch in Exhibit Hall**

1:00 p.m. **Exhibits close; exhibitor move out**

1:00-2:30 **Technical Sessions** (0.15 CEU each)
- **1R.** 2nd Edition LRFD Manuals—Volumes I and II
- **3R.** Lean Engineering
- **8R.** The Total Team: Let's Get it Together
- **12R.** Safety = Dollars
- **15R.** Experience from Wind Damage & Design Load Requirements
- **16R.** Software Requirements

2:40-4:10 **Technical Sessions** (0.15 CEU each)
- **2R.** Effective Use of High-Strength Steel in Building Construction
- **5R.** Building Innovations
- **10R.** TQM Personnel Issues
- **13R.** Building Retrofit

SATURDAY, MAY 21

7:00 a.m.-Noon **Continue exhibitor move out**

TECHNICAL SESSIONS

1 2nd Edition LRFD Manuals—Volumes I and II

Moderator: Bill Dyker, Garbe Iron Works, Inc.

Speakers:

Nestor Iwankiw, AISC

Charlie Carter, AISC

Since the release of the 1st Edition LRFD Manual and Specification, education, training, and actual engineering experience during the last seven years has stimulated an increased understanding and awareness of limit states design. A number of improvements have been made from such usage and new research. These changes have been included in a new 2nd Edition Manual of Steel Construction—LRFD and a 1993 LRFD Specification. These will be summarized and their implications explained.

Thursday at 1:30 p.m.
Friday at 1:00 p.m.

2 Effective Use of High-Strength Steel in Building Construction

Moderator: William Ashton, Egger Steel

Speakers:

Abe Rokach, AISC

Chia-Ming Uang, Northeastern University

As a design professional are you using the most economical grades of steel for building construction? New mill processes allow you to take advantage of higher yields without an increase in cost.

A proposed specification to assist engineers in selection of appropriate steels for structural shapes in buildings will be presented. AISC will show several design studies demonstrating cost benefits using high strength steel and considering deflection and vibration. Prof. Uang will offer a new design procedure of balancing frame strength and ductility requirements in seismic frames.

Thursday at 3:10 p.m.
Friday at 2:40 p.m.

3 Lean Engineering

Moderator: Bill Liddy, AISC Marketing, Inc.

Speakers:

Mark Holland, Paxton & Vierling Steel Co.

Engineer—TBA

The relationship between the engineer and the fabricator can make or break a project. Connection design is one of the most important parts of a structure. A good working relationship between the engineer and the fabricator can catch any connection problems before they surface and can solve any problems that do come up. The two speakers in this session will discuss how they work together as a team in building a steel building.

Thursday at 10:45 a.m.
Friday at 1:00 p.m.

4 Electronic Data Transfer: Now and Beyond

Moderator: John Bailey, Havens Steel Co.

Speakers:

Harry Moser, DuPont

Sayle Lewis, Fluor Daniel

Learn how this important new state of the art technology is revolutionizing the engineering and detailing industries. Gain insight into how to utilize this system to take advantage of the cost savings and cycle time reductions currently available. Actual project case histories and ideas for future applications will be presented for discussion.

Thursday at 10:45 a.m.
Friday at 10:00 a.m.

5 Building Innovations

Speakers:

Tom Sputo, Consulting Engineer

Neil Wexler, P.C.

Engineers and fabricators are coming up with many new innovations for the use of structural steel in buildings, much of this exciting new work can save money and time. Sputo will discuss how he uses conventional structural shapes to produce a cost efficient gable frame building. Also presented will be a practical use of semi-rigid beam-to-column connections. Much research on this topic has been discussed, now we can see how it is being used.

Thursday at 10:45 a.m.
Friday at 2:40 p.m.

6 Quality Certification: Directions for the 90's

Moderator: Tom Schlafly, AISC
Speakers:
Tom Schlafly, AISC
TBA
The AISC Certification Program is in the process of focusing on the demands of the '90s. The aim is to provide the premier certification program that is relied on by builders and owners from coast to coast.
Thursday at 10:45 a.m.
Friday at 10:00 a.m.

7 Steel Interchange—Connections

Moderator: Robert O. Disque, Besier Gibble Norden
Speakers:
Geoffrey Kulak, University of Alberta
Omer Blodgett, Lincoln Electric Co.
The popular *Modern Steel Construction* question and answer forum is here at the Steel Construction Conference. Two connection experts will discuss and answer questions that the audience may have. The session will be moderated by Robert Disque and will include *Geoffrey Kulak on bolting and Omer Blodgett on* welding. These speakers will be able to help with all the design, fabrication and erection problems just like the Steel Interchange column.
Thursday at 1:30 p.m.
Friday at 10:00 a.m.

8 The Total Team: Let's Get it Together

Moderator: Jerry Milligan, Falcon Steel Co.
Speakers:
Bill Treharne, Broad, Vogt & Conant, Inc.
Eric Waterman, National Erectors Association
Bill Lindley, W & W Steel
Erectors need to advise fabricators and engineers of techniques that have and will help minimize the erector's costs. Fabricators need to press erectors for help with simplification and overall man-hour savings.

Discussion will include erector/fabricator pre-bid planning for efficient erection techniques and connection design, as well as alternate suggestions for the design engineer.
Thursday at 1:30 p.m.
Friday at 1:00 p.m.

9 Research into Practice

Speakers:
W. Samuel Easterling, Virginia Tech
Ahmad M. Itani, California Dept. of Transportation
Much work has been done on semi-rigid beam-to-column connections, *Easterling* will discuss semi-rigid beam-to-girder connections. His presentation will discuss the research that is in progress and the design recommendations that will be the result of the research.

Seismic research will be the topic of *Itani's* presentation. Special Truss Moment Frames will be introduced and compared to the behavior of Special Moment Resisting Frames.
Thursday at 10:45 a.m.
Friday at 10:00 a.m.

10 TQM Personnel Issues

Moderator: Sid Blaauw, Paxton & Vierling Steel Co.
Speaker: Leo Peacock, Paxton & Vierling Steel Co.
Total Quality Management is a system that can work in steel fabrication shops. Mr. Peacock is the employee of a fabrication firm and is implementing a Total Quality Management system in that firm. He will give us the benefit of his experience in tailoring this successful management scheme to our business. Topics for discussion will include problem solving, employee empowerment, and partnering.
Thursday at 1:30 p.m.
Friday at 2:40 p.m.

11 International Developments

Moderator: Theodore Galambos, University of Minnesota
Speakers:
J. W. B. Stark, TNO - Bouw
Ben Kato, Takenaka Corp.
Fabricators and designers in 1994 will be very interested in the international marketplace. Overseas development will be the topic of this session. *Stark* will discuss the Eurocode requirements for steel and composite structures. *Kato* will present the new Japanese standard in *limit-states design.*
Thursday at 1:30 p.m.
Friday at 10:00 a.m.

TECHNICAL SESSIONS

12

Safety = Dollars

Moderator: Terry Peshia, Garbe Iron Works, Inc.

Speakers:

Richard Morgan, CNA
Byron Spencer, Norman Spencer
Gretchen McAlinden, *Norman Spencer*
Charles McCarthy, CNA
Byron Sinclair, CNA

What has AISC done to help the industry properly insure itself at the right price? How are we dealing with design responsibility risks from the perspective of the detailer, the fabricator, and the professional engineer members? What are the major causes of fabricator losses and what has the AISC Safety Task Force done to help our members minimize or avoid expensive claims?

Thursday at 1:30 p.m.
Friday at 1:00 p.m.

13

Building Retrofit

Speakers:

Terry R. Lundeen, Ratti Swenson Perbix, Inc.
Peter A. Timler, Sandwell Inc.

In the present construction marketplace some of the only work available is renovation and retrofit of existing structures. Two projects will be presented in this session that show how steel can be used in this type of work. *Lundeen* will present the renovation of a steam generating plant into a biomedical research facility. *Timler* will discuss how a building at the British Columbia Institute of Technology was retrofit incorporating the new seismic provisions.

Thursday at 3:10 p.m.
Friday at 2:40 p.m.

14

Marketing—Getting Your Share In Good Times and Bad

Moderator: Phil Stupp, Stupp Bros. Inc.

Speakers:

Andy Johnson, AISC Marketing, Inc.
Robert Shaw, Steel Structures Technology Center, Inc.

All fabricators were not created equal. The first portion of this workshop will show ten positive marketing steps the fabricator can take to get in front of the pack. Points to be presented are not theoretical but rather straight forward and practical. One of the basic tenants of good salesmanship is to KNOW YOUR PRODUCT. The second portion of this workshop will present the essential facts a fabricator needs to know to sell fabricated structural steel over other materials. Each participant will receive take home material to serve as a working reference.

Thursday at 3:10 p.m.
Friday at 10:00 a.m.

15

Experience from Wind Damage & Design Load Requirements

Speakers:

Lawrence G. Griffis, Walter P. Moore and Associates, Inc.
Robert J. Wills Jr., *AISI*

Many times serviceability requirements control the design of a structure. *Griffis* will discuss serviceability of steel buildings in regard to wind, specifically covering deformation and motion perception. *Wills* will review Hurricane Andrew and the effect that this devasting hurricane had on buildings in South Florida and the impact on the building codes.

Thursday at 3:10 p.m.
Friday at 1:00 p.m.

16

Software Requirements

Speakers:

Souhail Elhouar, Virginia Tech
Kevin Parfitt, Penn State

This session will be an open discussion for professionals to discuss their software needs. The first session will be geared towards fabricators and the requirements of fabricator software. The repeat of this session will focus on the engineering community and their software requirements.

Thursday at 10:45 a.m.
Friday at 1:00 p.m.

1994 NATIONAL STEEL CONSTRUCTION CONFERENCE
DAVID L. LAWRENCE CONVENTION CENTER
PITTSBURGH, PA • MAY 18-20

HOTEL REGISTRATION

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NUMBER

EXPIRATION DATE

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of Steel Construction
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HOUSING BUREAU DEADLINE DATE:

APRIL 10, 1994

Analysis Of Unbraced Frames With Partially Restrained (PR) Connections

By Roberto T. Leon

Roberto T. Leon, an associate professor in the Department of Civil and Mineral Engineering at the University of Minnesota-Cities, is the 1993 T.R. Higgins Award Winner.

Current specifications for the design of steel frames require designers of partially restrained frames (Type 3 or PR) to provide either experimental or detailed analytical studies that prove that the connection "have a predictable proportion of end restraint." Although semi-rigid action has been recognized in the codes for more than 40 years, few if any designers make use of true PR connections because of this onerous requirement. Nonetheless, these connections have received extensive use through the escape clause provided by the code which turns PR connections into "simple framing" ones (LRFD Specification, Section A.2).

In a recent article (Modern Steel Construction, May 1993), the author described the behavior and design of semi-rigid composite connections. These are connections that utilize the negative moment capacity provided by continuous reinforcing bars across column lines (Figure 1). Extensive experimental and analytical work has shown that these connections have wide applicability in moment-resisting frames up to 10 stories in areas where wind or moderate seismic loads govern the design. The author believes that semi-rigid composite connections have reached the stage of development where they qualify for an exception to the PR requirements similar to that granted to "simple framing" connections.

A key question that always arises in conjunction with the use of Type 3 or PR frames is the level of analysis required in the design phase. The typical moment-rotation (M-Θ) curve for a PR composite connection is shown in Fig. 2. This curve corresponds to the connection shown in Fig. 1 with a A572 W24 beam, 8 #4 Grade 60 bars in the slab (A_{br} = 1.6 sq. in., F_{yr} = 60 ksi)), an A36 seat angle (5/8-in. thick by 7-in. wide, A_{sL} = 4.38 sq. in., F_{yL} = 36 ksi), and an A36 web angle (3/8-in. thick by 12-in. long, A_{wL} = 9 sq. in.). The bottom connection to the beam is made up of four 1-in. A490 bolts (A_b = 2.40 sq. in., F_v = 44 ksi).

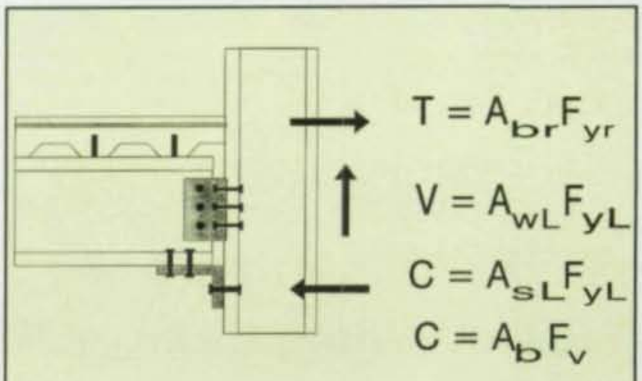


Figure 1

There are at least six major problem that need to be addressed in conjunction with such a M-Θ curve:

The connection is non-linear from early on in the moment-rotation curve.

The connection behavior is not symmetrical, i.e. the positive and negative bending behavior are dissimilar.

The connection is only active for loads applied after the concrete has hardened, i.e., mostly for live loads.

Because the moments in the beam may change from positive to negative, the section cannot be assumed as prismatic.

Since PR frames are perceived as

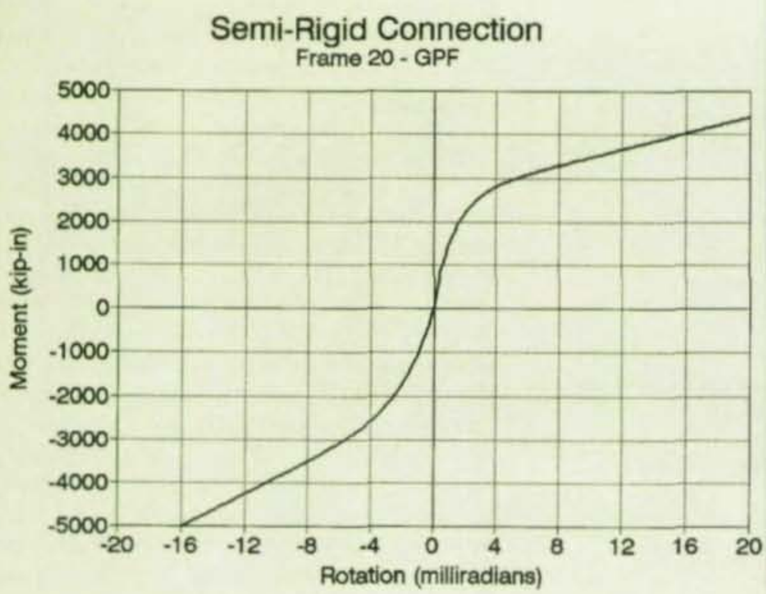


Figure 2

drifting more than rigid (FR) ones, a more thorough investigation of second-order effects needs to be carried out.

The connection softens considerably as the ultimate strength is approached leading to numerical stability problems in the analysis.

The logical solution to all the problems listed above can be found in a two-level approach to design. For the serviceability range an analysis using linear springs can be used provided a good approximation to the initial stiffness of the connection is available. For the ultimate strength range a second-order plastic analysis approach can provide a quick and accurate value for the collapse load. In the next few paragraphs we will illustrate how to apply this to a regular eight-story, three-bay frame (see Table 1 for frame details).

As noted above, to calculate drifts at the service load level the initial stiffness of the M-Θ curve must be known. This can be done in several ways if an explicit equation is available. A first approach may be to differentiate the M-Θ equation to obtain an explicit equation for the tangent stiffness. The problem of what rotation to use in this latter equation remains since the value of stiffness changes constantly. For the type of connection of interest here (Type 3 semi-rigid composite), a value of 1.5 milliradians is recommended. For the connection shown in Fig. 2 the tangent stiffness at 1.5 milliradians is 662 kip-in/milliradian.

Another approach that avoids the difficulties often encountered in differentiation is to arbitrarily assume that the serviceability limit will be reached at some particular value of rotation. Since designers often limit drift to H/400 (H is the height of the structure) it is convenient to assume that the serviceability rotation (Θ_{ser}) corresponds to this deflection. If one assumes that both beams and columns are rigid and that all deflections are concentrated in the connection, the serviceability rotation corresponds to 2.5 milliradians. By substituting this value of rotation into the M-Θ, the value of the serviceability moment (M_{ser}) can be found. A secant stiffness ($k_{ser} = M_{ser}/\Theta_{ser}$) can then be obtained directly. For the connection in Fig. 1, the secant stiffness at 2.5 milliradians is 789 kip-in/milliradian. This secant stiffness approach should be conservative for any equivalent static load analysis.

At the end of this preliminary analysis the designer can check whether the Θ_{ser} corresponds to the assumed one. If the connection rotation is less than assumed and the drift does not exceed the desired limit no further analysis is required. If the drift is excessive a better approximation to the real tangent or secant stiffness can be found by utilizing the actual rotation output by the computer program.

It is always useful to run an analysis for the same frame with rigid connections to assess the relative contribution of the elastic deformation of both beams and columns versus that of the PR connection. This will allow the designer to

Finding Moment Rotation Curves

A computer program has been developed by the RMR Design Group to generate moment rotation curves for partially restrained connections.

PRCONN runs on IBM-compatible computers and offers 31 different connection options, including both composite and non-composite. It also gives the option of bolting or welding and constant or variable load beam lines. And, it offers uniform and concentrated load options with midspan deflections.

For more information, contact: RMR Design Group, Inc., 4421 E. Coronado Dr., Tuscon, AZ 85718 (602) 621-8251.

Table 1: Example Frame

Level	Interior Column	Exterior Column	Beams
1-2	W14x145	W14x90	W24x55
3-4	W14x109	W14x82	W24x55
5-6	W14x90	W14x61	W24x55
708	W14x82	W14x53	W24x55

Frame spacing = 28 ft.
Story heights = 14 ft.
All beams and columns F_y = 50 ksi
All beams are composite, $I_{comp} = 0.4\, I_{neg} + 0.6\, I_{pos} = 1920$ in.^{4}
DL = 60 psf; LL = 100 psf; LL_{red} = 62 psf

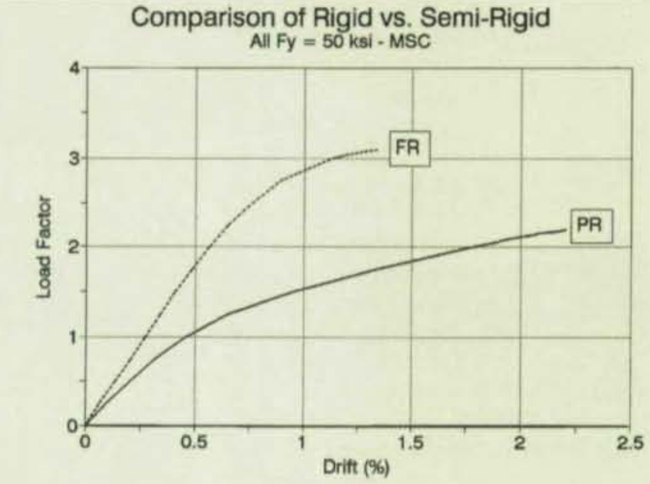


Figure 3

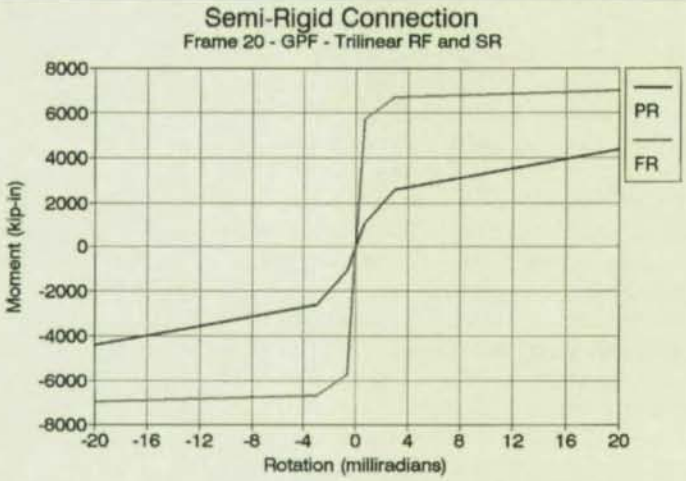


Figure 4

determine whether his connections are sufficiently stiff. For design purposes a minimum value for the ratio of the elastic deformations of the frame to those due to connection rotation can be assumed. The author recommends that elastic deformations make up two-thirds to three-quarters of the total deformations.

Making the assumption that this ratio is two-thirds is equivalent to assuming that the rotation in the connection will be smaller than 0.83 milliradians ($\Theta_{ser}/3 = 2.5/3$). This implies that the Θ_{ser} value suggested above to determine the secant stiffness is very conservative. However, experimental evidence shows that the initial stiffness of connections very widely. Deviations of 50% to 100% in initial stiffness from the values predicted by explicit equations are not uncommon at low deformations even for specimens carefully constructed in the laboratory. Thus a conservative approach such as the one described above is not unreasonable. As the designer gains experience with the design of these systems he can adjust the values of k_{ser} that he would like to use for his preliminary analysis.

The complete lateral load-drift behavior for the example PR frame is shown in Fig. 3. A similar curve for a FR (Type 1 or rigid) frame is also shown. The curves shown correspond to the case of 1.0D + 1.0L + X.XW, where X.X is the load factor plotted in the vertical axis. The design load then corresponds to a load factor of 1.0. The drift has been normalized to H/400, so a value of 1 in the horizontal axis corresponds to the usual allowable drift.

To simplify the analysis a tri-linear M-Θ was used, and the M-Θ curves used are shown in Fig. 4. In both cases only the connections to the interior columns were assumed to transfer moment since a slab overhang is required to make the exterior connections effective for the composite case. This overhang was assumed not to exist in this case.

In many cases the effect of the exterior connections can be significant. For the example frame described in Table 1 making the exterior connections effective results in 20% decrease in drift for the PR case. Thus it is reasonable to use a PR steel connection (top and seat angle, for example) at exterior columns to stiffen frames using composite connections if the slab overhang is not available.

For the ultimate strength limit state a plastic analysis is recommended. The plastic analysis of multi-story frame is not common in practice because the number of mechanisms that need to be checked increases with the height and number of loads. For the case of a semi-rigid composite frame the situation is considerably simplified because the connections are so much weaker than the composite beams in positive bending. This means that only a beam-sway mechanism needs to be checked (Fig. 5). The collapse load can be calculated simply as:

$$(N+1)\, M_{p,col} + ((N-1)*S) * (M^{+}_{p,conn} + M^{-}_{p,conn})_{inte} + (S)\,(M^{+}_{p,conn} + M^{-}_{p,conn})_{exte} = \sum (P_i * H_i)$$

where N is the number of bays, S is the number of stories, P_i and H_i are the loads and heights at each story, and the subscripts "inte" and "exte" refer to the exterior and interior connections. For the PR frame in Fig. 3 this results in a load factor for lateral loads of 3.49. For the FR frame the corresponding value is 5.33. These values are only about 60% of the collapse load given by the exact analysis.

Unfortunately the simple calculation given in the above equation cannot be used directly in design because the P-δ effects cannot be ignored. The work done by the axial loads in the columns needs to be included into the equation above. There are at least two simple ways of doing this. The first one is by using the Merchant-Rankine formula [Horne, M.R., and Morris, L.J., Plastic Design of Low Rise Frames, MIT Press, 1982]:

$$\lambda_R = \frac{\lambda_p}{(1 - \frac{\lambda_p}{\lambda_{CRIT}})}$$

where λ_R is the actual collapse load factor, λ_p is the plastic collapse load factor, and λ_{CRIT} is the elastic critical load for the frame. The latter can be quickly computed by an eigenvalue analysis of the stiffness matrix for the structure. This formula has been shown to give reasonable values for large ratios of lateral to vertical loads, and to be conservative for large vertical to lateral load ratios.

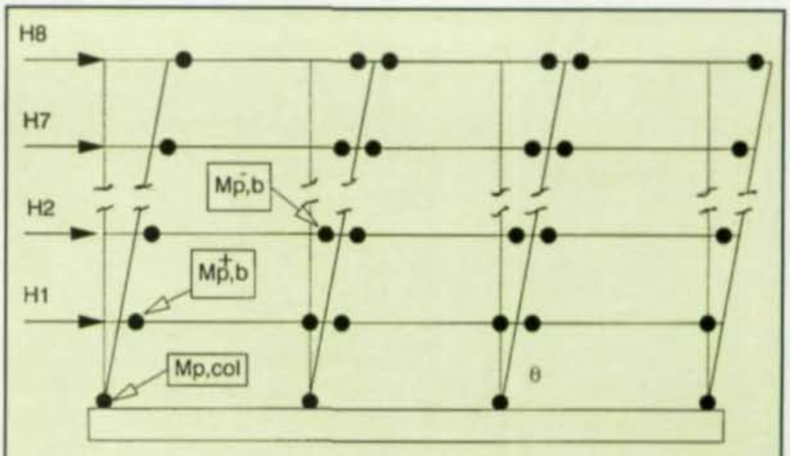


Figure 5

A second approach is a variation of the Merchant-Rankine approach that has been proposed by Horne and Morris [Horne and Morris, idem] based on the analysis of many frames. In this case the collapse load is given by:

$$\lambda_K = \frac{\lambda_p}{1 + 2.5\lambda_p^2 \left(\frac{\sum P\Theta\delta}{\sum M_p \Phi} \right)}$$

where λ_K is the new load factor, P are the axial forces in the columns at ultimate, δ are the elastic interstory displacements corresponding to λ_p, and Θ and Φ are the rotations. The 2.5 in the denominator is the ratio of the inelastic deflection at collapse to the elastic deflection at the collapse load and may need to be adjusted for PR frames. The use of this second approach results in collapse load factors 8% smaller than the exact calculation.

The two-level design approach described here combines both simplicity and safety. Most modern computer software includes linear springs as an option, making the calculation of the service load drift simple. The plastic design approach at ultimate not only gives a good feel for the frame behavior, but also highlights the importance of second-order effects. More detailed information on design of composite PR systems will be found in an upcoming issue of the AISC *Engineering Journal* (first quarter 1994).

Federal Highway Administration Guidelines

Economical Steel Bridge Design

By Vasant Mistry, P.E.

When designing steel bridges, it used to be sufficient to determine a least-weight solution to obtain the most economical design. However, over the past decade, material costs have decreased approximately 10% while the cost of labor has increased nearly 20%. This means designers must reduce fabrication costs to achieve substantial cost savings.

Fabrication and erection costs can be reduced through thoughtful and optimal design of structural connections. Another result of changing market place conditions is the fact that the designer must consider an increased number of options for maximum cost-effectiveness. This article presents some of the important features of economical steel bridge design and some guidelines to achieve improved design.

A least-weight solution no longer guarantees the most economical bridge design

Design Methods (WSD/LFD)

In most cases, Load Factor Design (LFD) is more cost-effective than Working Stress Design (WSD). The LFD method provides a uniform and consistent level of live load safety. All bridges designed by LFD and all elements in such bridges have approximately the same margin of safety against failure. In other words, LFD structures are designed more in accordance with the manner in which they behave.

One of the most important differences between the LFD and WSD methods is the way required load or safety factors are applied in ensuring the limit state strength of the structure. In WSD, one uniform factor of safety is applied to both dead and live loads while in LFD the factor of safety applied to dead load is lower than that applied to live load. The lower factor of safety for dead load is justified because of the much lower uncertainties associated with dead load than with live loads.

The effect on required section modulus resulting from these differences in safety factors, and other differences between the methods, varies with the live load to dead load ratio or the span involved. LFD can produce cost savings of 5% to 12% in spans up to 200 ft., and 12% to 20% in longer spans.

The LFD method has been steadily gaining acceptance among bridge designers due to its economical and more realistic design. Recently, the Load and Resistance Factor Design (LRFD) Specification has been adopted by AASHTO. Designers will have additional control and confidence in design parameters when using LRFD.

Steel Grades

A proper mix of Grade 36 steel and high-strength steels should be selected. High-strength steels are advantageous when strength is the major design criterion. However, when deflection, stiffness or some other serviceability criterion governs, it may be advantageous to use Grade 36 steel because of its greater section modulus. With steady narrowing of the cost difference between high-strength steel and Grade 36 steel, the added strength often becomes well worth the modest—or even non-existent—cost premium.

Unpainted Weathering Steel

Unpainted weathering grade steels used in proper locations and/or under proper conditions result in both short- and long-term savings. On a first-cost basis, unpainted weathering steel may save approximately 10%-20% compared with painted steel, depending on the coating system used. It is even more cost-effective on a long-term basis when elimination of future maintenance painting is considered.

Weathering steels are currently supplied under AASHTO Specification M270 (ASTM A709) with grades 50W, 70W and 100W available. However, the use of weathering steel should be considered with caution in applications that involve:

1. Exposure to highly corrosive fumes;

2. Severe marine conditions with repeated wetting by salt spray or fog;

3. Burial in the ground;

4. Submerged without adequate protection of steel;

5. Details that are subjected to run-off containing de-icing salt.

(For further information on the use of weathering steel, refer to the FHWA Technical Advisory "Uncoated Weathering Steel In Structures" T5140.22, October 3, 1989.)

Corrosion of weathering steel members is most likely to occur on horizontal surfaces and in crevices and re-entrant corners. On I-girder bridges it has been found that locations where corrosive attack is most likely are bottom flanges, gusset plates, longitudinal stiffeners, splices of horizontal and sloped members, and where flanged gusset plates contact bearing and intermediate stiffeners. Therefore, good design and detailing practice should include:

1. Eliminating crevices and minimizing re-entrant corners;

2. Providing for good drainage at low points when girders slope away from the center supports and toward the end supports;

3. Changing the flange thickness instead of the width where welded flange splices are used, because changes in width may cause uneven water flow. Good detailing practice such as described in AISC Marketing's recently published "Uncoated Weathering Steel In Bridges" will provide safe, economical, long-lasting steel bridges *(for information, call AISC Marketing at 412-394-3700)*.

Span Length/Span Layout

In most cases, rolled beams are less expensive per pound of fabricated steel for simply supported short span bridges than welded steel plate girders. However, plate girders use less steel and as a result can sometimes have lower total cost.

Table 1 shows a few examples of typical girders and relative cost effective span lengths.

Layouts with shorter spans are cost effective when the substructure cost is relatively small. Longer spans are generally cost effective if substructures require costly foundation work. Computer programs are available to help the bridge engineers arrive at cost effective span arrangements. The optimization program "Simon" will soon be available from AISC Marketing in a PC version.

Table 1:

Type of Girder	Cost Effective Span Lengths
Rolled Beams	50-ft. to 90-ft.
Plate Girders	80-ft. to 250-ft.
Box Girders	150-ft. to 250-ft.
Trusses	350-ft. to 900-ft.
Cable Stayed	800-ft. to 2,000-ft.

Girder Spacing

Substantial savings can be achieved by using wide girder spacing. These cost savings result primarily from reductions in web material and fabrication costs resulting from the reduced total length of girders. Fabrication labor costs tend to be roughly proportional to total length of girders. A reduction in number of girders normally also results in savings in erection, shipping and maintenance costs. Such savings are partially offset when the deck thickness has to be increased to accommodate the wider girder spacing. However, additional savings result from the reduction in the number of cross frames and bearings. Therefore, selection of girder spacing is one of the most important influences on economy of a steel girder bridge. Generally, the widest practical spacing results in the most economical structure for spans greater than 80-ft.

> Reducing the number of girders can cut fabrication, erection, shipping and maintenance costs

For longer span girders, larger spacing, approaching 20-ft., may be economical. Such spacing can be achieved by using higher strength concrete and transverse prestressing for the deck slab.

Sub-stringer framing systems can permit wide girder spacing with conventional slab designs. In this system, the girders are spaced at about 18- to 20-ft. and a single rolled-beam stringer is placed midway between and supported on cross frames or floor beams.

Composite Design

The use of composite construction in positive-moment regions has been generally accepted and is usually necessary for maximum economy of girder bridges.

The use of composite construction in the negative moment region of continuous spans over the piers should be considered. Even if negative moment regions are designed as non-composite, AASHTO allows inclusion of the deck concrete in computing moments of inertia for deflection calculations and for determining stiffness factors used in calculating moments and shears.

Web Design

Most web optimization computer programs investigate girders within a practical range of web depths in specified increments. Within each depth, iteration is done in web thickness by 1/16-in. increments, assigning a cost index to each web thickness.

Unstiffened webs are generally more economical for web depths 50-in. or less

Both constant web thickness for the entire length of the girder as well as variable web thickness have been used in the plate girder designs. The variable web thickness design usually costs 1% or 2% less than constant web thickness design. In general, variable thickness web design is suggested for long-span plate girders.

Because labor costs are rising while material costs are decreasing, web design can have a significant impact on the cost of plate girder fabrication.

The transverse stiffeners should only be placed on one side of the web, with the exception of diaphragm connections, which are required on both sides. Transverse stiffeners should not be specified to bear on both the top and bottom flanges unless this is an absolute design requirement. Fitting transverse stiffeners is a very time-consuming operation because each stiffener has to be individually cut and ground to fit each location.

The often-asked question is: "How much thicker does it pay to make a plate girder web in order to reduce the number of transverse web stiffeners or eliminate them?" Unstiffened webs are generally more economical for web depths approximately 50-in. or less. For web depth greater than 50-in., consideration should be given for showing options for stiffened and unstiffened webs. Both options should be detailed on the plans, allowing the contractor the alternative of providing a thicker web plate and eliminating the need for stiffeners on plate girders. One rule of thumb is to use a 1-2-4 estimating guide. That is, if the girder steel is $1 per pound, then cross-frame and diaphragm steel is $2 per pound and stiffener and shear connector steel is $4 per pound.

Longitudinal stiffeners, which often are used in conjunction with transverse stiffeners on longer spans with deeper web girders, should be placed on the opposite side of the web from the transverse stiffeners. The intersection of longitudinal stiffeners with transverse stiffeners causes additional shop labor costs and also results in fatigue-prone details. The intersection of a diaphragm connection plate and longitudinal stiffeners should be carefully detailed to minimize fabrication costs and fatigue-prone details. AISC recommends that if longitudinal and transverse stiffeners are on the same side of the web, the longitudinal stiffeners should be continuous and the transverse stiffeners should be interrupted. Studies have shown that longitudinal stiffeners usually are not economical for spans of less than about 300-ft.

Flange Plates

Costs of plates for flanges can represent a significant portion of material costs. However, the labor costs of fabricating flange plates can vary greatly as a result of combination of design, purchasing and shop practices. Often, bridge software designs emphasize least weight options, which means the design mixes a wide variety of sizes and shapes, which is not practical. The result is that the fabricator cannot economically buy material.

It generally does not pay to vary flange widths at a field splices because of the high cost of tapering the ends of the wider flange plate. The narrowing of flanges should be eliminated for small or even modest weight savings. The increased labor costs incurred by splicing often more than offsets the material cost savings. However, since the break point is different for each fabricator, and also will vary from job-to-job, the best solution is to give the fabricator options. As a general rule, an average of about 700 lbs. flange material should be saved to justify the introduction of a flange shop splice. When flanges have to be spliced, the widths should be kept the same whenever possible. This allows for splicing the flanges for several girders before cutting to width.

Also, allow some flexibility in shop splice locations. Flange plates must be nested to obtain widths that can be ordered from a mill. Sizes should be kept to 1/8-in. increments for 1-in. to 2-in. thick plates and 1/4-in. increments for plates over 2-in. thick. Typically, there is a 48-in. minimum order width. If there are only one or two flange plates of the same thickness on a job, additional material must be ordered by the shop to obtain these plates. Minimum flange thickness should be kept in the 3/4-in. to 1-in. range. Thinner flanges will increase the cost due to problems with heat distortion during welding, handling during shipping and erection, and concerns for lateral stability while placing deck concrete.

Haunches

For multiple-span continuous girders, site conditions or aesthetics may require haunched girders. Where underclearance is not restricted, studies have shown that these designs are not competitive with constant girder depth designs for spans of 350-ft. or less. In cases where bids were invited on haunched versus parallel flange designs for structures in the 400-ft. span range, all steel bids were for the constant girder depth design. It is recommended that haunched designs be considered only for spans greater than 400-ft.

Simpler Details

To minimize costs, bridge details must be as simple as possible. There are a large variety of details and many of each type, which makes it difficult to present them all. However, a few examples of crossframe and diaphragm details are worth discussion.

Secondary members, such as crossframes, diaphragms, lateral members, etc., represent approximately 10% of total structure steel weight and therefore often are given much less attention at the design stage. As a result, their design frequently incorporates a disproportionate amount of shop labor for the weight.

Crossframes, diaphragms and lateral bracing can be made simpler by using the following suggestions:

1. Contractors should be given the option to either bolt or weld diagonals and struts to a gusset plate.

2. Gusset plates groove butt welded to the stems of WTs and angle legs should be avoided. Use lap joints with fillet welds wherever possible.

3. There should be as few cuts as possible on gusset plates.

4.Tee stems should be turned outward to simplify connections.

5. Flat bar sizes should be used for stiffeners to reduce cutting, grinding and material cost.

6. Lateral bracing should be eliminated wherever possible in accordance with the AASHTO Specifications.

7. Each detail should be examined for: simplicity; potential stress risers; over-welding (whether in weld size, weld length, weld preparation and grinding or penetration requirement); and classification of fatigue category.

Paying more attention to secondary members can result in big dividends

Cutting Costs

A major part of the cost of the steel framing system of a bridge—often more than half—is the fabrication, shipping and erection. There are some general principles that can be followed to reduce construction costs.

Fabrication

1. Open communication is necessary between the designers and the people who build the structures. Bridge design and construction is a team effort and communication of design requirements between fabricator and engineers is crucial for a safe and economic structure. Keep abreast of current costs of various steel products used in structural design. Steel fabricators can supply current base prices upon request.

2. Odd sections that may not be readily available or that are seldom rolled should be avoided. These can result in costly delays.

3. Excessively stringent mill, fabrication and erection tolerances, beyond state-of-the-art practices, will probably reduce the number of bidders and raise the cost to the owner. ASTM A6 tolerances and those established by AASHTO, AWS, ANSI, and AISC have served the industry well and should be adhered to except under extraordinary circumstances.

4. Wherever possible, fillet welds should be used. Full penetration welds are more expensive and often are used unnecessarily.

5. Wherever possible, limit weld sizes to the minimum required. Keep in mind the maximum single pass fillet weld size is 5/16-in.

6. Newer coating systems are much more tolerant of abrupt changes in surface profile. Therefore, current standard requirements for extensive grinding to form smooth radii at plate edges can be substantially modified. Many new coating systems only require light beveling of the edges.

Studies have shown that rough edges do not need to be ground smooth to ensure paint adhesion

7. Quick drying paints should be specified. More and more bridge owners are requiring second and third coats in the shop. It is time-consuming and expensive for the fabricator to wait for the paint to dry.

8. When the cleaning and painting system is such that bolt friction values are not reduced, consideration should be given to allow paint on field contact surfaces. This will eliminate the need to mask the contact surfaces.

9. Only members that must be fracture critical should be labeled FCM. Fracture-critical members cost considerably more to fabricate and inspect than ordinary members. It is often cost-effective to take steps to minimize the number of members that must be classified as fracture critical.

10. Reducing the number of fabrication operations normally reduces fabrication costs, but often increases material costs. The most economical design usually results from balancing these costs.

11. Duplication usually reduces fabrication costs. For example, duplicating flange sizes, web sizes, stiffeners, diaphragms and bracing within a single bridge or multi-bridge job will generally result in savings.

Erection

1. Girder segment length should preferably be limited to about 120-ft. and weight to approximately 90 tons. Anything larger makes shipping and erection more expensive unless shipment by barge is available.

2. Field splices should be located close enough to each other so that individual pieces will be stable without requiring special stiffening trusses, falsework, etc. As a general rule, AISC recommends that the unsupported length of the field piece divided by the minimum width of compression flange should be less than approximately 85.

Fatigue Prone Details

Most cracks found in the existing bridge steel superstructure are related to fatigue prone details. These should be carefully examined at the design stage and eliminated if possible.

1. Fatigue behavior of web gusset plates: A small gouge or undercut at the weld toe will significantly reduce the fatigue resistance of the web gusset plate detail. Cracking will develop below the Category E' fatigue resistance curve when this type of defect is present.

2. Fatigue behavior of transverse stiffeners: Test results suggest that stiffener details are not likely to develop fatigue cracks in service. Available field test data indicate that the constant amplitude fatigue limit will not be exceeded enough to cause significant fatigue damage to in-service structures.

The findings presented in NCHRP Report 336 "Distortion-Induced Fatigue Cracking In Steel Bridges," 1990, are appropriate for girders with transverse connecting plates subjected to out-of-plane distortion. Small web gaps were found to be very susceptible to fatigue cracking. An increased gap length of 4-1/2-in. (100mm) reduced the sensitivity to fatigue cracking for moderate levels of out-of-plane movement. To eliminate lateral out-of-plane distortion, the AASHTO Specifications (Art. *10.19.3.2*) requires diaphragm or cross-frame connection plates to be rigidly connected to both top and bottom flanges.

However, the AASHTO specifications require that the vertical connection plates such as tranverse stiffeners that connect diaphragms or cross frames to the beam or girder shall be rigidly connected to both the top and bottom flanges.

3. Fatigue behavior of cover plate details: Limited data on large-scale cover-plated beams corresponding to Category E' demonstrates that fatigue cracks formed at cover plates without transverse end welds when the constant amplitude fatigue limit (CAFL) was exceeded by 4.3% or more of the variable amplitude cycles. The effective stress range was below the CAFL for this reported load spectrum.

4. Retrofitting web cracks: Variable amplitude loading tests verified the adequacy of arresting fatigue crack extension in girder webs by holes placed at the tips of the cracks. For plates having a yield stress of 36 ksi, a hole diameter between 3/4-in. and 1-in. is usually sufficient.

5. Typical fatigue prone details and/or their locations:

- intersecting welds of lateral connection plates;
- transverse connection plates subjected to out-of-plane distortion;
- floor beam-to-girder connections; stringer-to-floorbeam connections (out-of-plane bending);
- rolled beam diaphragm connections;
- coverplate end transverse welds;
- termination of coverplate longitudinal welds;
- web plate penetrations including box pier caps;
- web and flange splices;
- web cope holes;
- lateral connections to girders and floor beams;
- pin and hanger connections;
- backing bar splices;
- repair holes with partial or complete plug welds;
- tack welds;
- gouges in flanges.

Table 2:
Cost Comparison Among Various Bridge Bearings

Type of Bearing	Initital Cost	Maintenance Cost
Elastomeric	Low	Low
Disk	High	Moderate
Sperical	Very High	Moderate
Pot	High	High
Pin, Roller, Rocker	Moderate to Very High	Moderate to Very High

Other considerations

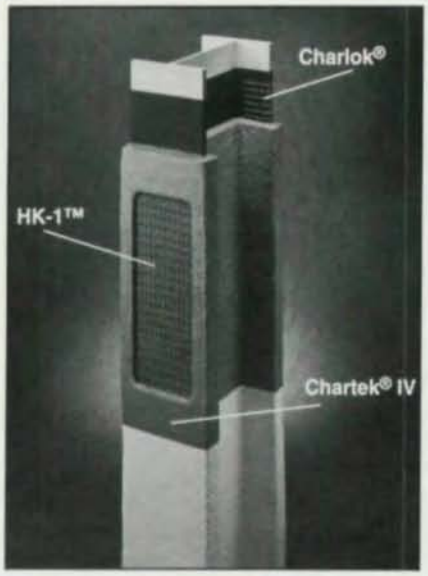
1. Jointless bridge decks: One of the most important aspects of design, which can affect structure life and maintenance costs, is the reduction or elimination of roadway expansion joints and associated expansion bearings. Unfortunately, this is too often overlooked or avoided.

Designers should always consider the possibility of minimum or no-joint construction to provide the most durable and cost-effective structure. Joints and bearings are costly to buy and install. The most frequently encountered corrosion problem involves leaking expansion joints and seals that permit salt-laden run-off water from the roadway surface to attack the webs and flanges of the bridge members, bearings and pier caps below. Many of our most costly maintenance problems originated with leaky joints. To eliminate these problems, bridges should be designed and constructed with continuous superstructures and no transverse deck expansion joints provided unless absolutely necessary. Steel superstructure bridges up to 400-ft. long have been built with no joints, even at the abutments. And Tennessee DOT has built steel bridges up to 2000-ft. long with no joints except at the end abutments.

Continuous superstructures can help to eliminate the problems caused by leaky joints

2. Elastomeric or pot bearings: The bridge and its bearings should be considered and designed as a unified system. Consideration should be given to use of elastomeric bearings or pot bearings instead of expensive, custom-fabricated steel rocker bearings. Although elastomeric bearing pads are by far the lowest in cost of any of the bearing details in the AISC study, they are not being used to the extent that AASHTO specifications permit and economy justifies. Careful attention to the design of bridge details, based on objective analysis, rather than precedent, can significantly reduce the cost of steel bridges. The long-term performance of steel bearings is the constant bane of bridge maintenance engineers.

Vasant Mistry, P.E., is with the U.S. Department of Transportation, Federal Highway Administration.



Passive Fire Protection

Textron Specialty Materials has recently introduced CHARTEK IV Fireproofing and HK-1 mesh reinforcement. This new generation of passive fire protection was developed in response to industry demand for a lighter weight material that is easier to apply but still provides the same benefits of longevity and protection against severe fire exposure of steel members as the existing CHARTEK materials. The material is a spray-applied epoxy intumescent that expands, foams and chars up to several times its original thickness when exposed to fire. It uses no harmful solvents during application.

For more information, contact: Textron Specialty Materials, Two Industrial Ave., Lowell, MA 01851 (508) 934-7502.

Fire Retardent Paints

FlameControl Coatings offers a line of fire retardent paints, varnishes and mastics for structural steel. The coatings comply with federal, state and local building code requirements and have been tested for compliance with a wide range of ASTM, UL, and ULC CAN standards.

For more information, contact: FlameControl Coatings, Inc., Main P.O. Box 786, Niagara Falls, NY 14302 (716) 282-1399; fax: 716/285-6303.

Fire Protection Topcoats

Fire Research Laboratories has added two new topcoats to its line of fire protective coatings. TopCoat A is an interior, waterborne clear overcoat that provides extra durability and protection when used over fire protective finishes. It also exhibits excellent UV-resistance. TopCoat X is a waterborne pigmented overcoat designed specifically for exterior applications. It may be tinted using universal colorants.

For more information, contact: Fire Research Laboratories, 5364 Pan American Freeway N.E., Albuquerque, NM 89109 (800) 877-3473.

Spray-Applied Fireproofing

CAFCO Deck-Shield I from Isolatek International is a medium densisty, abrasion-resistant fireproofing for exposed steel surfacs that require protection from day-to-day abuse. Spray-applied and econimcal, the material dries quickly to a smooth finish with exceptional durability. It is non-combustible and certified 100% asbestos-free. The dry product is applied with low-pressure pneumatic spray equipment and requires only minimal amounts of water, which is introduced at the spray nozzle. It provides fire ratings up to four hours in accordance with recognized fire test procedures.

For more information or a copy of the CAFCO Deck-Shield I brochure, contact: Isolatek International, 41 Furnace St., Stanhope, NJ 07874 (201) 347-1200; fax: 201/347-9170.

Intumescent Fire Protection

PPG Industries has introduced PITT-CHAR XP fire protective coating, a 100% solids, flexible, epoxy-based intumescent material. This patented coating uses a flexible, crosslinked epoxy resin system, providing enhanced protection from physical, chemical and climactic exposures, resulting in long term corrosion and fire protection. The coating is designed for ease of application and rapid cure, allowing structural steel to be shop coated before assembly. The coated steel is typically ready to be hoisted and shipped after an overnight cure. It is applied by airless spray and presents no hazards from solvents, asbestos, heavy metal or known carcinogens. PITT-CHAR XP is fully UL listed and has passed both the UL1709 hydrocarbon fire test and the UL263/ASTM E-119 fire test.

For more information, contact: PPG Industries, Technical Sales Department, 151 Colfax St., Springdale, PA 15144 (800) 531-0264.

Cementitious Fireproofing

Grace has developed a high-density spray-applied cementitious fireproofing, Monokote Type Z-146, for industrial and manufacturing facilities as well as exterior uses. It is designed to exceed standards for in-place physical properties, ease of use and fire protection consistent with the needs of industrial fireproofing applications. It can be applied using a wide variety of common plaster pumping equipment, eliminating the need for specialty pumps typically required for other high density fireproofing products.

For more information, contact: W.R. Grace & Co., 62 Whittemore Ave., Cambridge, MA 02140-1692 (617) 876-1400.

Spray-Applied Intumescent Coating

Albi high-performance fireproof coatings are engineered for structural steel exposed to weather, physical abuse, and other aggressive environmental factors. Albi products have undergone extensive UL hydrocarbon fire testing and meet stringent code and insurance requirements. Thematerial is spray-applied and requires no troweling, tamping or other labor intensive application procedures. Albi Clads are thin, hammer-hard, smooth, durable intumescent fireproof coatings with exceptional aesthetic appearance. Now available are Abli Clad 900 and Albi Clad TF, water-based intumescent fireproofing coatings. The coatings can be applied in a wide range of ambient temperatures, from below freezing to 100 degrees F. At thicknesses ranging from 1/8-inch to 3/4-inch, they closely follow the contours of steel and offer up to 3-1/2 hours of fire protection.

For more information, contact: Albi Manufacturing, Division of StanChem, Inc., 401 Berlin St., CT 06023; (203) 828-0571; fax: 203/828-0571.

Decorative Fireproofing

A thin film intumescent fire protection, A/D Firefilm, has been developed by A/D Fire Protection Systems to provide an aesthetic coating while protecting structural steel from fire. On the surface, the coating appears to be just a colorful paint. but hidden beneath is a thin film coating which can provide up to a two-hour fire rating. In a fire, however, the coating softens and then expands to form a meringue-like layer,up to 100mm thick, which insulates the structure from intense heat for up to two hours. The second component of the system, a decorative topcoat, acts as a protective layers and serves as the colorful finish. Application is by either brush, roller or spray and the coating is resistant to abrasion and most chemicals. A full range of decorative colors are available in a satin finish.

For more information, contact: A/D Fire Protection Systems, Inc., 420 Tapscott Road, Scarborough, Ontario, CANADA M1B 1Y4 (416) 292-2361; fax (416) 298-5887.

Steel Protection System

A structural steel fire protection system offering 1, 1-1/2, 2 and 3 hour ratings per ASTM E119/UL263-8.1 fire test is now available from Thermal Ceramics. It also offers either a 1, 1-1/2 and 2 hour rating per UL 1709 hydrocarbon fire test. The lightweight system won't crack or chip and requires no cleaning or coating of members to assure bonding. It installs at any temperature and eliminates overspray control and extensive cleanup.

For more information, contact: Thermal Ceramics, P.O. Box 923, Augusta, GA 30903 (706) 796-4328; fax (706) 796-4323.

Structural Steel Wrap Systems

Thermal Ceramics' FireMaster fire prtection system for structural steel members provides up to a three hour ratings per UL 263 and up to a two hour rting per UL 1709 test methods. The system offers cost effective, ultra-lighweight protection of columns and beams without the problems associated with spray-on systems. The flexible material is installed as manufactured, and therefore provides the customer with reliable protection at a fixed coverage rate. The installation is fast and simple, requires no special skills, and uses tools commonly found on most construction sites.

For more information, contact: Thermal Ceramics, P.O. Box 923, Augusta, GA 30903 (706) 796-4290; fax: 706/796-4328.

Fire Sprinkler Program

Hydronics Engineering is marketing a Windows-based and Macintosh-based computer program for fire sprinkler hydraulics. The programs use Newton-Raphson-Laguerre network convergence algorithms for calculating grids, trees, loops or hybrid piping systems. There are utilities for calculating hydrant flows, adjusted K-factors, pipe turbulence and velocity analysis.

For more information, contact: Hydronics Engineering, 34119 Fremont Blvd., Suite 609, Fremont, CA 94555 (800) 845-9819; fax (510) 475-8122.

Hydraulic Design Software

New versions of the widely used HP4M and HP6M Fire Sprinkler Hydraulic Design Programs have been introduced by MC2 Engineering Software. HP4M designs grid type sprinkler systems and the HP6M designs tree and loop systems. The new versions feature totally new screens for user inputs, allow the use of function keys, and have extensive default values.

For more information, contact: R.S. McClintock, President, MC2 Engineering Software, 8107 SW 72 Ave., Miami, FL 33143 (305) 665-0100; fax (305) 665-8035.

Sprinkler Control Software

HASS (Hydraulic Analyzer Sprinkler Systems) allows theuser to quickly calculate grids, trees, loops, fixed water spray systems, and any other arrangement of piping. for any arrangement, you can develop a piping layout, readily adjust pipe sizes to optimize the design, feed piping in one area from piping in another area, and calculate either minimum end head flow or the total supply available.

For more information, contact: HRS Systems, Inc., 2193 Ranchwood Dr., NE, Atlanta, GA 30345 (404) 934-8423; fax (404) 934-7696.

Coating Thickness Gauge

The Elcometer 345 is a truly shirt pocket-sized coating thickness gauge. It measures any non-magnetic coating on steel or all non-conductive coatings on non-ferrous substrates with a built-in or separate probe. Sizes range from a four-key keyboard with no record keeping to a nine-key keyboard with full statistical analysis and memory for up to 10,000 readings. Most models have a built-in RS232 output, enabling data to be sent to a data logger, printer or PC. The 300S model has a number of advanced features, such as batch analysis, high-low limits, large memory (8K), a wide range of statistics, and a variety of calibration methods (including single point and two point).

For more information, contact: Elcometer Inc., 1893 Rochester Industrial Dr., Rochester Hills, MI 48309 (313) 650-0500; fax (313) 650-0501.

Fire Protection Software

STEMFIRE, from AISC Software, determines safe and economical fire protection for steel beams, columns and trusses. It is intended for use by architects, engineers, building code, fire officials and others interested in steel building fire protection. STEMFIRE is based on rational procedures developed by AISI that extends the published UL fire resistive designs to other possible rolled structural shapes and common protection material requirements. For a required fire rating, STEMFIRE determines the minimum spay-on thickness for various rolled shapes as well as the ceiling membrane or envelope protection for trusses. This methodology is recognized by UL and has been adopted by the three national model building codes. The software database contains all pertinent steel shapes and many listed UL Fire Resistance Directory construction details and fire ratings. In this manner, user search time is minimized and the design and checking of steel fire protection is optimized. (Available only on 5-1/4 inch disk.)

To order a copy, send $96 (NY, CA, and IL residents add sales tax) to: American Institute of Steel Construction, Software Sales, P.O. Box 806276, Chicago, IL 60680-4124 or call (312) 670-2400.

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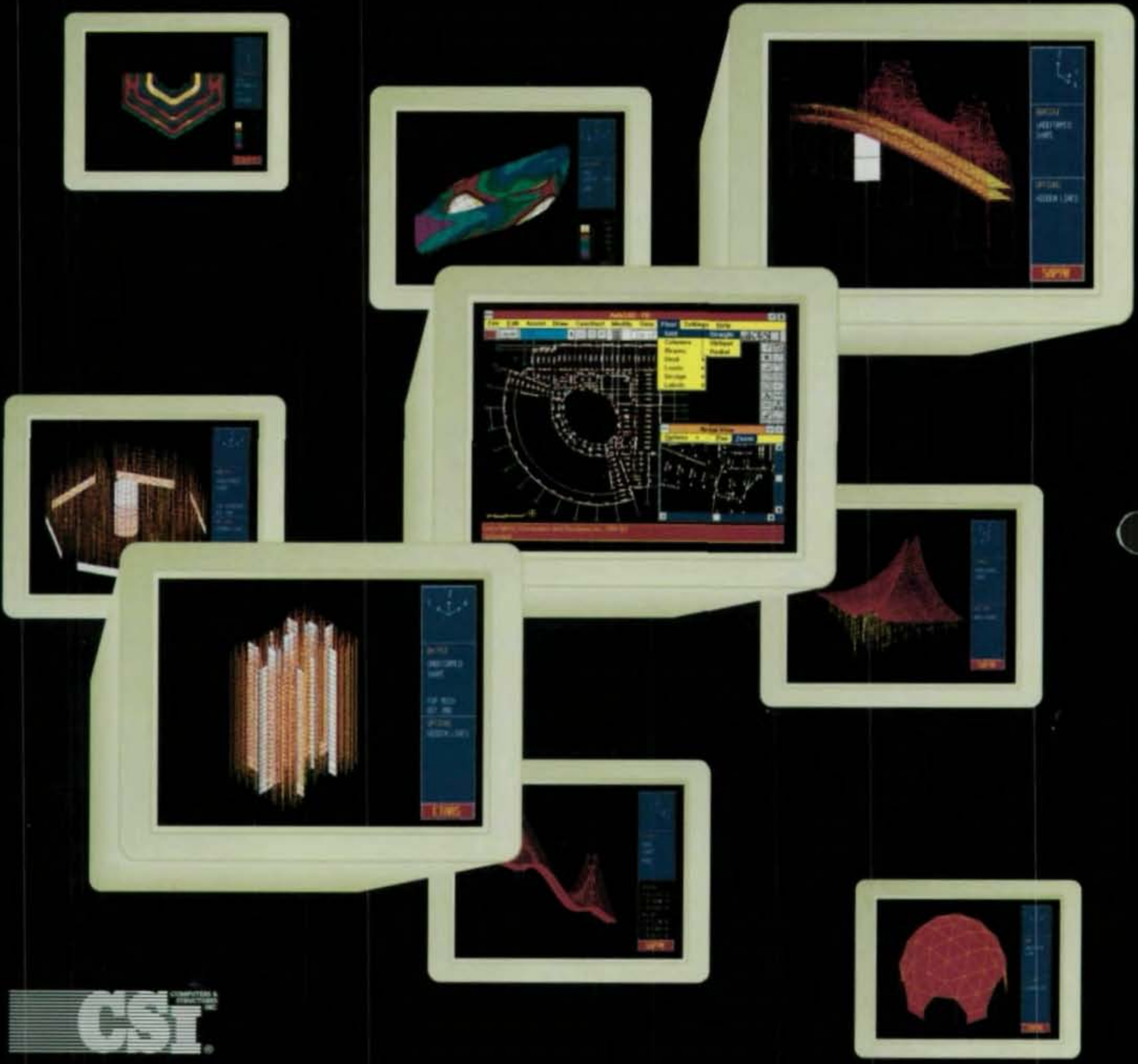

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