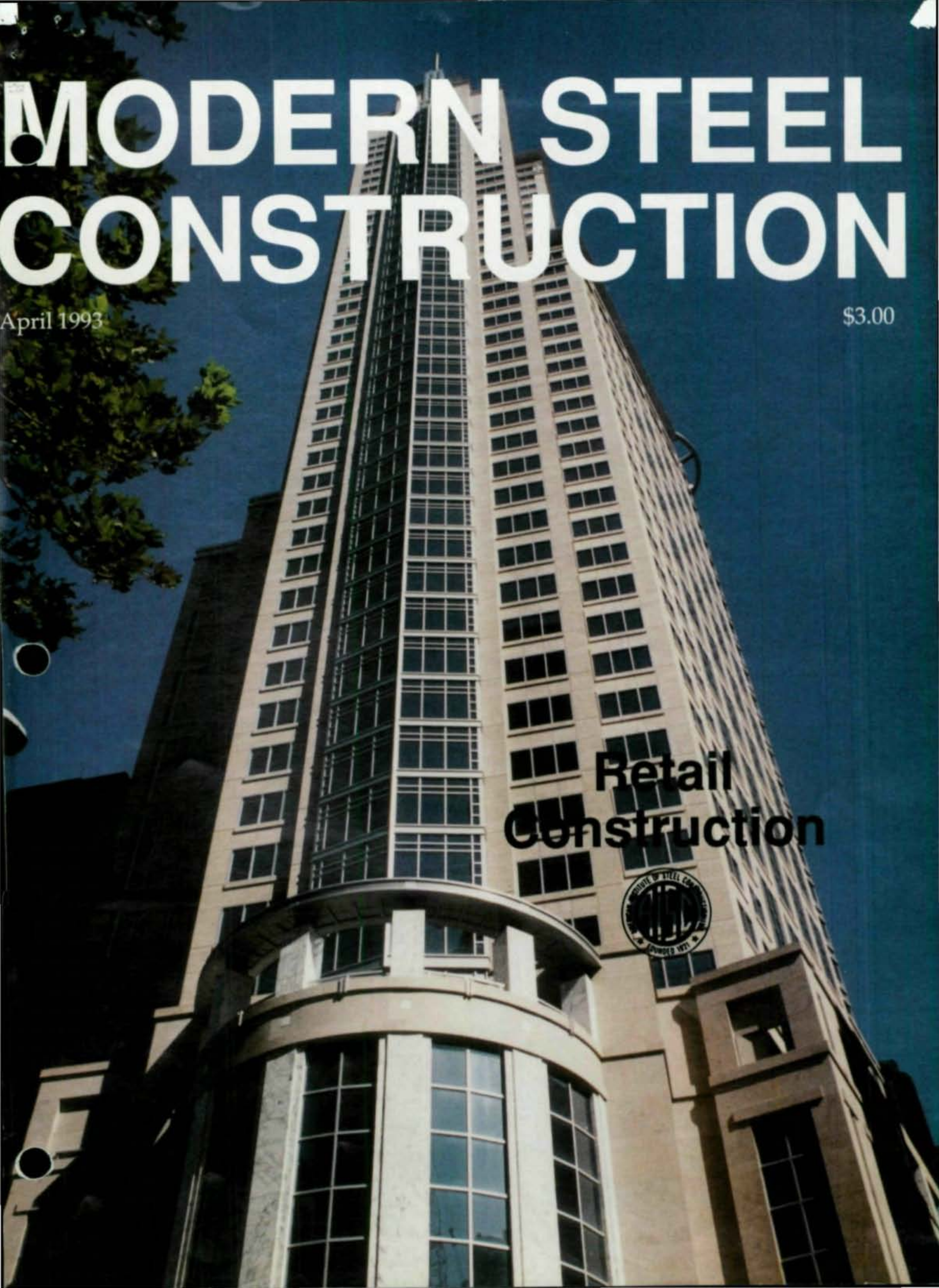
MODERN STEEL CONSTRUCTION

April 1993

$3.00



Retail Construction

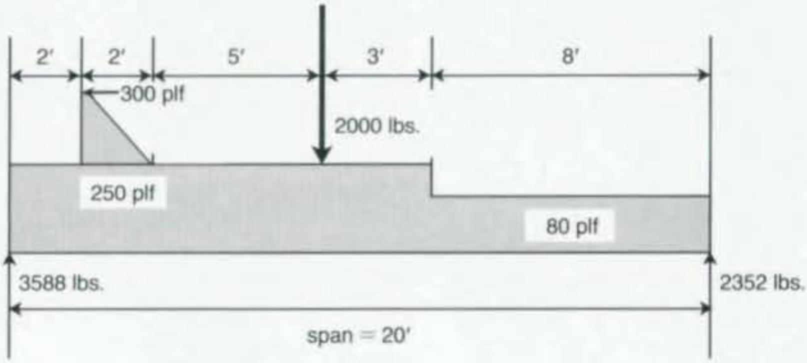
NEW COLUMBIA JOIST CO.

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

JOIST DESIGN DATA SHEET
No. 1

HOW TO SPECIFY SPECIAL JOISTS

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



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

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

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

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

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

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

Grade School.

Introducing the one steel for North America: Chaparral Steel's A36/A57250 steel. This mill-certified steel satisfies multigrade requirements in the U.S., Canada and Mexico. The A36/A57250 steel meets all the specifications of our A36, A572 Grade 50, 44W and 50W. It's everything you like about these grades rolled into one. Plus, it costs the same as the A36, and has the same carbon equivalent range, an important factor for welding and formability. It's also just as easy to get, thanks to our innovative shipping techniques and central location. Call your Chaparral representative today and order the structural steel that makes the grade a number of ways: Chaparral's A36/A57250.

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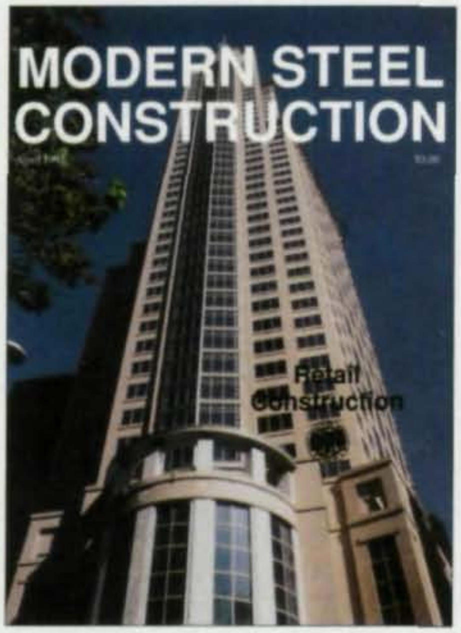
MODERN STEEL CONSTRUCTION

Volume 33, Number 4 April 1993



Rather than build another box-like structure, the developer of The Chifley Square Building in Sydney favored a highly articulated building such as those popular today in the United States. For more information on this project, turn to page 34.

FEATURES

NEWS AND DEPARTMENTS

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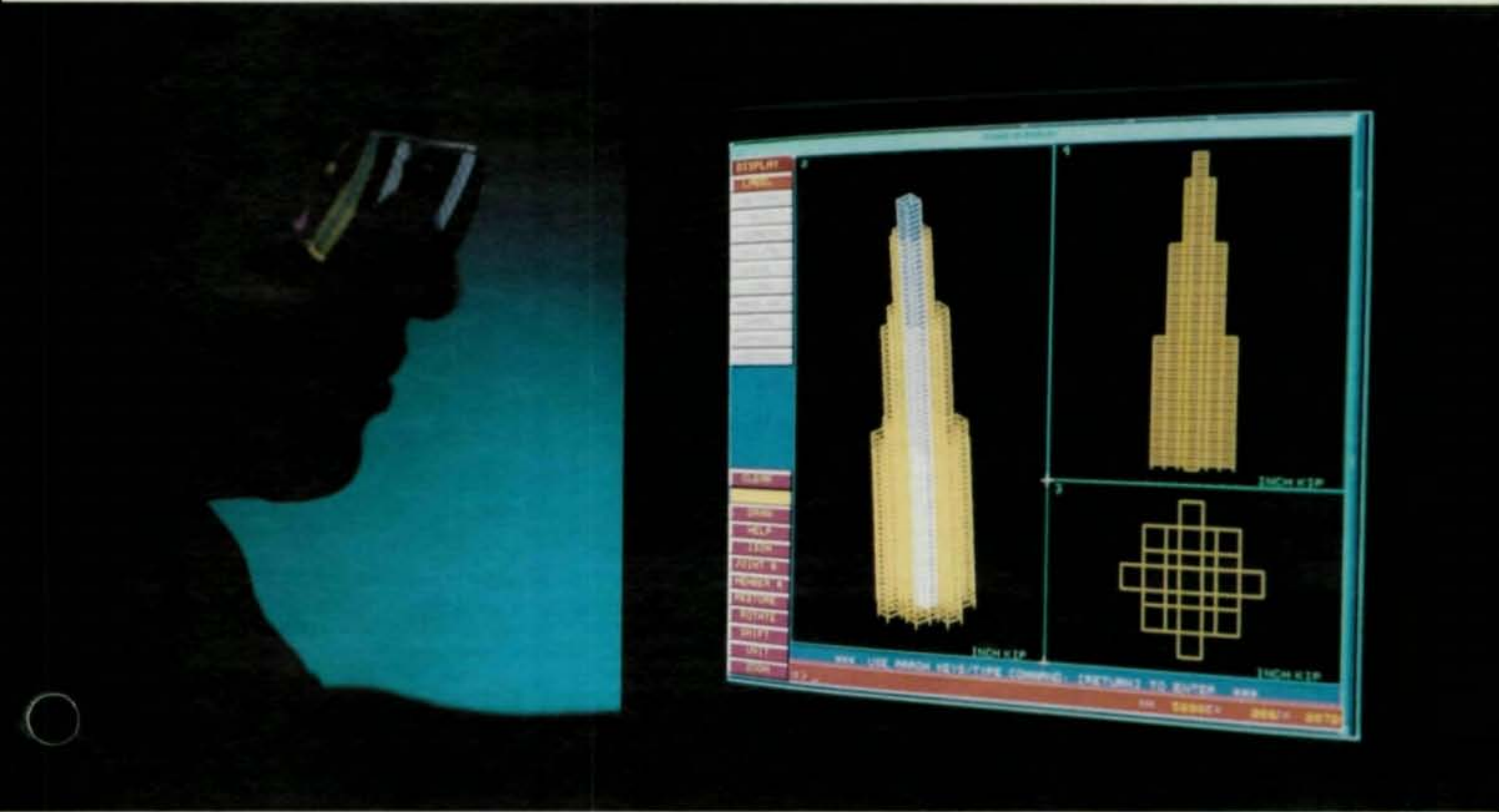


Endless Cycle

Even though one National Steel Construction Conference has just concluded, it's not too early to start thinking about next year's event. Specifically, AISC is now soliciting papers to be presented at the 1994 NSCC in Pittsburgh on May 18-20.

Unlike this magazine, which concentrates on successful projects, the Steel Conference looks for innovative design, fabrication and erection techniques. As Patrick Newman, AISC Senior Staff Engineer, states: "After attending a session, you should be able to take home either a time or cost saving technique. The idea behind the conference is to provide practical information that a designer or fabricator can readily use in his or her office." Of course, papers can use projects to illustrate the technique.

For example, at this year's conference, several engineers from Skidmore, Owings & Merrill presented a paper on a Barcelona hotel that demonstrated several advances in fire protection for exposed steel structures as well as an interesting application of exoskeleton design. Another interesting paper, by Lawrence A. Kloiber of LeJuene Steel Co. presented practical information on the design of tube connections. And Mulach Steel presented a paper on reducing parking structure costs.

Topics of general interest include—but are not limited to—seismic design, composite construction, LRFD, heavy framing, fire protection, computer aided design and detailing, residential construction systems, minimizing floor-to-floor height with steel construction, eccentric-braced framing, and semi-rigid connections.

If you've developed any innovative design concepts and are interested in presenting a paper in 1994, send a one-page abstract to: Patrick Newman, Senior Structural Engineer, American Institute of Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001. Or if you have any questions, call him at (312) 670-5417 (he's also looking for suggestions on improvements in the conference that you'd like to see made). Deadline for submission is July 15, 1993. **SM**

P.S. For more information on this year's show, check out the March 1993 issue as well as next month's magazine.


TO MAKE SURE OUR TENSION
CONTROL SYSTEMS
OUTPERFORM THE COMPETITION,
WE PUT A HEX ON THEM.


A325

Steel Interchange

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

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

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

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

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

A quick equation for calculating Rigid Frame Displacement from Lateral Loads without resorting to a computer:

It is sometimes necessary to rapidly calculate a rigid frame displacement from the lateral load without the computer application.

Simple theoretical formulas are presented below; they were derived using a moment-area theorem for symmetrical rectangular frames.

Nomenclature:

h = frame height
I_{BM} = moment of inertia of beam
I_{COL} = moment of inertia of column
L = frame span
P = lateral load
E = modulus of elasticity
Δ = displacement

For two-hinged frame (Figure 1):

$$\Delta = \frac{Ph^2}{6E}\left(\frac{h}{I_{COL}} + \frac{L}{2I_{BM}}\right)$$

For fully fixed frame (Figure 2):

$$\Delta = \frac{Ph^3}{12EI_{COL}}\left(\frac{3K+2}{6K+1}\right)$$

$$K = \left(\frac{I_{BM}}{I_{COL}}\right)\left(\frac{h}{l}\right)$$

These simple formulas could be very useful even in the age of computers. The book, *Rigid Frame Formulas* by A. Kleinlogel is a very good reference.

Rudolf J. Budesky
City of San Francisco
San Francisco, CA

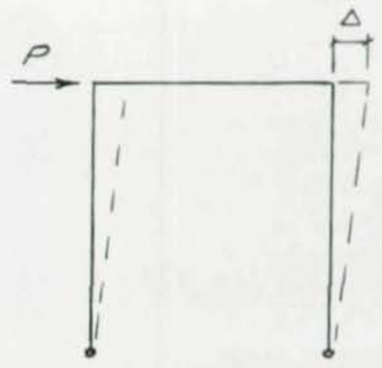


Fig.1 Two-hinged frame

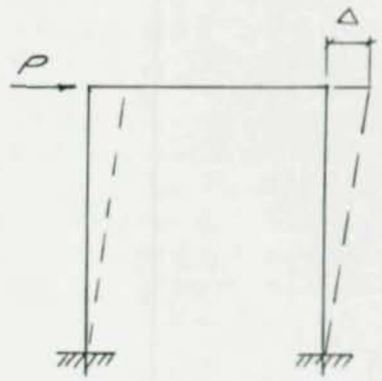


Fig.2 Fully fixed frame

Steel Interchange

New Questions

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

Connection Holes

Does anyone have any information pertaining to the acceptability of burning connection holes in connection material using an automated burning device? AISC only addresses the burning of short and long slots and indicates that it is acceptable with proper cleanup.

Mike Pentecost
Roscoe Steel and Culvert Co.
Billings, MT

Weld Considerations

In developing the strength of a connection or the required length of weld to develop the member strength, the welds "a" and "b" were first considered (see diagram at right). These welds were parallel to the load. Then as the codes and research continued weld "c" was included, thus increasing the capacity of the member. Later weld "d" was introduced.

However, in the current AISC design criteria (Allowable Stress Design and Load and Resistance Factor Design) this weld has not been addressed. Upon questioning AISC, I was referred to the American Welding Society Code. Upon examination weld "d" was omitted here as well. Additionally, my review of current materials on this particular subject has yielded no further information.

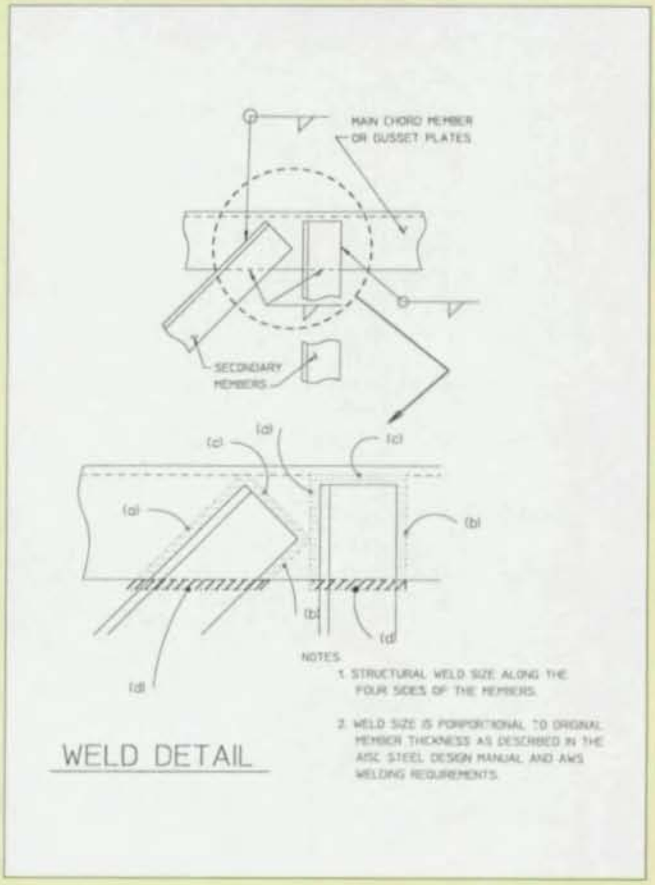
Questions:

A. Can weld "d" be used to develop or determine the strength of the connection (member)?

B. If weld "d" is used to develop the strength of the connection, are there restrictions or parameters that must be placed on the weld (i.e. placement, size, etc.)?

C. Does weld "d" take away from the joint efficiency (strength)? With "d" there is a concentrated area of weld, inducing large amounts of heat because of the welding process. Because of this introduction of heat, material properties are altered resulting in members which are less ductile than ASTM A36 steel.

James R. Seale, P.E.
EIMCO P.E.C.
Salt Lake City, UT



Steel Joists Make It Easier To Build Well For Less

To make building design more efficient, the Steel Joist Institute has provided three new tools: a computer diskette to determine vibration characteristics, the SJI 60-Year Steel Joist Manual, and a Catalogue of Standard Specifications and Load Tables.

The Steel Joist Institute (SJI) has created a computer program to assist the qualified professional engineer in determining probable vibration characteristics of floor systems using open web steel joists. This program is designed for use in conjunction with the institute's Technical Digest #5 "Vibration of Steel Joist-Concrete Slab Floors."

This program allows the designer to calculate swiftly and easily the frequency and amplitude resulting from transient vibration caused by human activity on a joist-concrete floor. The "what if?" scenario—variations in slab thickness, concrete strength, joist size, joist spacing, floor decking, live and dead loads, span lengths—can be accomplished in seconds.

The program is user friendly, can handle spans up to 100 ft. and can accomplish in seconds calculations that previously required several hours. It is available on 5¼ and 3½-in. disks and is IBM PC compatible. A comprehensive user's manual is included.

Long-span joists provide broad, column-free expanses like the sanctuary of this Charlotte, N.C., church.

Steel joists, steel girders and steel deck make shopping center construction fast and economical.

The SJI 60-Year Steel Joist Manual is also now available. The new, 318-page 60-Year Manual replaces the 50-Year Digest and features 98 more pages of information. The practical section in the Manual is designed to aid the professional by listing four helpful categories:

- The various building documents required and what use they can be.
- Building site information and equipment needed.
- Step-by-step investigative procedures.
- Time-saving data for use when analyzing existing structures.

Another helpful reference that's now available is the SJI 1992 Catalogue of Specifications and Load Tables. All of the 1992 revisions are prominently listed so that specifiers can review these changes quickly and easily.

The section of fire-resistive assemblies has been expanded and completely revised. It lists the requisite criteria for using K-series joists in an assembly and includes a simple, five-step procedure for selecting the proper and most economical joist. In addition, the catalogue contains over 75 floor and roof assemblies listed in an easy to use chart for quick reference, with specific UL designations for fire ratings from one to four hours.

Last year, Underwriters Laboratories, Inc. increased the allowable design stress of fire-rated steel joists by 36% for floors and 18% for roofs. The allowable tensile stress of joists used in most fire-rated assemblies has been increased to 30,000 psi for floors and 26,000 psi for roofs, as compared with the previous maximum stress level of 22,000 psi. The new standards now make it more economical to achieve desired fire resistance ratings without added expense for heavier joists.

For information contact the Steel Joist Institute, 1205 46th Ave. North, Myrtle Beach, SC 29577.

Steel Joist Institute Members:

Canam Steel Corp.
Delong's Inc.
East Coast Steel
Gooder-Henrichsen
John W. Hancock, Jr., Inc.
The New Columbia Joist Co.
Owen Joist Corp.
Quincy Joist Co.
SMI Joist Co.—A Commercial Metals Co.
Socar Inc.
Valley Joist Div./EBSCO Industries
Vulcraft Div./Nucor Corp.

Book Review: *Connections in Steel Structures II: Behavior, Strength, and Design*

By Charlie Carter

Connections in Steel Structures II: Behavior, Strength, and Design is the Proceedings of the Second International Workshop on Connections in Steel Structures, held in April 1991 in Pittsburgh. Sponsored by the European Convention for Constructional Steelwork, the U.S. National Science Foundation, the University of Pittsburgh, and the American Institute of Steel Construction, the workshop provided a forum for the assessment of ongoing connections research, a review of current developments in connections, and the identification of needed future connections research.

The invited participants at this conference were internationally recognized experts in the field of steel connections. Whereas a textbook would attempt to address the widest range of topics possible, the proceedings focuses on the latest research and developments in several, very specific topics. Furthermore, the international flavor of this publication is unmistakable with authors from the United States, Canada, Mexico, several countries throughout Europe, the former Soviet Union, and Australia. The resulting text provides the reader with both a knowledge of work performed in other countries and insight into how this work compares with current practice in the reader's country.

In all, fifty papers by 77 authors and co-authors were given and can be categorized in these general topics: bolts; welds and local strength considerations; predesigned and special connections; composite connections; semi-rigid connections; available connections software; global behavior of semi-rigid connections; examples of frame design; and economy of design. A final segment documents the current research needs.

Its variety of topics results in a very thorough reference on recent research and current developments in steel connections. *Connections in Steel Structures II: Behavior, Strength, and Design* (Pub. #G455) is available from AISC for $44 plus $5 shipping and handling. Call (312) 670-2400 ext. 433.

Bolts—The four papers presented on this topic addressed three specific areas. In *Differences Between European and American Design Rules*, the AISC LRFD Specification and Eurocode 3 are contrasted and the author proposes suggestions for the resolution of differences which exist. Verification of Quality Assurance on European 4.6 and 8.8-Bolts and *Bolt Preloads in Laboratory and in Field: Conditions of Acceptance* both address quality assurance issues. The former presents the results of random sample testing verification of European 4.6 (A307) and 8.8 (A325) bolts. In this paper it was found that 8.8 bolts were completely satisfactory, while precautionary measures were recommended for 4.6 bolts. In the latter paper, the quality assurance performance of various bolt tightening methods was investigated. The final paper in this segment, *Use of Snug-Tightened Bolts in End-Plate Connections*, presents tests results and recommendations for use.

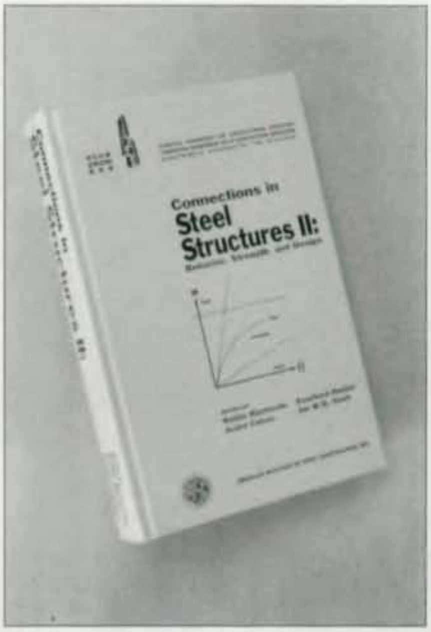
Welds and Local Strength Requirements—Five papers were presented on this topic. *Influence of Base and Weld Metal Strength on the Strength of Welds* explores the validity of basing design weld strength predictions on the mean value of the strength of the base and weld metals. *Forces in Beam-to-Column Connections* presents an alternative approach to the determination of forces in semi-rigid and rigid beam-to-column connections. A simplified design approach to concentrated forces at beam-to-column connections is proposed in *Plastic Analysis and Simplified Design of the Compression Zone of a Beam-to-Column Connection*. The final paper in this topic, *Review of International Design Criteria for Fillet Welds in Hollow Structural Section Truss Connections* contrasts the approaches to weld proportioning in hollow section trusses in the United States, Europe, and Canada and preferred design approaches are proposed.

Predesigned and Special Connections—Seven papers were presented on this topic. New developments in single plate and tee shear connections, stiffened seated shear connections, and bracing connections are summarized in *Recent Developments in Connection Research and Design in the USA*. A review of the evolution of AISC design aids is made in *Bolted Framing Angle Connections Design Aids: Past and Present*. In *Simple Beam-to-Column Connections*, four simple connection commonly used in British practice are reviewed with general guidelines and recommendations for ensuring rotational flexibility and stability. The paper *A Design Approach for Semi-Rigid Connections in Cold-Formed Steel Industrial Racks* describes current design procedures based on the recommendations of the Rack Manufacturers Institute. *Welded Hollow Section Connections Under Predominantly Static Loading* presents design recommendations for hollow section joints in lattice structures (trusses). The effect of deformations on the soil-footing interface on the rotational resistance characteristics of a foundation support is the subject of *Steel Baseplate-Footing-Soil Behavior*.

Composite Connections—Eight papers were presented on this topic. *Semi-Continuous Composite Frames in Eurocode 4* explains the approach for semi-rigid composite frames taken in Eurocode 4 and identifies the difficulties encountered and needed research. In *Parametric Study of Composite Frames*, the behavior of semi-rigid composite frames is compared favorably with that of similar rigid frames to show that the use of semi-rigid composite construction is valid. The force transfer mechanisms involved in semi-rigid composite connections is investigated in *Slab and Beam Load Introduction in Composite Columns*, while a model for analysis is proposed in *Energy-Based Prediction for Composite Joints Modeling*. The ability of composite connections to meet the necessary performance criteria is assessed in *Tests on Composite Connections*. Rotational behavior and the outcome of cyclical testing is presented in *Semi-Rigid Composite Joints: Experimental Studies*. The non-linear behavior of this connection is further described in *The NonLinear Behavior of Composite Joints*. Finally, the effect of cyclical loading is further explored in *Cyclic Load Analysis of Composite Connection Subassemblages*.

Semi-Rigid Connections—Four papers were presented in this topic. *Reliability of Rotational Behavior of Framing Connections* addresses the statistical variations caused by fabrication and erection tolerances. Refinements to existing methods are presented in *Plastic Capacity of End-Plate and Flange Cleated Connections: Predictions and Design Rules*. The effect of cyclical loading is examined in *Analysis of Flexibly Connected Frames Under Non-Proportional Loading*. In *Moment-Rotation Characteristics of Bolted Connections*, the effect of column flexibility is considered for bolted semi-rigid connections.

Connections Software—Three papers, describing connection design software, were presented. The Eureka "CIMSTEEL" project, a European approach, was described in one paper, while a second described AISC's expert system,



CONXPRT. *A CAD System for Semi-Rigid Joints in Non-Sway Steel Frames* elaborates on the analysis method, design approach, and features of this limit-states based program.

Global Behavior of Semi-Rigid Connections—Eight papers were presented. *Connection Moment-Rotation Curves for Semi-Rigid Frame Design* examines the use of actual versus simplified representations of the connection characteristics. The relationship between joint flexibility and frame stability is investigated in *Connection Response and Stability of Steel Frames*. In addition to this the effect of force distribution and deformations are examined in *Prediction of the Influence of Connection Behavior on the Strength, Deformation, and Stability of Frames, by Classification of Connections*. The semi-rigid nature of joints in hollow sections is investigated in *Semi-Rigid Connections in Lattice Girders Formed with Hollow Structural Sections*. The results of a study of seismic behavior of rigid and semi-rigid frames is presented in *Cyclic Behavior of Frames with Semi-Rigid Connections*. A similar study is presented in *Dynamic Tests of Semi-Rigid Connections*. Finally, an analytical modelling approach is described and interpreted in *Analytical Modeling of Cyclic Behavior of Bolted Semi-Rigid Connections*.

Examples of Frame Design—This topic contains six papers. Proposed design recommendations are summarized in *Behavior of Semi-Rigid Connections and Implementation in Frame Design*. The use of LRFD in semi-rigid frames is demonstrated in *Design Analysis of Semi-Rigid Frames with LRFD*. A simplified approach allowing the inclusion of flexibility effects on frame design is presented in *Practical Design Allowing for Semi-Rigid Connections*. *The Effect of Connection Flexibility on Portal Frame Behavior* examines both full scale testing and analytical modeling in flexibly connected portal frames. The effect of static and dynamic loadings on frame stability is examined in *Analysis of Frames for Stability*. *An Analytical Analysis and Design System for Steel Frames with Partially Restrained Connections* describes a method for modelling semi-rigid beam-to-column connections in steel frames.

Economy of Design—Four papers were presented. In *Economy of Semi-Rigid Frame Design*, the cost benefit of using semi-rigid connections is evidenced. The cost benefit of the "temporary" semi-rigid frames used in a 30-story building in Mexico City is described in *Temporary Flexible Connections in the Construction Process*. For bracing connections, a cost comparison of different design methods is presented in *A Cost Comparison of Some Methods for Design of Bracing Connections*. Finally, the use of cold-formed members and semi-rigid columns is investigated in *Semi-Rigid Lightweight Steel Frame SKELETON*.

Current Research Needs for Connections in Steel Structures—The final segment of this publication presents the current research needs for connections in steel structures. These needs were developed from the technical papers presented and the ensuing discussions.

Charlie Carter is Staff Engineer—Structures with AISC.

AISC Lecture Series: New Ideas In Structural Steel

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

The four-part seminar covers:

- Low-rise buildings.
- Design of connections.
- Eccentric braced frames.
- Partially restrained connections.

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

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

Steel Bridge Forum (April 1 in Baltimore) sponsored by American Iron & Steel Institute. Topics include: Guide to Selection of Bearings and Short Span Steel Bridges. Contact: AISI, 1101 17th St., N.W., Suite 1300, Washington, DC 20036-4700 (202) 452-7100; Fax (202) 463-6573.

Transportation Planning for Livable Communities (April 2-3 in San Francisco, April 23-24 in Atlanta, and April 30-May 1 in Winter Park, FL) sponsored by FHWA, National Trust for Historic Preservation, and others. Workshops explain the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA). For information, call (800) 937-6847.

Workshops for Rural and Small Urban Officials (April 6-7 in Burlington, VT, and May 18-19 in Hershey, PA) sponsored by National Association of Counties, U.S. Dept. of Agriculture, NACE, and USDOT. Sessions will focus on new opportunities for roads and bridges, rural transit, economic development and research and technical assistance. For information, call: (202) 720-8042.

Steel Bridge Forum (April 22 in Raleigh, NC) sponsored by American Iron & Steel Institute. Topics include: Overview of Steel Service Centers; Skewed & Curved Bridges; Short Span Steel Bridges; and Guide to Selection of Bearings. Contact: AISI, 1101 17th St., N.W., Suite 1300, Washington, DC 20036-4700 (202) 452-7100; Fax (202) 463-6573.

Symposium on Project Management (May 13-14 in Chicago) sponsored by Association for Project Managers in the Design Professions. Topics include: state of the art project management techniques; the emerging concept of partnering; creating a Total Quality Management component to each design project; making effective project decisions; new project management directions for the economy of the '90s; and case studies including managing quality on the new International Complex at O'Hare Airport. Contact: APM at (312) 472-1777; Fax (312) 525-0444.

Lasers in Fabricating Conference (May 18-20 in Schaumburg, IL) sponsored by the Fabricators & Manufacturers Association. Topics include: equipment selection and cost justification; precision laser cutting techniques; YAG, fiber-delivered processing; laser/punch combination machines; high-energy thermal processing; programming laser cutting systems; CO^2 multiaxis processing; material handling equipment; equipment specifications; maintenance; and safety. For more information, call (815) 227-8202.

1993 Symposium on Computer Integrated Building Sciences (June 10-11 in Anaheim, CA) sponsored by the International Council for Building Research and Documentation. Topics will include: automated construction (using no people); automated fabrication; robotic tools for construction; and 3D modeling. An additional session will feature the new Disney Concert Hall Project, which has been designed on 3D CAD and has used NC for the cutting of stone and structural steel. For more information, contact: Harold Jones, SCIBS'93, 1700 Asp Avenue, Norman, OK 73037-0001 (405) 325-1947; Fax: (405) 325-7968.

STEEL NEWS

Correspondence

Dear Editor:

In the January, 1993 issue of *Modern Steel Construction* you published a list of various structural steel shapes and the principal producers of these shapes. What is not clear, however, is whether all shapes listed are available in A36 and A572 Grade 50 materials. It has been my understanding that A572 Grade 50 material is available for W shapes only.

M. Garkawe
Director, Contract Design Dept.
Foster Wheeler Energy Corp.
Clinton, NJ

Editor's Response: Most structural shapes, including angles and channels are available in A36, A572 and A588. In addition, some mills offer special grades for bridge design.

An updated version of the Structural Shapes Availability List will appear in the July 1993, followed by regular updates in each January and July issue.

Clarifications

The phone number for EJE Industries was incorrectly listed in the February issue of MSC. The correct phone number is (800) 321-3955. We regret the error.

The following chart should have been included on page 16 of the December 1992 issue:

Section	LP	LO	P	e
	(ft)	(ft)	(kips)	(in)
W36x300	96	75	464	11.37
W33x152	69	48	350	10.25
W16x40	24	15	119	2.26

In the above table, LP is the maximum span with prestressing, LO is the maximum without prestressing, P is the prestressing force, and e is the eccentricity.

Atlanta To Host Steel Bridge Symposium

The National Symposium on Steel Bridge Construction is scheduled for Nov. 10-12 in Atlanta. Presented under the auspices of the Council for the Advancement of Steel Bridge Technology, the symposium is intended to create a dialogue between owners, designers, and builders. Co-sponsors include AISC, FHWA, AASHTO, and AISI.

Workshop sessions will kick-off the symposium on November 10. Anticipated topics include: shop and field painting; relevant EPA requirements; and economical and innovative steel bridge design and construction techniques.

Following the workshops will be technical sessions featuring such topics as: performance reports on weathering steel; horizontally curved bridges; bridge reports on major structures; innovations in steel design; and workshop reports.

In addition, a special Student Bridge Competition demonstration will be held. (The Student Bridge Competition is a series of regional events where teams of engineering students from approximately 100 schools with civil engineering programs compete to design and construct a 20' working scale model. The students are judged on the weight of the bridge, its load-carrying capacity, aesthetics, and speed of erection.)

The winners of the 1993 Prize Bridge Awards will be honored at a November 11 dinner banquet. A slide presentation will highlight the winning projects. An entry form for the Awards program appears on pages 24 and 25 of this magazine.

For more information on the Prize Bridge Symposium, contact the AISC Membership Services Dept. at (312) 670-5420; Fax (312) 670-5403.

1993 Prize Bridge Jury Named

Entries are now being accepted for the 1993 Prize Bridge Awards. To be eligible, a bridge must be located within the U.S. or its territories, be built of fabricated structural steel, and have been completed and opened to traffic between May 1, 1988 and April 30, 1993. Bridges are judged based on aesthetics, economics, and design/engineering solutions.

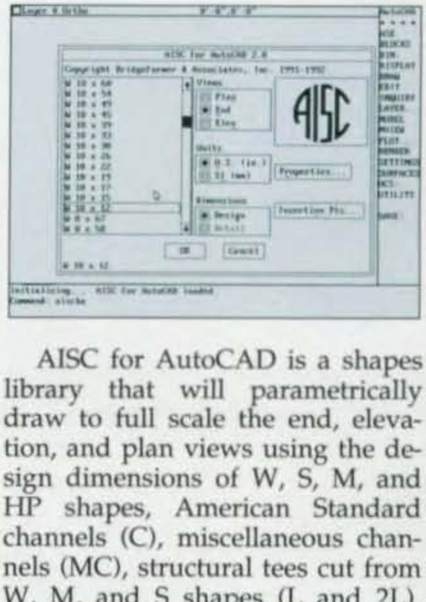
Categories are: long span; medium span, high clearance; medium span, low clearance; short span; grade separation; elevated highway or viaduct; movable span; railroad; special purpose; and reconstructed. Deadline for entry is June 18, 1993, and there is no entry fee. A complete entry form appears on pages 24-25 of this issue.

This year's jury is comprised of: Frederick Gottemoeller, a consultant with Frederick Gottemoeller & Associates; James McCarty, president of ASCE and a consultant; James Powers, president of Envirodyne Engineers; and Joseph Siccardi, a staff bridge engineer with the Colorado Dept. of Highways.

For more information, contact: Christy Depkon, AISC Director of Public Affairs, AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001 (312) 670-5432; Fax (312) 670-5403.

New Version Of *AISC For AutoCAD*

AutoCAD Release 12 users can now save even more time doing detail drawings with Version 2.0 of AISC for AutoCAD. The latest release offers several enhancements, including: a more intuitive dialogue box user interface with point-and-click list box shape selection; metric support; optional insertion point selections; and overall increased speed. Version 2.0 takes full advantage of the improvements made to AutoCAD in Release 12.



AISC for AutoCAD is a shapes library that will parametrically draw to full scale the end, elevation, and plan views using the design dimensions of W, S, M, and HP shapes, American Standard channels (C), miscellaneous channels (MC), structural tees cut from W, M, and S shapes (L and 2L), structural tubing (TS), and pipe. The shapes correspond to data published in Part 1 of both the 1st edition *LRFD Manual of Steel Construction* and the 9th edition *ASD Manual of Steel Construction*.

Version 2.0 runs only in AutoCAD Release 12 and above. Version 1.0 is still available for AutoCAD Release 10 and 11 users. The cost for either version is $120.

For ordering information, contact: AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001 (312) 670-2400; Fax (312) 670-5403.

Design Guide On W-Shapes

The sixth AISC Design Guide, "Load and Resistance Factor Design of W-Shapes Encased in Concrete (D806)" is now available. The publication, which was authored by Lawrence G. Griffis, P.E., of Walter P. Moore Associates, Inc., covers composite columns comprised of rolled wide flange shapes encased in reinforced structural concrete with vertical deformed reinforcing bars and lateral ties.

Part One covers composite frame construction, reviews important practical considerations, and presents pertinent design criteria. A set of suggested design details in given in Part Two. Part Three includes five examples, while Part Four contains comprehensive design tables.

A companion computer program, CMPOL, is available to generate composite column design tables as described in Part Four. The software cost is $80, and includes the Design Guide.

The other five AISC Design Guides previously published are: D801—Column Base Plates (by John DeWolf and David Ricker); D802—Design of Steel and Composite Beams with Web Openings (by David Darwin); D803—Serviceability Design Considerations for Low-Rise Buildings (by James Fisher and Michael West); D804—Extended End-Plate Moment Connections (by Thomas Murray); and D805—Design of Low- and Medium-Rise Steel Buildings (by Horatio Allison). Each design guide costs $16.

For more information or to order a copy, call the AISC Publications Department at (312) 670-2400.

T.R. Higgins Award

Roberto T. Leon, an expert on steel composite connections, has won the 1993 T.R. Higgins Lectureship Award sponsored by AISC. Leon, selected for his papers on semi-rigid composite connections, is an associate professor in the Department of Civil and Mineral Engineering at the University of Minnesota.

Leon gave a presentation based on his papers at the NSCC in March and will give an additional six lectures during 1993 and 1994. Lectures are tentatively scheduled for Oct. 14 in Kansas City and Oct. 15 in Houston.

For more information, contact: Robert F. Lorenz, AISC Director of Education & Training, (312) 670-2400; Fax (312) 670-5403.

Innovative Design Cuts Costs

The use of composite construction with partially restrained connections reduced framing costs on a 34,000-sq.-ft. retail building by 27.4%

At first glance, the new Mt. Kisco Furniture Store in Mt. Kisco, NY, looks like most of the other nearby structures. Closer examination, however, reveals an innovative structural system that minimizes material use.

Up until October 1990, the Mt. Kisco Furniture Store was a thriving retail establishment in downtown Mt. Kisco, NY. Then tragedy struck in the form of a raging fire that destroyed the structure. The owners, the Saroken family, moved quickly to replace the building which had long housed the family business.

"The owners wanted the replacement building to fit into the context of the community, but they also wanted the best looking building on the block," according to Kenneth Nadler, a principal with Nadler Philopena Architects. "We photographed the entire street and our design tries to pick up the scale and proportion of the nearby properties." The result is a limestone facade with classic features such as brass light fixtures. "We wanted the structure to be reminiscent of a turn-of-the-century cast iron building, but with modern touches," he explained. The two-story building houses a Gap store on the first floor and the family furniture business on the second.

But while the architectural design of the building is traditional, the structural design is thoroughly modern.

Typically, a small 34,000-sq.-ft. (3160 m^2), two-story retail store would be designed with simply supported girders. Instead, the engineer, N. Wexler Consulting Engineers, New York City, decided to use composite girders with partially restrained connections (Wexler calls his design a Restrained Girder System, or RGS).

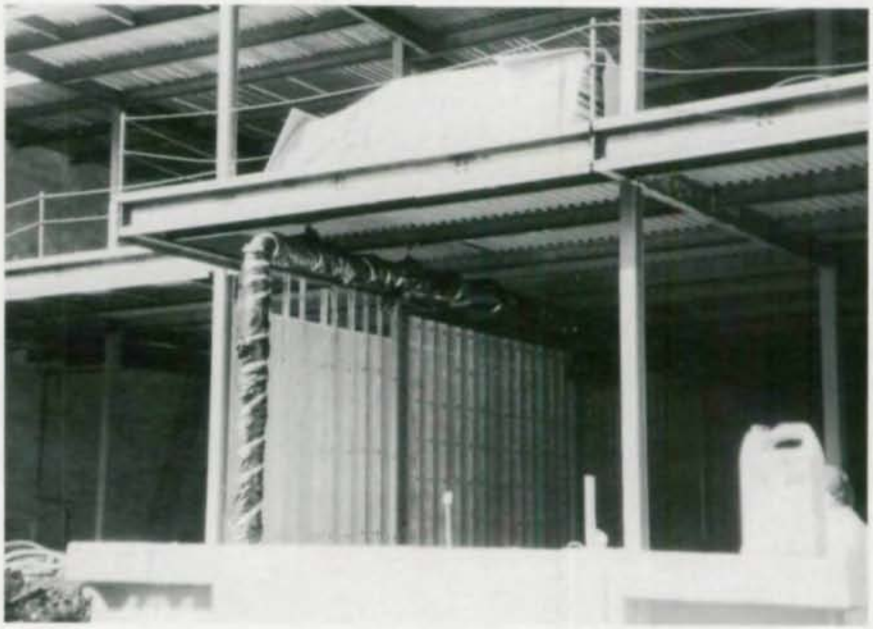
The reason for the design, according to Neil Wexler, P.E., was economics. "Simply supported girders would have been W14x34; with restrained girders, only W14x22 were required," he explained. "The entire project used only 76 tons (68950 kg) of structural steel, resulting in about 4.5 lbs. of steel per sq. ft. (22 kg/m^2)—a very efficient structure."

Wexler has been working with composite girders and partially restrained connections for several years and has recently begun giving presentations on the subject.

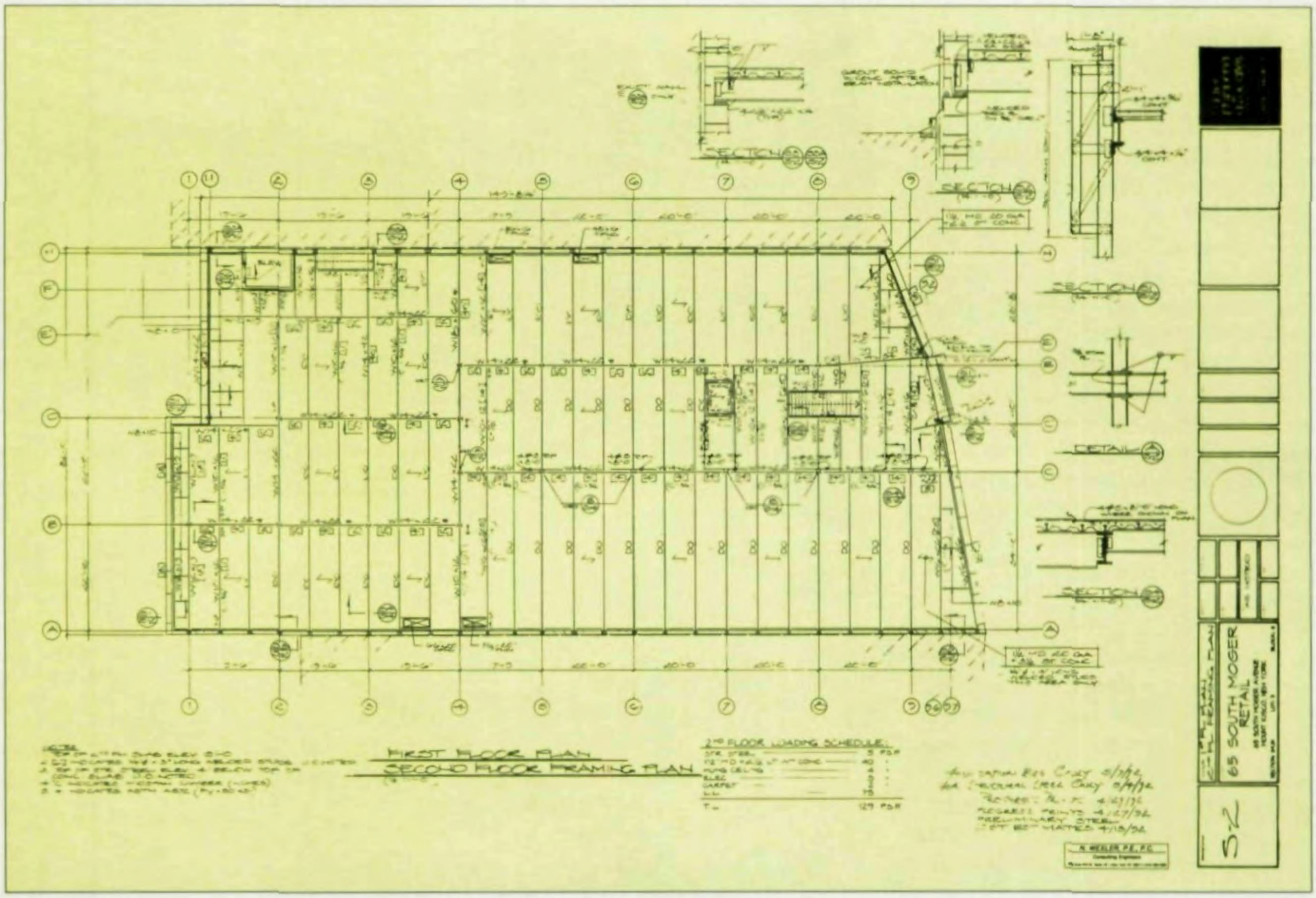
"The traditional design for buildings with steel frames is based on composite girders with simple connections," Wexler writes. "The disadvantage of this traditional design is that the entire moment requirement is at one portion of the girder, resulting in large size girders. Also, girders have large mid-span deflections during construction, when the concrete is wet. In order to eliminate these disadvantages, the designer specified camber or temporary shoring. However, since both of these methods are costly and difficult to implement, contractors often preferred to do without them and instead increased girder sizes even further. With partial restraint connections, girder sizes can be decreased and deflections reduced."

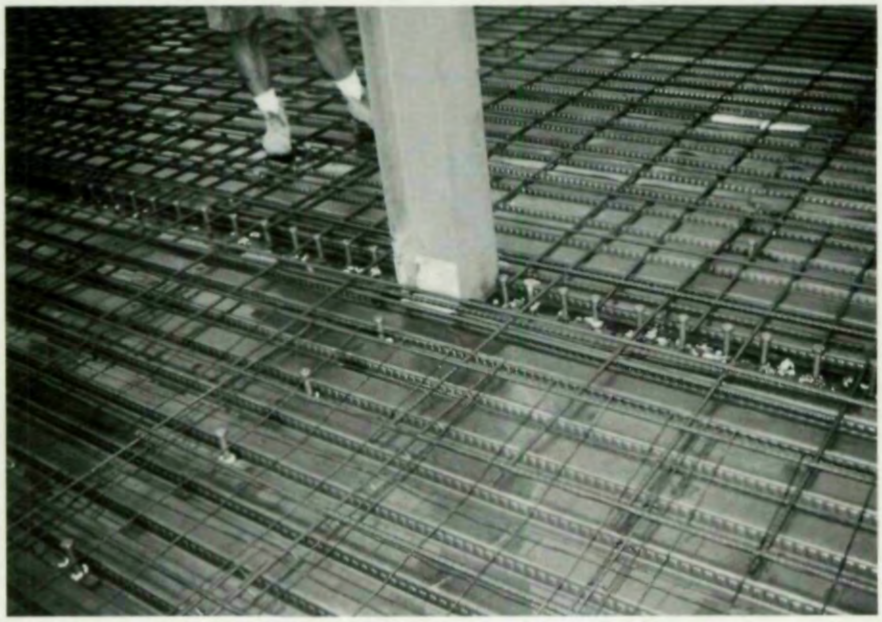
With RGS, two different restraint types are possible, according to Wexler. "When deflections during construction are large, and/or the girder sizes are governed by construction loads, girder-to-column moment connections are preferred. When deflections do not govern, and the girder size is governed by superimposed loads, negative concrete reinforcement bars are preferred." Due to the small calculated deflections during construction, only concrete reinforcement was used for the Mt. Kisco Furniture Store project.

As with composite construction with simple connections, the design of a partially restrained system is done in two phases—a construction phase when the concrete



By using composite construction with partially restrained connections, the project's designer reduced the typical girder size from W14x34 to W14x22. The RGS design also reduced the number of studs required, though it did require the member to be cambered.





is wet and a final phase after the concrete hardens.

During the construction phase, the steel girder alone supports all the loads. "Some steel girders with simple connections have significant mid-span deflections at this phase," Wexler explained. "Introducing moment connections results in reduced mid-span deflections." The reduction can be dramatic. For a beam with fixed connections, the mid-span deflections can be reduced by as much as 58% if only one end is fixed and by 80% if both ends are fixed.

"To provide the rigidity required for this phase, the end moment connection must be designed as a rigid connection (AISC ASD Type 1 Construction)," he added. "During this phase all connection components are stressed elastically. The connection is strong enough to hold the original angles between members unchanged, reducing the mid-span moment and deflections.

During the final phase, the steel girder acts compositely with the concrete. "Once the concrete hardens, superimposed loads such as partitions, mechanical, ceiling and live loads are applied. At this time, the moment at the girder end wants to increase. This increased end moment is restrained by the couple produced by negative concrete reinforcement and the girder bottom flange. When the connection has reached its elastic capacity, it will deform plastically," according to Wexler. "All excess moment "shaken off" by the semi-rigid moment connection is now transferred to the middle section of the girder. Since this middle section is composite with the concrete, it is both strong and rigid. Therefore, any deflections associated with the final phase are small."

Wexler does caution that the design engineer must investigate for each individual project the sensitivity of the system to deflections when the concrete is wet. In some cases, the engineer will need to give recommendations for the concrete pour sequence and the acceptable locations of construction joints.

End Moment Connections

For moderate size moments, Wexler uses an end plate type connection. "It performs well as a rigid connection during the construction phase and as a semi-rigid connection during the final phase," he explained. "An end plate is a particularly good choice because not only does it deliver forces to the column but it also reinforces the column by spreading compression forces over larger areas, just like a bearing plate, thus reducing the need for compression column stiffeners. It is especially economical when full penetration welds are not required."

Another acceptable connection, according to Wexler, are top and bottom angles. "Angles are a good choice because the bottom flange is reinforced against local buckling by the horizontal leg of the angle." For this project, an L8x4x½ was used to enure that the girder bottom flange bears against the column. The project utilized 2½" (64 mm) of light weight concrete poured over 1½" (38 mm) metal deck. At one location, where the span is 34' (10.4 m), the concrete thickness was increased to 5" (127 mm) in order to decrease floor vibrations. Bay sizes are typically 23' x 23' (7.01 m x 7.01 m).

Increasing the connection size beyond that required for full fixity during the construction phase is unnecessary. One way to evaluate a girder with moment connection is by making use of moment connection rotation curves. "With RGS, a composite girder with partial restraint behaves just like a steel girder with full restraint when the concrete is wet," according to Wexler. "After the concrete hardens, and additional loads are superimposed, the connection provides additional restraint until yielding of the reinforcement bars; then the girder behaves just like a simple supported composite girder."

Additional Restraint

Research done at Queens University in Kingston, Ontario (Canada) by Professors Karl Van Dalen and Hernan Godoy reveals that additional moment strength can be achieved at the beam column connection if only 0.46% of the concrete slab area is provided as slab reinforcement, reports Wexler. "This additional strength is at least equal to the ultimate moment capacity of the composite beam and is not influenced by the type of connection between the steel elements. The rotational capacity of the composite beam-column connection also is at least equal to that of a conventional, non-composite rigid steel connection.

Wexler adds that the AISC *Manual of Steel Construction (LRFD)* allows calculations of the negative design moment strength based on plastic stress distribution of the composite section provided that the following are met:

- Shear connectors are located in the negative moment region.
- The slab reinforcement is adequately developed.
- The steel beam is compact and braced.

"The designer can use this additional strength to reduce the girder size further," Wexler said. "Only additional studs and negative concrete reinforcement are needed." He does caution, however, that Van Dalen's research shows that in order to ensure a uniform cracking pattern in the slab in the vicinity of the column that at least twice the minimum area of steel reinforcement be extended on each side of the column centerline.

Other Considerations

Unbalanced loads: While unbalanced loads might overstress noncomposite steel girders with partial restraint, this is not the case with composite girders for most common buildings as long as adequate concrete reinforcement is provided.

Ductility: Ductility is associated with the ability of the joint to rotate after yielding. Joint rotation can be prevented by premature local or overall buckling of the bottom flange and buckling of the web. The use of under-reinforced sections assures adequate post-yielding rotations, Wexler stated.

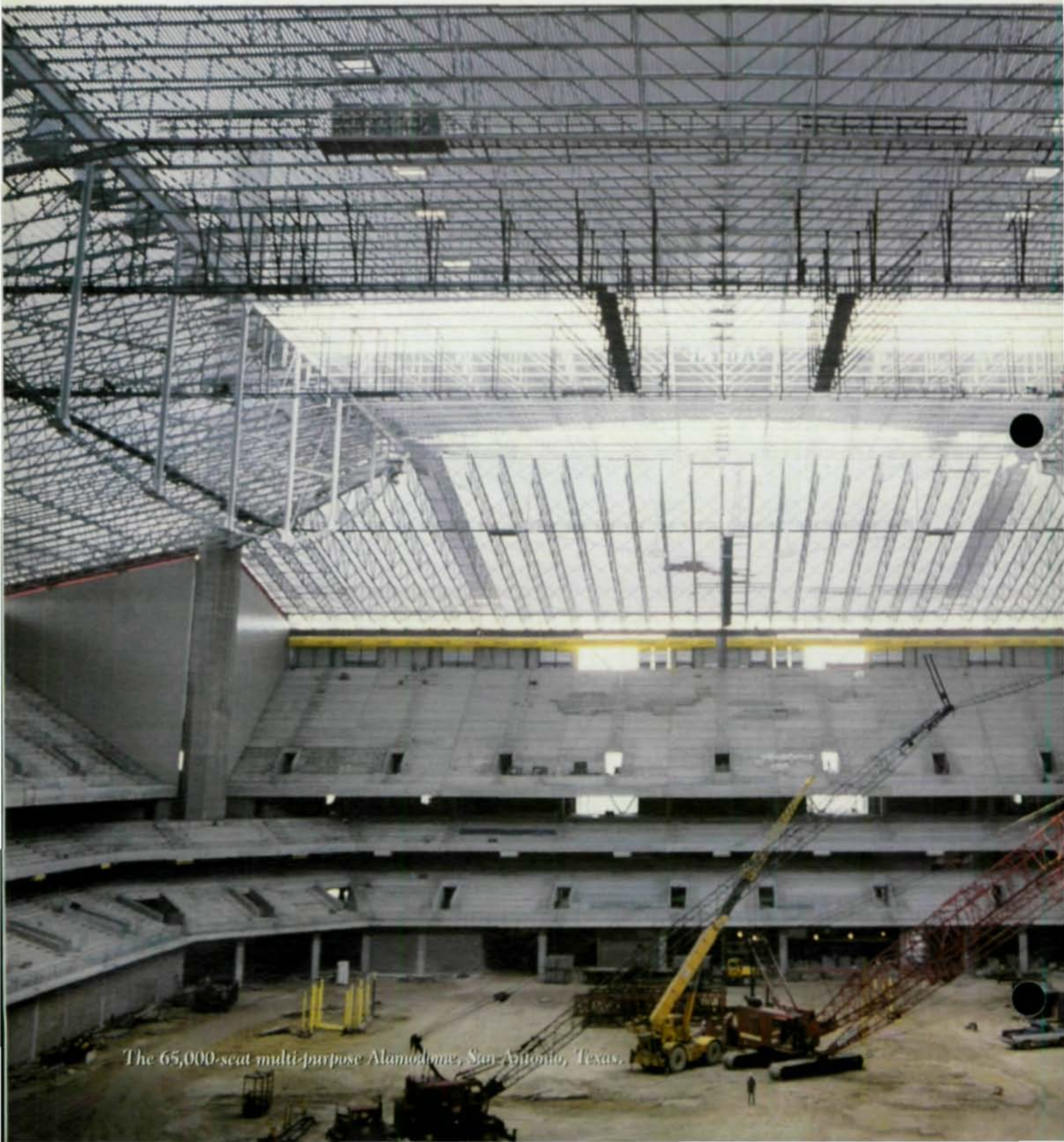
Composite Studs: Stud design criteria is similar to composite girders without restraint with the exception that if top reinforcement is used for restraint then additional studs are required between the point of maximum negative moment and point of zero moment. "The number of such studs shall be selected to develop the negative moment," Wexler said.

Cost Savings

The cost savings on the Mt. Kisco project were dramatic. With simple supported girders, the project would have used A36 W14x34 girders with no camber and no rebars and 40 studs. Wexler reports that in the New York City area, the 20' girder plus the 40 studs would cost approximately $610 (on projects outside of New York City, Wexler has paid as little as $450).

The composite system with partially restrained connections, however, required A36 W14x22 members with ¾" (19 mm) camber, four #5x8' long rebars and 34 studs. The cost in New York City for the 20' (6.09 m) girder, camber, rebars and studs is approximately $443 (outside of New York City, about $328), for a savings of $167 per member, or more than 27% compared with other design methods.

IF YOU'RE WONDERING ARE THE ANSWER, REME



The 65,000-seat multi-purpose Alamodome, San Antonio, Texas.

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As you might have noticed, we're proud to have been a part of a project like this. We'd be no less proud to assist you. Call any one of our plants listed below, or consult Sweet's 05100/VUL and 05300/VUL, to see how we can make your next project a success.

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Architects: Marmon Barclay Souter Foster Hays and HOK Sports Facilities Group; Structural Engineer: W.E. Simpson Co. Inc.; Project Manager: Day & Zimmerman, Inc.; Structural Contractor: Lyda Incorporated; Steel Fabricators: Crown Steel Inc. and Industrial Mechanical Co.; Steel Erector: John F. Beasley Construction Co.

AISC 1993 Prize Bridge Competition

Rules

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

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

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

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

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

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

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

Grade Separation Basic purpose is grade separation.

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

Movable Span Having a movable span.

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

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

Reconstructed Having undergone major rebuilding.

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

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

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

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

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

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

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

AISC 1993 Prize Bridge Competition

Entry Form

Entry Date__________

Name of Bridge__________ Completion Date__________

Location__________ Date opened to traffic__________

Category in which entered__________ Approx. total cost__________

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

Vertical clearance__________ Steel tonnage__________ Painted: Yes__________ No__________

Structural system(s) (describe briefly here)__________

Innovative Concepts__________

Descriptive data: Attach separate sheets (see entry requirements)

No. of photographs enclosed: Color prints__________ 35 mm slides__________

Design Firm:__________ Phone

Address:__________ Street — City and State — Zip

Person to contact:__________ Title

Consulting Firm (if any):__________ Phone

Address:__________ Street — City and State — Zip

Person to contact:__________ Title

General Contracting Firm:__________ Phone

Address:__________ Street — City and State — Zip

Person to contact:__________ Title

Steel Fabricating Firm:__________ Phone

Address:__________ Street — City and State — Zip

Person to contact:__________ Title

Steel Erecting Firm:__________ Phone

Address:__________ Street — City and State — Zip

Person to contact:__________ Title

Owner:__________ Phone

Address:__________ Street — City and State — Zip

Person to contact:__________ Title

This entry submitted by:

Name:__________ Title

Firm:__________ Phone

Address:__________ Street — City and State — Zip

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

Urban Transformation

An addition to a nearly half-century old department store allowed the creation of a modern urban mall

These two views show both the existing center and the new addition. Photo by Air Survey Corp.

By Thomas A. Bouffard, P.E.

Although City Place was originally designed in the 1940s as a six-story, standalone department store in downtown Silver Spring, MD, by the 1990s it had been transformed into a multi-tenant space. The project's owner recognized that its size and design were inadequate to compete with suburban malls but was hindered in any redevelopment plans by restrictions governing the site's density.

Fortunately, the developer requested and was ultimately granted a zoning variance that allowed an increase in site density.

The new plan called for a horizontal addition to nearly double the size of the existing center to 385,000 sq. ft. (35800 m^2), including space for 70 retailers, a 10-screen multiplex theater, and 85,000 sq. ft. (7900 m^2) of office space. Complicating the project, however, was the need to integrate the old structure with the new construction while also planning for a possible future addition. And, to satisfy the developer's financing arrangements, the construction needed to be completed in only 20 months while occupancy on the upper two levels of the existing building was maintained.

"Our first goal was to assemble consultants with a strong retail background that could quickly evaluate our proposed project and begin to prepare a fast-track set of contract drawings," explained Ray Podlasek, project manager with Petrie Dierman Kughn, the McLean, VA, based developer. "We also wanted to have a general contractor on board during the design phase to assist with cost control."

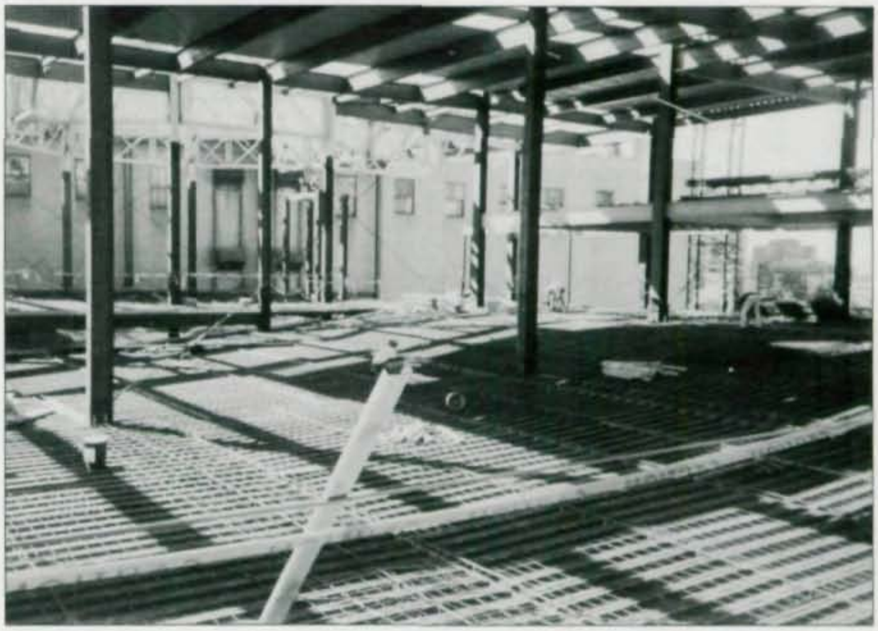
Design Goals

"It was a challenge to create an urban mall that had the look and feel of a traditional regional mall where land is more abundant," explained Tom Georgelas, a principal with Anderson, Cooper, Georgelas, the project's shell architect. "Since the site was [to be] fully utilized, our design, necessarily, took on a vertical layout."

But as designers have long known, shoppers prefer horizontal layouts. "We wanted to minimize the verticality of the mall so we positioned entries on multiple levels, which gives the impression the mall is lower than five floors," explained Bill Beitz, a principal with James P. Ryan Architects and Planners, Farmington Hills, MI, the project's interior architects. "To improve shopper circulation, we designed a long atrium with limited bridge crossings that encourages more foot traffic as people move between levels and entries. Finally, we placed the escalators at opposite ends of the mall and the stair and elevator off the main atrium."

Egress requirements became a main design issue in developing the interior layout of the building. Factors such as a 2,300 seat theater on the fifth floor, provisions for a future eight-story addition above the fifth floor, 300,000 sq. ft. (27900 m^2) of retail space, and exterior grades sloping one-and-one-half levels across the site all had to be accommodated in the design without sacrificing the retail lease requirements.

The final design utilized the lower four stories of the existing concrete structure for retail space and the upper two stories for office space. The existing escalators were removed and the openings were infilled. The new construction consisted of four levels of retail perfectly matching the existing floor-to-floor heights and a fifth level theater. The theater is two-stories high, so the roof line of the addition matched the roof line of the



The designers chose a 30′ x 30′ bay size to maximize flexibility, including the possibility of adding an office or hotel tower. The construction also included a covered, 32′-wide pedestrian bridge that was framed at each side with coverplated W36x300 girders.

The exterior of the center retains urban elements but also shares some design features with typical suburban malls. The design also was intended to stress the centers overall horizontal layout.

existing structure.

Structural Considerations

"Steel was chosen as the structural system for the building because of the flexibility it offered the design team," explained Wayne Bryan, P.E., a principal with Ehlert/Bryan, Inc., the project's structural engineer. "The steel structure allowed for long column-free areas in the common mall, in some cases spanning up to 70′ (21.3 m). Steel also gave us the ability to use a 30′ x 30′ (9.14 m x 9.14 m) typical bay, which worked well for both the retail and the future tower."

Unshored composite construction utilizing ASTM A572 Grade 50 high strength steel combined with a lateral bracing system of cross and eccentric K braces was used. This system provided cost control while allowing the framing flexibility the architects would need to design a successful mall. The project used a total of 373 tons of A36 steel and 1,441 tons of A572 Grade 50 steel.

Mill camber was used throughout the project. The cambering proved to be an effective method to control dead load deflection and, in turn, cost. Without camber the floor beam at 10′ (3.05 m) on-center in a typical 30′ x 30′ (9.14 m x 9.14 m) bay would have been a W16x31. Cambering allowed a reduction to a W16x26, representing a materials savings of 180 lbs. (81.7 kg) per beam. The cost of mill cambering was $0.02 per lb. or $15.60 per beam. This resulted in a net savings of $90 per beam or $90,000 for the entire project. The typical girders, due to their size and stiffness, did not require cambering. However, many of the long span members were cambered up to 1¼" (32 mm).

As the standard mill tolerance of cambered beams is minus 0" plus ½" (13 mm), typical cambers ranged from 50% to 75% of the immediate dead load deflection. This eliminated the possibility of over cambered beams that could affect shear stud cover and the slab thickness, which is critical for the floor fire rating.

To meet the two-hour fire rating requirements, a 3" (76 mm), 20 gage composite deck was initially selected as the most economical for the 10′ (3.05 m) spans. However, the general contractor, Glen Construction Co., Inc. of Gaithersburg, MD, showed that a 3¼" (83 mm) lightweight concrete topping over a 2" (51 mm) 19 gage, composite deck would save $80,000 in concrete cost, which would more than make up for the added cost for a 19 gage deck.

During the shop drawing phase, the deck supplier suggested an alternate girder/deck detail whereby the deck was continued over the girder. This eliminated cutting the deck in each bay as would be required with the more traditional deck detail. "Our review focused on the two main components of the detail that would need to be addressed in order to allow substitution," stated Bryan. "First, we needed to assure ourselves that the paired stub arrangement over the girders would not be adversely affected. Second, that the erector could install the deck using standard industry tolerance and still maintain a good alignment between the deck joint and girder center line."

Pedestrian Bridge

The project included a covered, 32′-wide (9.75 m), 90′-long (27.4 m) pedestrian bridge that was framed at each side with cover plated W36x300 girders on the floor and roof level. The girders were cambered 3" (76 mm).

Support for the bridge required that the new steel columns and footings be placed within the existing buildings. The columns were spliced at each level and separately inserted through holes created in the existing concrete floor. The end

of the bridge was designed with an invented stiffened seat made up of wide flange members. Full penetration moment welds between the W36 girders and end column provided the mechanism to resist the eccentric support. This support was fixed to the column using a traditional rocker plate pintle connection. The opposite end of the bridge was supported on free-standing concrete columns with an expansion joint at the adjoining parking structure.

The bridge, due to its size and location, required a substantial amount of pre-planning. Erection of the bridge would require the closing of a major downtown street for three days. The 60-mile (96.6 km) travel path from the fabricator to the site had to be carefully selected since the W36 girders were being shipped full length. The girders had to be properly oriented on the trucks to be ready for the lifting as the street was not wide enough to reposition the steel.

Serviceability

"Based on our retail experience, we have learned that some of the longer beam and girder spans, 50′ (15.2 m) and over, can be susceptible to vibration problems," explained Bryan. "Specifically, long span beams that frame into girders on only one side." As technology moves forward, spans tend to increase and material stiffness decreases. These two factors can combine to make vibration analysis all the more critical.

For this project, semi-rigid connections were used to add stiffness to the long-span members. The connection consisted of the addition of beams on the opposite size of the girder, bottom plates welded to the girder, additional headed studs, and reinforcing bars within the slab. The beams were designed to support 75% of the live load.

The fifth floor of the new construction has 10 individual auditoriums for the cineplex. The floors required four different slopes from the back of the theater to the front and the large theaters (60′ wide or 18.3 m) were required to have a curved floor. The curved floor allows the seating to be set on a radius improving the sightlines to the screen.

Various framing schemes were investigated but the tried-and-true approach of unshored composite structural steel was selected. Infill beams were placed across the width of the auditorium at the locations of changes in the floor slope.

In the smaller theaters, the floor construction consisted of 3¼" (83 mm) lightweight concrete fill over 2" (51 mm) metal deck. Infill beams were spaced at a maximum of 10′ (3.05 m) on center. In the large theaters with the radial seating, the same framing scheme was used; however, the thickness of the concrete fill varied to maintain the curved floor contours. The floor slopes were framed by placing the beams at the chord of the arc of the slope changes. This system proved to be an ideal way to solve this framing condition. A sophisticated table of dimensions keyed to a typical plan and elevation was developed to provide the necessary information to the detailer and contractor.

Lateral Stability

The lateral stability of the building, including the lateral resistance for the Phase II Tower, consisted primarily of cross-bracing and some concentric and eccentric K bracing. Bracing was located at the perimeter of the building to maximize tenant space planning options. The cross-bracing consisted of A36 double angles connected to gusset plates using ⅞"-diameter (22 mm) A325 high-strength slip critical bolts.

The K bracing was used as a result of the shell architects desire to have one large window display per bay on the public sides of the building. The K bracing consisted of A500 Grade 46 tube sections slotted and field welded to the gusset plates.

Renovation Work

Re-using the existing 1940s building required a substantial renovation. The building was a traditional cast-in-place concrete structure framed with one-way concrete pan joists and girders. The new architectural plan called for expanding the loading dock by two bays, installing escalators at each level, adding a new freight elevator and adding a 4,800-sq.-ft. (446 m^2) mechanical penthouse on the existing roof.

Two bays of concrete framing on each level were removed to allow for the escalators and new floor framing. Structural steel was used to frame out each area to support the perimeter of the existing concrete floor. The trickiest part of the construction was drilling the existing concrete for the expansion bolts. Due to the age of the structure, the concrete was extremely hard and difficult to drill. Standard impact rotary hammers were ineffective in drilling the 25 mm holes. The erector found success by using a water-cooled, diamond-tipped drill.

Thomas A. Bouffard, P.E., is a project engineer with Ehlert/Bryan, Inc., a structural engineering firm with offices in McLean, VA and Southfield, MI.

AISC ANNOUNCES

VOLUME II CONNECTIONS MANUAL

A S D / L R F D

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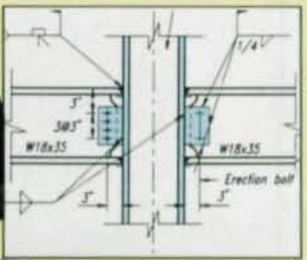

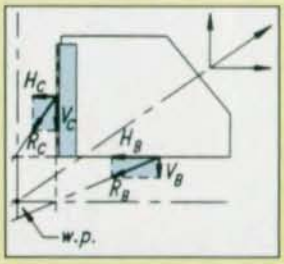

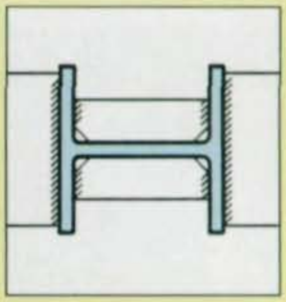

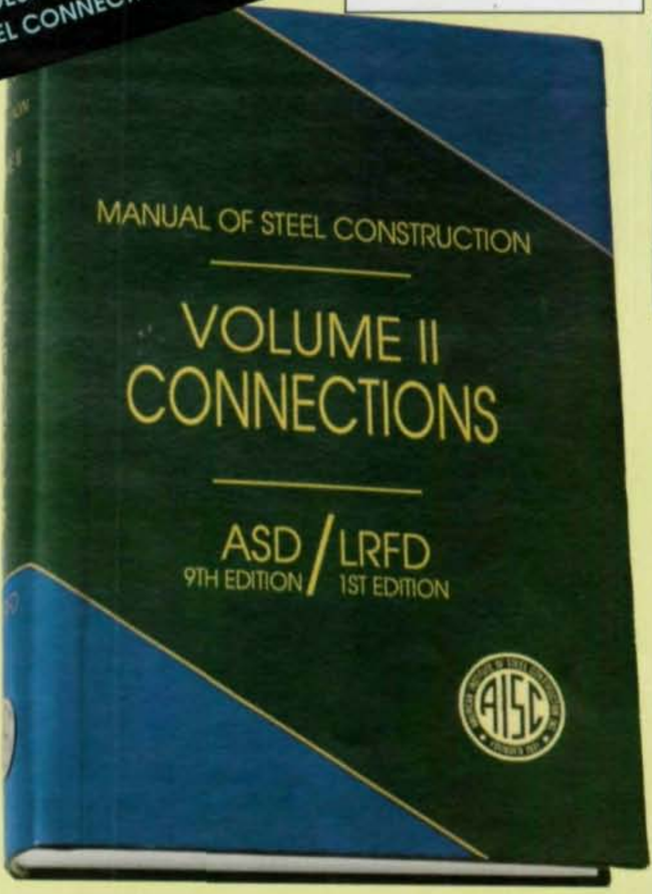

▲ *Volume II Connections* is a companion and extension to the AISC *Manual of Steel Construction* (both LRFD, 1st Edition, and ASD, 9th Edition).

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Attracting Attention

To survive in Las Vegas, an old retail center had to expand and create a new, bolder image for itself

Before and after: The old center was run-down and could not compete with newer Las Vegas Centers. The new center is designed to attract attention.
Finished photography by Timothy Hursley

As Las Vegas has become glitzier, more modest retail operations have had to upgrade to compete for business. Despite its prime location across from the Stardust Hotel on "The Strip," the small retail center and an adjacent 1950s-style motel on the Gold Key site had fallen on hard times.

After careful consideration, the developer opted to demolish the motel and replace it with new retail space which would be combined with the existing stores to form a new retail complex.

"The existing retail building was fairly old and had been tacked on to and renovated in a piecemeal fashion," explained Raul Anziani, a project architect with ELS/Elbasani & Logan Architects, Berkeley, CA. "It had stucco, wood and metal finishes with no rhyme or reason to the design—there was no uniformity." Structurally, most of the existing building was steel framed with concrete block or wood infill for partition walls.

The new plan was to build an addition to increase the rentable space from about 25,000 sq. ft. to 54,000 sq. ft. (2325 m^2 - 5020 m^2) while at the same time recladding the old building to create a uniform image. And, befitting its location, the image chosen was loud.

The entire center is clad in multi-colored metal panels—silver, gray, green and chrome. And even though the center is essentially a single story, the designers chose to extend the cladding up an additional story. "When you're doing a retail project, there's always a desire on the part of the client to

make it stand out as much as possible and to attract attention," said Donn Logan, FAIA, a principal with ELS. "Las Vegas is a place where you can transcend the genre to where garishness becomes art."

Towards that end, the panels jut out here and there and rise up to different elevations. "There's an angularity to the site itself since one of the adjacent streets is angled," Logan said. "This design is reflective of its site and location."

And topping it off is a series of both free-standing and attached trellises and canopies. "Besides attracting attention, they also provided shade, which was very important in Las Vegas' climate," Anziani explained. As Las Vegas becomes more of a family vacation destination, pedestrian traffic has increased, so this structure is much more pedestrian oriented than older Las Vegas buildings.

Of course, supporting all of the cladding and ancillary structures effected the structural design.

The new structure is steel framed with a wood joist roof, according to Raymond Khoury, S.E., a structural engineer with Martin & Peltyn Structural Engineers, Las Vegas. "Because of the high facade, wind governed the design," he said. While the building's roof line is approximately 15' (4.57 m) high, in some places the parapet wall extends twice that height—and it needed to be supported on cantilevered W12x40 steel columns. General contractor on the project was Taylor International Corp., Las Vegas.

"The wood diaphragm holds the structure together, but lateral loads are picked up by K braces spaced approximately 100' (30.5 m) apart along the 300' (91.4 m) length of the structure," Khoury explained. In addition, there is a row of steel beams at both the roof level and at the top of the columns.

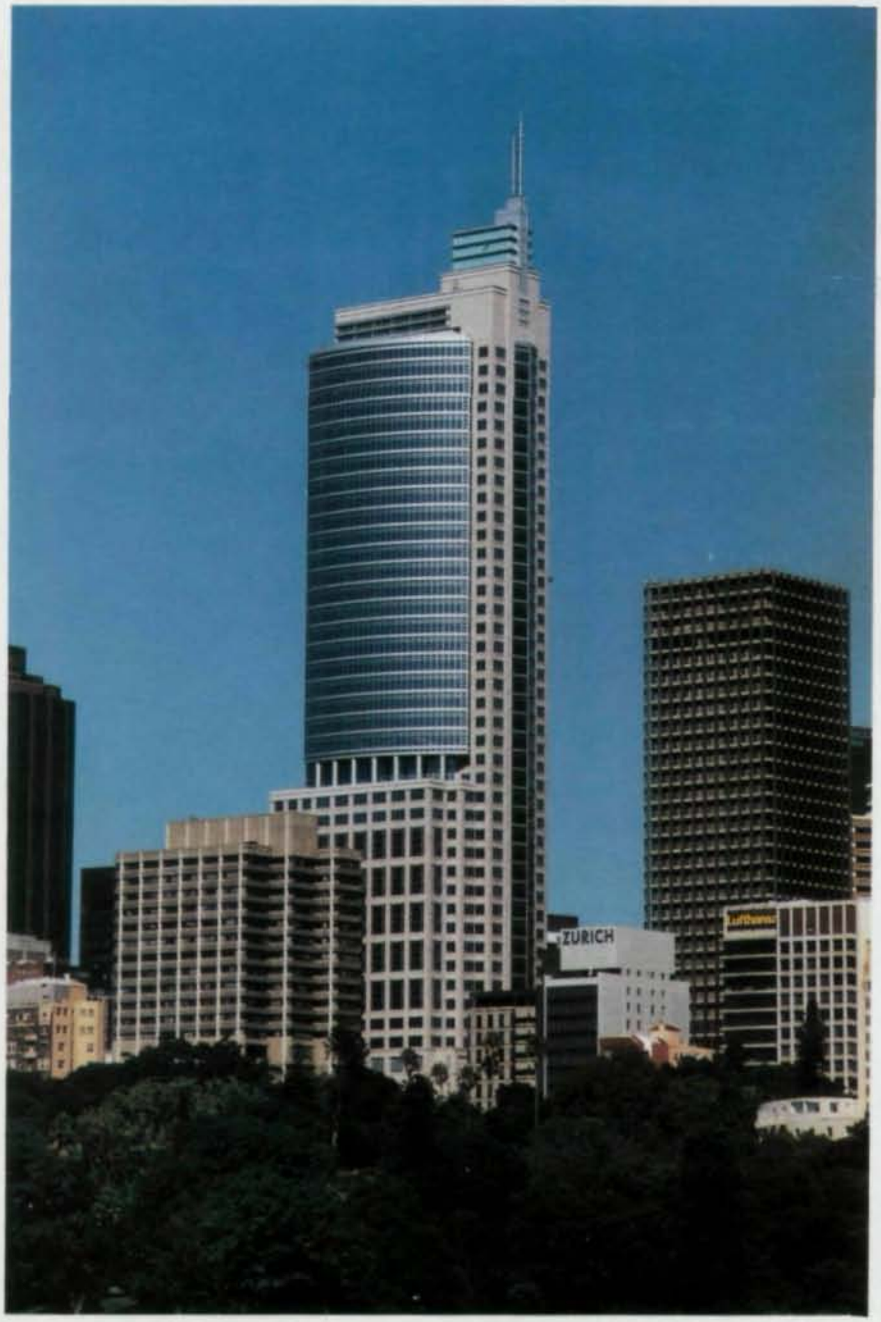
The beam line at the roof level are W14x22 wide flange sections, while 6x4x¼ tube shapes were used at the top of the structure. "We used tubes for the top beam line because they are exposed in some places," Khoury said. Clip angles are welded to the tube and then bolted to the wide flange columns on the top and bottom.

Several large trellises, some of which are as much as 50' (15.2 m) high, are free standing and are supported on tube columns. To further enhance the structure's angularity, cut and bent wide flange sections were used to support the trellises' metal roofs. The tube columns range from 5x5x¾ to 12x12x½. The shading material at the top of the trellis is supported on T-shapes cut from W21x44.

During the day, the trellises and canopies provide shade, while at night they act as bright beacons to attract attention to the site. Both the trellises and canopies are supported on steel members that were left exposed to create visual interest.
Photos by Timothy Hursley.

Australia Rediscovers Steel

An aesthetically intricate building provided a toe-hold for steel to penetrate a concrete stronghold



The Chifley Square Building facade is highly articulated to help create a transition from the local street front lines to the overall city grid, while also enhancing views of Sydney Harbour.

By Leonard M. Joseph and Thomas Z. Scarangello

Why would engineers in very English, very modern Australia look to the United States for technical expertise? There's a one-word answer: Steel.

During the past few decades, straight-sided, flat-topped concrete towers have become nearly universal for Australian commercial construction. Steel construction was generally limited to heavy industrial applications such as ore loaders or mills.

But rather than building another box-like building, the developer of The Chifley Square Building in Sydney favored a highly articulated building design by architects Kohn Pederson Fox of New York and Travis Partners, Ltd. of Sydney. And in another departure from the "Australian-norm", the developer had no preconceived notions about structural framing systems.

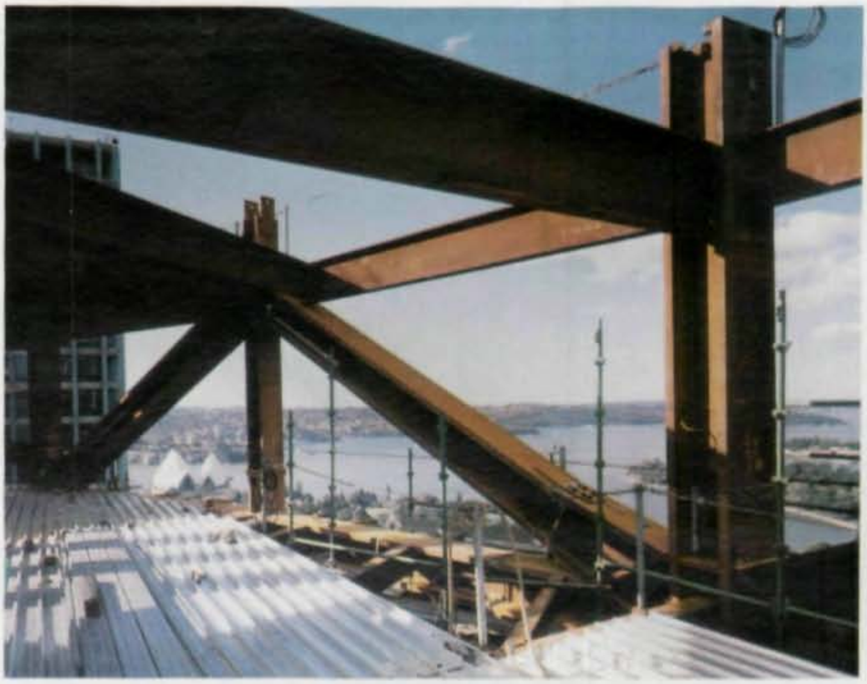
Thornton-Tomasetti/Engineers was brought into the design process for this new high-rise office tower by Flack + Kurtz Australia Pty. Ltd., a structural/MEP engineering design firm, to study alternative highrise structural systems and develop the wind resisting system. This included the opportunity to give steel framing another look. Steel offered three potential advantages: speedy erection, easy cabling access, and cost savings.

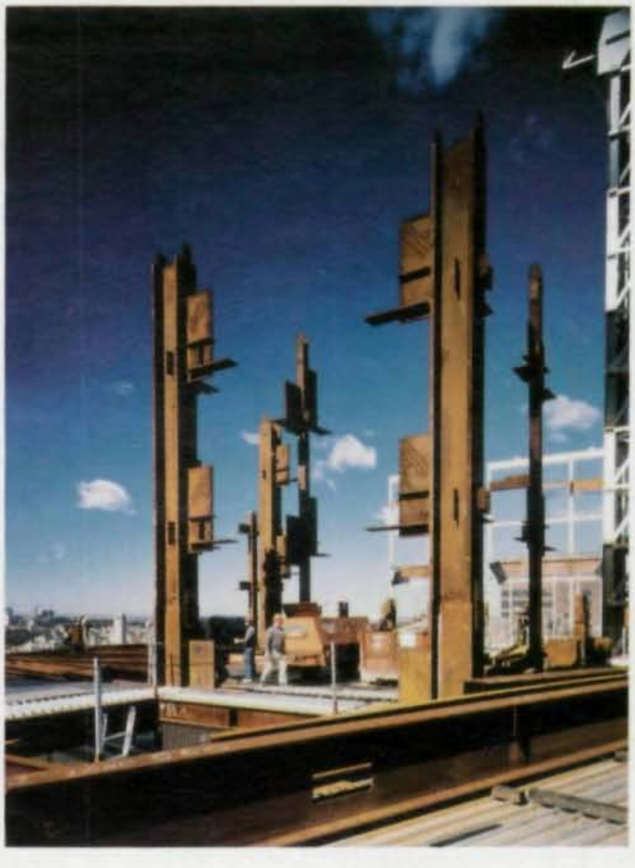
Erection Speed. Based on US experience, steel frame erection at a rate of two floors a week or better was anticipated, excluding start-up and non-typical floors. High-rise concrete construction in Sydney usually proceeds at about one floor a week and was subject to delays from bad weather. While some up-front time would be needed to fabricate the first tiers of steel, that time would be quickly made up for on a building of this height (43 numbered floors above grade, equivalent to about 50 stories). Although financial and weather conditions caused some delays on this project, steel erection was able to proceed whenever there was a break in the weather. In contrast, the rainiest winter in recent memory would have severely slowed concrete work more, causing delays at all points in the construction cycle—lathing, forming, placing and stripping.

Cabling Access. The owner envisioned The Chifley Square Building as home for financial and trading firms capitalizing on Australia's Pacific Rim location and western European cultural background. Three or four computers per trader, and 50 trading stations or more per floor, made cable management a critical concern.

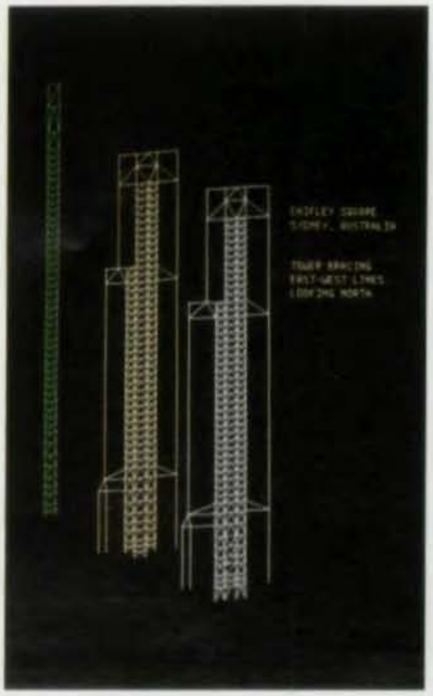
On each level raised "computer flooring" gave cabling flexibility, but adequate riser space and access was also needed between floors. While cable riser space can be provided in cores of both concrete-framed and steel-framed buildings, access to risers differs. The "classic" Sydney high-rise has concrete walls for shafts and shear resistance. Making post-construction openings is noisy, messy, costly, and subject to structural limitations. However, a steel-framed core, with lateral bracing in the form of diagonal members and shaft walls in gypsum board, can accommodate new openings quickly, easily and inexpensively.

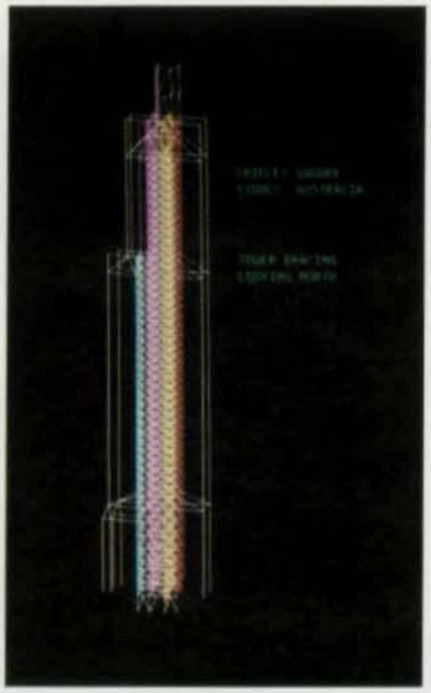
Construction Cost. Unlike the U.S. with its multiple steel suppliers, concurrent steel demand in other sectors of the economy could have been significant in Australia since there is a limited pool of labor and materials in that country. Demand that outstrips domestic supply may simply go unfilled—an unacceptable situation for construction of a building with high





The transfer truss pictured above is located at one of the many setbacks with a view of the famed Sydney Opera House and Sydney Harbour. The core columns shown at left have bracing gussets with extended bottom plates for ease of erection. Note that the bracing connection plates stop short of the column face. Also, note the narrow rib deck.





carrying costs, but fortunately sluggish industrial demand at the time of design meant that such a situation would not occur.

Building Design

The Chifley Square Building has a very compact core, T-shaped on lower levels and L-shaped above. This maximizes net rentable space on the floors, but is too slender for adequate lateral stiffness.

To improve lateral stiffness, outriggers at three levels engage two outrigger columns on each building face. This core-and-outrigger system has two advantages: It permits a highly articulated facade, with curves, setbacks, notches and overhangs which would have been impractical with a perimeter tube or frame system, and it uses some of the outriggers for double duty as pickup trusses at setbacks. However, design of this very complex 3-dimensional structural system required extra engineering effort.

For deflection control, steel was located at the optimum, or most efficient, locations. Relative member efficiency was determined using virtual work. For each direction of interest we applied a virtual load at a point of interest - usually the top occupied floor - and lateral wind forces over the tower height. Torsion used a "virtual couple" and wind loads offset from the center of stiffness. For each direction, the contribution of each member to the point of interest was determined.

Members were ranked by relative efficiency (contribution per unit volume of the member), and the deflection for the trial member layout was compared to the target deflection. If deflection was greater than target, the helpful effect of increasing the most efficient member was considered. Note that member efficiency drops as material is added, so efficiency was continually checked and lower-rated members were increased when they came "in the running."

Upper limits for member sizes were also established and checked. Since changing member sizes in an indeterminate structure changes force patterns, the analyses were automatically re-run with new trial sizes, subject to minimum sizes for strength, until convergence was reached. Optimization was done round-robin for the X- and Y-directions, and then for torsion.

Once "optimized" sizes were determined, a final analysis set the design forces. Member strengths, as established by the Australian code, were checked against an envelope of wind and gravity forces to cover vertical load minimums and maximums, and wind from all directions. Appropriate member sizes were then selected from a menu of possibilities, including many "do-it-yourself" sections that are a necessary part of working in an area with limited standard sections.

Presenting member sizes and forces on plots was straightforward, but one interesting twist was determining the force to be transferred *through* the core columns. This was necessary because the columns were built up of three to eight plates, and plate prying could be critical. The forces were determined by checking equilibrium joint by joint for each individual load case and combination.

As the architecture was being refined, significant changes in building shape and framing geometry required reanalysis and redesign. The automated analysis, design and presentation process permitted the last (15th) redesign, for a large rooftop penthouse to accommodate profitable microwave transmitters, to be performed in just two days.

Special Considerations "Down Under"

Australia's steel supplier, BHP Ltd., offered steel in two grades for construction use—250 MPa yield (equivalent to grade 36) and 350 MPa yield (equivalent to grade 50). As the forces and proportions of members were to be similar to those in US practice, these grades were appropriate.

Material supply also was a problem. Due to the relatively small size of the Australian economy, the limited number of potential suppliers (one) and the tendency (also observed in the US) to minimize stock in steel supply warehouses, virtually all steel for this project was mill ordered.

In addition:

- Lower floor columns required large, thick plates which were rolled infrequently.
- Lead times for mill orders required ordering before completion of design.
- The range of standard rolled

The basement core column shown above is built up from seven plates, while the split columns shown at right straddle a sidewalk arcade below.

shapes available (wideflange, channel, angle) is much more limited than in the US. The handful of standard shapes are limited to a series of steadily increasing sizes and weights, which may work for industrial use but ignores the different needs of beams and columns.

- Plate stock is supplied as slabs, and in only a few lengths and widths.
- Plate stock thicknesses vary in large steps.
- Plate is not available thicker than 100 mm (3.9").

Due to these limitations, some premiums were paid to ensure construction speed. Solutions included:

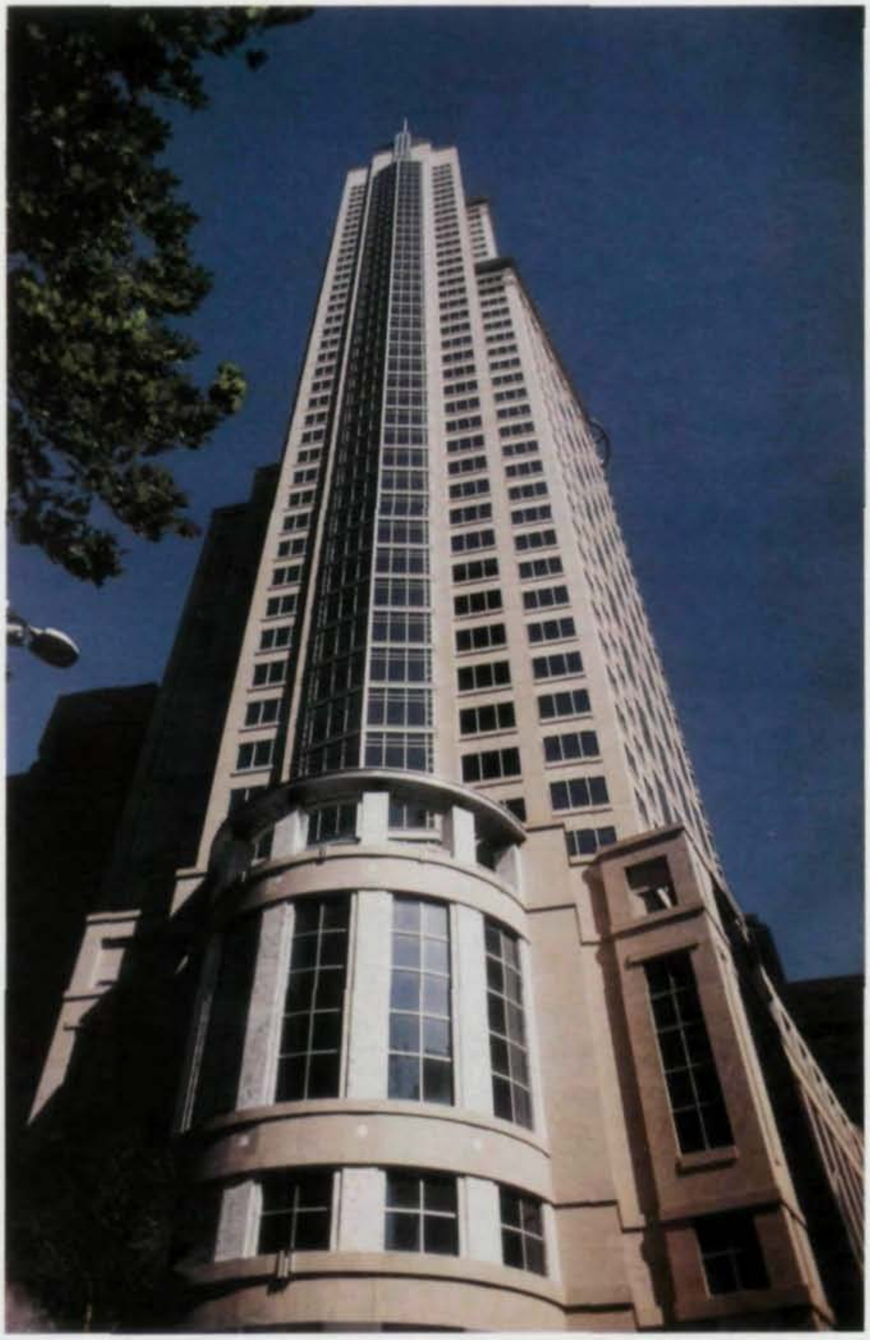
- Pre-ordering stock in advance of the design. By ordering a selection of plate we "mixed and matched" during design to minimize waste.
- Ordering 350 MPa stock even though much of the heavier steel would be deflection-controlled and could be 250 MPa material. This simplified ordering, stocking and tracking material in the "mix and match" process.
- Determining the range of likeliest member sizes and lengths for rolled sections, and ordering just enough overlength pieces to get going until the design was better defined.
- "Oversizing" typical floor beams (compared to U.S. practice) due to the limited range of beam sizes. For example, an unshored composite 45′ floor beam in the U.S. might be a W21x44, but the nearest equivalent in Australia was a 530 UB 82 (W24x55). This extra weight was not wasted, however. Additional live load capacity was inexpensively made available by an extra bolt added to beam end connections.
- Setting shipping lengths and splice elevations to work with as-shipped slab lengths (minus milling allowances). While the resulting splice locations may not have been as convenient as the usual 2′ to 4′ above the deck, the wastage avoided was worth it.
- Designing plate widths to work with as-shipped slab widths (minus trim and cutting allowances).
- Designing custom built-up sections for the many members beyond the range of available hot rolled sections. This included most tower column sections, girder sections and wind bracing sections.
- Building up sections from multiple 100mm plates. While built-up sections are commonly required in the U.S. for lower-tier core columns, they can usually be made as an I or box from three or four plates of 6" to 12" thickness. Using 3.9" plates required ingenuity to address the needs of fabrication, erection and force transfer through sections built from six to eight plates.
- Making base plates from 300 mm thick by 1800 mm square cast steel slabs. This approach had a material cost premium and required careful quality control and assurance but was still more economical than alternatives of stacked or stiffened 100mm rolled plates.

Slab construction

The standard deck profile is 50 mm (2"), which limits spans to about 10′, and has "dovetail" or "keystone" ribs with wide flats of maximum slab depth, requiring more concrete fill than a typical US pattern and boosting slab dead load from the 45-50 psf typical for stone concrete in the U.S. to about 65 psf. Lightweight expanded-shale aggregate was not a practical choice.

Load criteria

U.S. office floors have 50 psf live load, reduced to as low as 20 psf at columns. The Australian code calls for a 3 kPa live load, equivalent to


The north view of the tower shows the numerous setbacks and changes in plane on the building's east and west faces.

63 psf, reduced to about 31 psf at columns, about 55% greater than typical U.S. practice.

Australian code wind loading includes the influence of wind directionality, relative probability of occurrence in different directions and recognition of wind tunnel testing. In these aspects it is ahead of the various "model codes" in the U.S. and way ahead of the older municipal codes.

Seismic loading was not considered in Australian codes until the recent Newcastle earthquake. Code provisions have since been developed.

End connections

One interesting difference in steel detailing practice is in the approach to seated connections. In the U.S., seated connections are usually designed for a beam bearing length sufficient to accommodate a plain beam end without web crippling, plus construction tolerances. This gives a deep seat which imposes some bending in the column.

In Australia we have seen the popularity of minimum-depth seats which use beam end plates to bear on seats of single plates welded flat to columns. This approach minimizes eccentricity of applied load and eliminates any concern for beam crippling failure. Beam fabrication is increased but seat fabrication is simpler. We believe that their approach has possible application here for heavy reaction situations.

Steel construction provided significant benefits in schedule, cost and user flexibility for The Chifley Square Building, even in a traditional stronghold of concrete construction where limited shapes and plate stock required the time-consuming design and construction of many elaborate built-up shapes to accommodate the needs of tall building framing.

Leonard M. Joseph and Thomas Z. Scarangello are senior associates with Thornton-Tomasetti/Engineers in New York City.

Welding Exposition

More than 20,000 attendees are expected at the 1993 AWS International Welding Exposition and 74th Convention scheduled for April 27-29 in Houston. The technical program will include more than 25 sessions with more than 100 technical papers. Topics range from computer modeling to Programmed Automated Welding Systems to welding inverter power supplies. Highlighting the convention are industrial technology sessions concentrating on improving productivity and applying new technology. In addition, a session on ventilation will address upcoming OSHA requirements. Also, a thermal spray symposium will be held.

For more information, contact: American Welding Society, 550 N.W. LeJeune Road, P.O. box 351040, Miami, FL 33135 (800) 443-9353; Fax (305) 443-7559.

Welding Equipment

Burco has introduced a new line of welding equipment specifically designed for maintenance and repair applications. These applications demand power and portability. For example, the newly introduced 130 ST weighs only 22 lbs. yet has a 100% duty cycle rating at 105 amps. The company also produces a full line of plasma and MIG welders and a unique AC/DC portable TIG unit, as well as its standard line of arc welding, stud welding, engine drives and consumables.

For more information, contact: BURCO Welding & Cutting Products, Inc., Box 2115, Burlington, NC 27216 (919) 227-2737; (919) 228-0760.

Cutting Torch

The Henrob 2000's unique low-pressure/low-velocity flame equates to dramatically reduced gas consumption, permitting extensive use in the field with minimally sized gas bottles. Its twin-tip cutting design permits easy cutting of structural steel up to 1" thick with minimum kerf and clean, non-oxidized edges. The torch also can be used to weld difficult materials such as aluminum, stainless steel and cast iron, can braze without flux, cut thin-sheet materials with minimum distortion and without burning back galvanized coatings.

For more information, contact: Henrob Corporation, 3551 Voyager St., Suite 106, Torrance, CA 90503 (310) 214-4946; Fax (310) 542-5184.

Power Source

The Lincoln Electric Company has introduced the Invertec V300-PRO, an advanced technological source for stick and wire welding. It is a 300 amp arc welding power source that utilizes single phase (K1349-3 only) or three phase input power to produce either constant current or constant voltage outputs. It is lighter in weight than previous generations, making it ideal for jobs that require portability, such as construction. It is recommended for GMAW (MIG), FCAW (Innershield or Outershield electrodes), SMAW, GTAW (TIG with scratch start, K799 Hi-Freq Kit, or DC TIG starter) and Air Carbon Cutting.

For more information, request Bulletin E390 from: The Lincoln Electric Company, 22801 St. Clair Ave., Cleveland, OH 44117-1199 (216) 481-8100.

Stud Welding

Nelson studs literally anchor other members to the basic framework of structures. This design makes for maximum strength since the welds are actually stronger than the base metal. Since stud welding is at least three to four times faster than hand welding, it impressively reduces total man hours on a job. At the same time, stud welding does away with all the problems associated with holes in structural members, such as a weakening of the main frame and the need to seal holes to prevent leaks. In some types of construction—especially composite construction—further savings result from the use of lighter and less costly beams that also reduce a building's height and weight.

For more information, contact: TRW Nelson Stud Welding Division, P.O. Box 4019, 7900 West Ridge Road, Elyria, OH 44036 (216) 329-0400; Fax (216) 329-0526.

Power Supplies

A new welding power supply system from AFTEK offers all standard constant current welding processes: SMAW (stick electrode); GTAW (TIG); Submerged Arc; Air Arc Gouging; and GMAW (hard wire MIG all positions). Features include: 100% continuous duty (400A-24 arc volts; 650A-32 arc volts; 1000A-42 arc volts); adjustable amperage range in 5A implements (5A-445A; 5A-745A; 5A-1245A); standard input voltages

(230 / 460 / 575 - 3PH - 60Hz; 220 / 380 / 440 - 3PH - 50Hz; 380 / 400 / 415 - 3PH - 50Hz); thermostatically controlled alarms on rectifier to automatically shut-down power source in case of overheating; units can be paralleled together for higher amperage output; units are stackable for maximum power in minimal space; and three year warranty on power transformers, rectifiers, and current control resistors.

For more information, contact: AFTEK, Inc., 1023 Pineville Road, Chatanooga, TN 37405 (800) 742-5458.

Power Source

Built for rugged use at construction sites, the Miller XMT Rack product line has been expanded to include a second eight Pak and a new four Pak model. While the original XMT 8 Pak Rack was designed for operation on 230 VAC or 460 VAC power, the new 8 Pak Rack operates on 460 VAC or 575 VAC primary power. The new 4 Pak Rack is designed for 230 or 460 VAC primary power operation. With eight 300 amp power supplies and 40 amps of auxiliary power, XMT 8 Pak Racks weight just 1,700 lbs. Four 300 amp power supplies bring the weight of the XMT 4 Pak Rack to just 800 lbs. All of the models have a compact 63" wide x 40" deep footprint and can be easily transported from job site to job site. Fork lift access is provided on all sides, as well as a central eye for crane movement.

For free literature, contact: Miller Electric Mfg. Co., P.O. box 1079, Appleton, WI 54912 (800) 950-9353.

MIG Welding Gun

Bernard Welding Equipment Co. has announced the availability of its new "Q" Gun Series, an advanced MIG welding gun with three basic models: 500 Amp MIG gun; 350 Amp MIG gun; and 200 Amp MIG gun. In addition to exceeding the features included in most other MIG guns, this new generation of easy-to-handle, higher performance guns will provide users with consistent, high-quality welding performance, including less waste, simplified access, and easier operator handling. Dramatically new "jump liner" technology makes it possible to reduce liner costs by as much as 65%. In addition, a new 6" long jump liner enables the welder to replace the body tube liner without having to change the gun liner. The series also offers seven new heavy duty, two-piece, spatter resistant copper nozzles, as well as interchangeability with a broad range of EZ Feed II nozzles.

For a free information package, contact: Bernard Welding Equipment Co., P.O. box 667, Beecher, IL 60401 (800) 323-8768; Fax (708) 946-6726.

Inverters

PowCon's inverter welding power sources allows up to 75% reductions in size and weight of the transformer and up to 45% energy savings as compared to SMAW or GTAW power supplies. When combined with an engineered plastic housing, the result is a power source that weighs only one-third that of a conventional transformer and takes up significantly less floor space, yet provides the same power output. PowCon, which also sells plasma cutting tools, has entered into a marketing agreement with Kemppi to sell their product line of inverters in the U.S.

For more information, contact: PowCon Inc., 8123 Miralani Dr., San Diego, CA 92126 (800) 833-9353.

MIG Wire Feeder

M.K. Products has introduced a new integrated power supply and push-pull wire feeder, with capabilities of welding up to 50'. The CobraMIG 250 welding package includes a self-contained 250 amp CV, push-pull combination power supply and MIG wire feeder. The unit comes fully assembled and, after unpacking, can be up and running within 15 minutes. It has excellent short and spray arc characteristics and no spatter. It's foundation is a rugged, all-steel rolling cart with cylinder rack. Three optional kits further increase flexibility: remote voltage kit with 25' cable and 25' extension; digital meter kit; and a stitch/timer kit.

For more information, contact: M.K. Products, Inc., 16882 Armstrong Av., Irvine, CA 92714 (714) 863-1234; Fax (714) 474-1428.

STEEL MARKETPLACE

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