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Common weld washers are made of 16 gage material (0.0598") and have a 3/8" diameter hole. The weld should overfill the hole to produce a visible weld diameter of about 1/2". **Typical 3/8" x 16 gage washers are not recommended with deck design thicknesses equal to or greater than 0.028" (22 gage). Weld washers are recommended for welding in deck panels thinner than 0.028".**[1] Most form deck products are furnished in metal thicknesses ranging from 0.0269" (23 gage) to 0.0149" (28 gage) and therefore weld washers are recommended. The table is based on typical form deck material (grade E) such as used by **United Steel Deck, Inc.** to produce **UFS, UF1X, UFX-36, and UF2X** which are form decks ranging in depth from 9/16" to 2" - a catalog is available on request. A 70 ksi electrode was used for the tables.

Deck Gage	Thickness	Design Uplift (Tensile) Value, lbs. [2]			
		I  Weld washer/ one deck thickness.	II  Weld washer/ two deck thicknesses.	III  Weld washer/edge lap (at support)-- weld is eccentrically loaded.	IV Shear Value, lbs. [3] Shear strength values are based on a safety factor of 2.75.
28	0.0149"	740	850	510	435
26	0.0179"	760	890	530	565
24	0.0239"	810	980	560	855
23	0.0269"	830	950	580	1030

Values in I, II, and III are based on a safety factor of 2.5 but includes a 33% increase for wind loading.

(1) Luttrell, L.D. (1993),"Arc Puddle Welds and Weld Washers for Attachments in Steel Deck", Steel Deck Institute, P.O. Box 9506, Canton, Ohio, 44711.

(2) LaBoube, R.A. and Yu, Wei-Wen (1991), "Tensile Strength of Welded Connections", Final Report, Department of Civil Engineering Center for Cold Formed Steel Structures, University of Missouri-Rolla, Rolla, Missouri, 65401.

(3) Luttrell, L.D. (1987), "Diaphragm Design Manual, Second Edition", Steel Deck Institute, P.O. Box 9506, Canton, Ohio, 44711.

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MODERN STEEL CONSTRUCTION

Volume 34, Number 5 May 1994

The National Steel Construction Conference will be held May 18-20 in Pittsburgh and will feature more than 20 technical and general sessions along with an exhibtion of products for the steel fabrication, engineering and erection community.

FEATURES

DEPARTMENTS

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If you're looking for a good party, Pittsburgh probably isn't the place for you this month. But if you're looking for good, practical information on the design, fabrication and erection of structural steel, then head on over because the National Steel Construction Conference is coming to town. And this year's program looks awfully interesting.

Having grown up in New York, the highlight of the show for me will undoubtedly be the general session on the World Trade Center Explosion. Leslie E. Robertson, one of the world's most noted structural engineers, is scheduled to discuss the performance of the steel structure during and after the explosion and to offer technical recommendations for future design considerations. Also, Jack Daly, of Karl Koch Erecting Co., told me he'll be showing a videotape detailing the damage and highlighting what worked well—and what didn't.

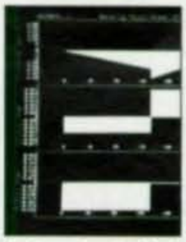
I'm also looking forward to several sessions on electronic compatibility for engineers, fabricators and detailers. Harry Moser from DuPont and Sayle Lewis from Fluor Daniel are scheduled to discuss how their two firms use electronic data transfer to reduce costs and increase efficiency. A technical session entitled "Lean Engineering", featuring Mark Holland from Paxton & Vierling Steel Co. and Sam Lawrence of Parsons, also is expected to deal with electronic data transfer. And finally, engineers and fabricators will have a chance during two open forums to describe their software needs and to tell AISC what tools are needed to improve design efficiency.

Of more immediate concern to most engineers and fabricators is a session on connection design featuring Geoffrey Kulak from the University of Alberta and Omer Blodgett from Lincoln Electric Co., two internationally recognized experts on, respectively, bolting and welding. After they present their papers, the two experts are expected to be available for a question and answer session modelled after the Steel Interchange section of this magazine.

As always, the conference will feature a product exposition with nearly 75 exhibitors. Several exhibitors are expected to introduce new products. For example, *Cad*vantage says they'll be demonstrating a detailing system featuring a Windows graphical interface system that runs on pen computers and recognizes handwriting. And Huck will be demonstrating their soon-to-be-introduced blind bolting system.

A complete conference program is bound into the center of this magazine and an exhibitor listing is printed on pages 65-72. *Modern Steel Construction* will be in the AISC booth (numbers 333 and 334). Be sure and stop by and say hello. **SM**

How to get from here



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

Steel Interchange
Modern Steel Construction
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Chicago, IL 60601-2001

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

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

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

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

When welding a steel that has dual certification (A36 and A572 Gr 50), is there a low hydrogen electrode requirement?

The following is a synopsis of an extensive welding study Chaparral Steel had performed on Dual Certified Steel.

In essence, multi-cert steel is an ASTM A572 grade 50 steel that, through stringent manufacturing controls, also conforms to the chemical and physical requirements of ASTM A36.

The extensive welding tests we conducted conclude that the Vanadium or Columbium/Niobium alloy additions do not adversely effect the A36 welding procedures. In other words if the multi-cert steel is to be used in an A36 application, the usual A36 welding procedures are applicable and A572 Grade 50 procedures are applicable if the steel is to be used in an A572 Grade 50 application.

J. H. (Ted) Temple
Chaparral Steel
Midlothian, TX

A test program was conducted at the request of Chaparral Steel in order to compare the weldabilities of A36/A572 Gr 50 dual grade steel and of A36 steel. Both steels are structural steels that are produced by Chaparral Steel in Midlothian, Texas. A36 steel is familiar throughout the industry. A36/A572 Gr 50 is a steel formulated to meet the overlapping chemical and mechanical specifications for both ASTM A36 and A572 Grade 50 steels. This "dual grade" capability was achieved by careful selection and control of steel chemistry.

The intent of this program was to demonstrate whether or not these two somewhat different steel products could be welded using identical welding procedures. The information to be generated in the test program was intended to develop a data base of weld test results to enable responses to questions concerning weldability and, moreover, the applicability of existing welding procedures to this dual grade steel. There was concern in the field as to procedures to the dual grade steel. There was concern in the field as to whether a steel with ostensibly higher properties would need different welding procedures from A36 steel. Also, the program would provide information to help field personnel convince welding inspectors that a particular welding procedure could be applied to a dual grade steel and to A36.

Welds were made in "thin" flanges and in "thick" flanges using welding processes commonly employed in the steel fabrication industry. GMAW, SMAW, FCAW and SAW processes were chosen for testing. Exactly the same welding consumables and welding parameters were used on the A36/A572 Gr 50 dual grade steel as on A36 steel. Actually, the welders were not aware that two different steels were being welded in the program. Consumables for each process were chosen that are proper for either A36 or for A572 Gr 50 steel. Tests were conducted in accordance with the weld qualification requirements and procedures of AWS D1.1 (1992) as a minimum.

Details of the test procedures and test results are presented in a separate test report. Conclusions from these data are presented below:

Weldability of the two steels is the same. Welds on the two steels (A36 and A36/A572 Gr 50 dual grade) using identical electrodes, fluxes, and welding parameters produced acceptable, equivalent welded joints. Welds were made utilizing SMAW, GMAW, FCAW, and SAW techniques and equipment. No difference in the welding process on the two steels was encountered by the welders. For weldability purposes, the two steels are interchangeable.

The weld program produced no weld cracking. Radiographic inspection revealed only isolated areas of light porosity. All bend specimens were acceptable. Tensile specimens all fractured in the base metal rather than in the weld or in the heat affected zone. Charpy impact tests at temperatures ranging from -20 F to 75 F showed no difference in behavior for welds in thin flanges. Welds in thick flanges exhibited slightly lower impact properties for dual grade steel than for A36 steel over the range of temperatures. Rockwell hardness readings revealed no harmful hard zones in any of the welds. Based on these data, it is appropriate for the user to utilize his usual welding procedure for the specified steel when welding A36/A572 Gr 50 steel.

R. J. Schiltz, Jr., Ph.D, P.E.
AADFW, Inc.
Euless, TX

Another response:

The following is in reply to the two questions regarding welding:

1. When welding a steel that has dual certification (A36 and A572 Gr. 50), is there a low hydrogen electrode requirement?

Table 4.1 of AWS D1.1 gives the requirements for filler metal/base metal combinations for *prequalified* welding procedures (WPS's). ASTM A36 is listed in Group I and A572 Gr. 50 in Group II. If one steel meets all the requirements for both materials classifications, it is reasonable to require the WPS's to meet all the requirements that would be applied to welding on either material. In the particular example, this would preclude the use of non-low hydrogen electrodes on this particular steel However, I do not believe this issue has been formally addressed the by D1 Committee.

WPS's maybe qualified by test. This approach could be used to permit the use of the same electrodes permitted to be used on A36 to be applied to A572 Gr. 50. It should be noted, however, that the tests used by D1.1 for WPS Qualification do not duplicate the restraint commonly associated with actual fabrication.

2. Is AWS D1.1 Table 4.1, Note 1, regarding joints involving base metals from different groups, applicable to this condition?

No. The purpose of this footnote is to address filler metal requirements for joints that involve two separate base metal groups. The strength of the filler metals employed need only match the requirements for the lower strength steel, although the filler metal must in all circumstances be low hydrogen.

Duane K. Miller, P.E.
The Lincoln Electric Company
Cleveland, OH

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

The design and installation of cranes is governed by the Crane Manufacturers Association of America's (CMAA) Specification #70, Specifications for Electric Overhead Traveling Cranes. Section 1.4.3 of this document states, in part:

"The lateral deflection (of the crane runway) should not exceed L/400 based on 10 percent of maximum wheel load(s) without impact. The vertical deflection should not exceed L/600 based on maximum wheel load(s) without impact. Gantry and other types of special cranes may require additional considerations."

In the absence of local building code requirements that are more stringent, the designer of an overhead crane installation should follow the CMAA requirements.

David Duerr, P.E.
2DM Associates, Inc.
Houston, TX

New Questions

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

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

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

What are tolerances for cambered members? When is it proper to cold camber or curve a member and when is it necessary to use heat?

When erecting steel beams on a brick wall, could the non-shrink gourt be omitted under a proper bearing plate, if the surface of the brick is smooth, clean of any and all debris and leveled?

Yaakov Roth
Brooklyn, NY

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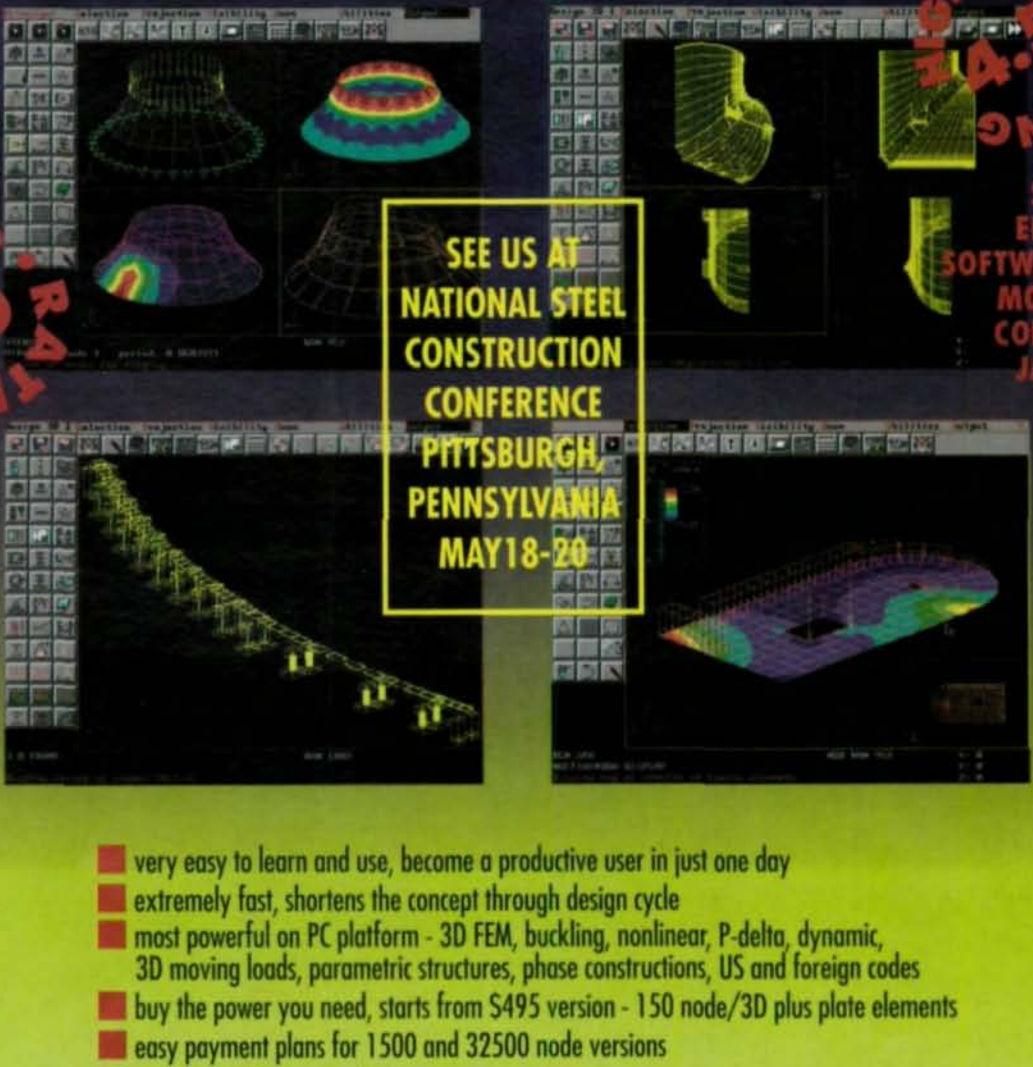

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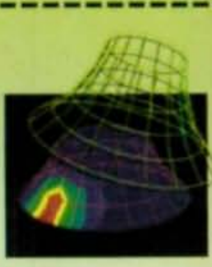

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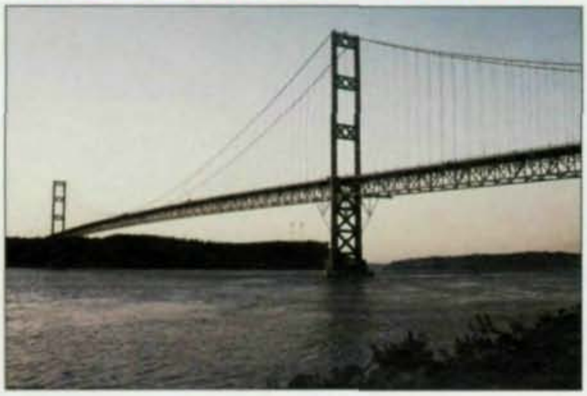

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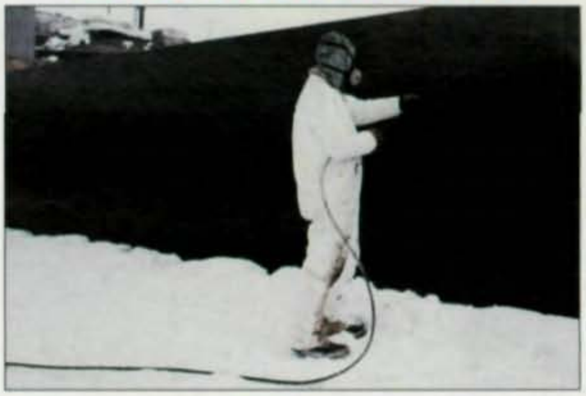

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Assessing Steel Damage In The Northridge Earthquake

Interim observations and recommendations of the AISC Special Task Committee on the Northridge Earthquake

A technical meeting was organized by the American Institute of Steel Construction, Inc. (AISC) on March 14 and 15 under the auspices of the AISC Committee on Specifications TC 113 on Seismic Design and the direction of its chairman, Egor Popov. Its objective was to assess the impact of the January 17, 1994 Northridge Earthquake on structural steel building frames. About 30 individuals participated in this professional exchange of information and discussion. The participants represented a wide variety of disciplines and expertise in seismic design, research, welding, steel construction, and code/regulatory bodies. In particular, the input from several prominent California consulting offices directly involved in related work provided valuable information.

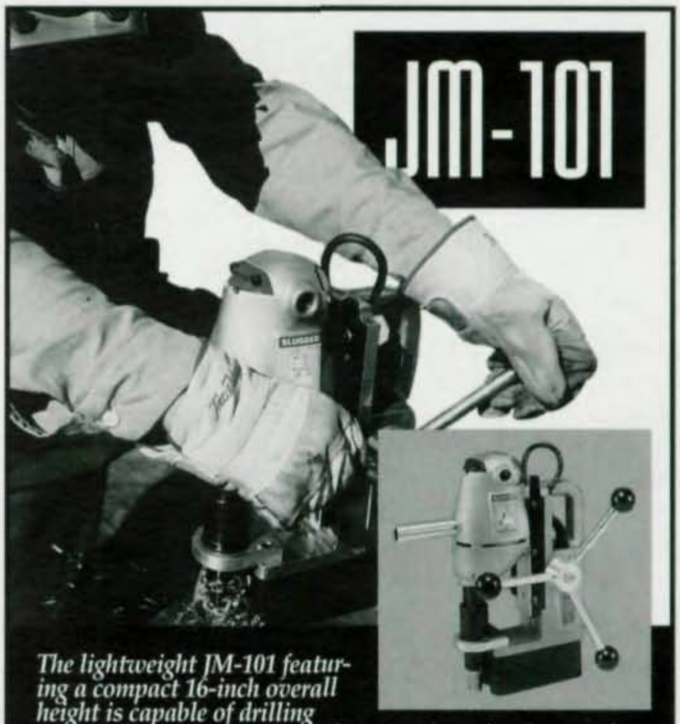
This Executive Summary of the AISC Special Task Committee meeting provides an overall description of the available damage information, evaluates its implications, and recommends several possible remedial alternatives. A more detailed report will be published in the June issue of Modern Steel Construction and the complete report will be available from AISC (send $5 to AISC-Earthquake Report, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001). These comments are offered only as interim general advice and are not intended to supersede, but rather to complement, any specific structural repair measures required or recommended by the responsible code jurisdiction, building owner, and/or the owner's engineering consultant. Appropriate research will soon be undertaken by AISC and other agencies to more fully explore the concerns raised by the unexpected structural steel damage and to develop definitive procedures for the connection designs in question if necessary.

It is important to emphasize that, although the Northridge Earthquake produced some extremely strong ground motions, there were no collapses of any steel framed buildings and no fatalities. In some cases these ground motions exceeded code required design spectra by a factor of more than two-and-one-half. There was no accurate count of steel-building related damage at the time of the meeting but various reports indicated that localized structural damage had been identified in 12 to 20 buildings (and current estimates now exceed 40 buildings).

Observations

A majority of the reported problems were in special steel moment frame connections that consisted of horizontal girders with welded beam-to-column flanges, bolted shear plates, sometimes with additional fillet welds connecting beam web to shear plates. Many of these special moment frames were arranged in two bay configurations that only partially enclosed the building perimeter. The yielding was intended to occur in the beams away from the connection. However, fracture in the region of this field welded connection, and some accompanying fracture of the column, occurred mostly in the vicinity of the beam bottom flange connection prior to overall beam yielding. Cracking was initiated in the region of the flange weld near the root at the back-up bar and then propagated in to the adjacent supporting column or the beam flange weld. This primary bottom flange fracture was accompanied, in some cases due to the force redistribution, by secondary cracking of the beam web shear plate, fracture of the web plate bolts, and/or top beam flange damage. There was no other observable general pattern to these moment connection failures. Reported structural damage ranged from none or relatively few to many connection fractures per floor level, and included mostly low and some high-rise buildings.

Various opinions and considerations were expressed on the observed problems. Although uncertainties about the effects of site specific ground motion persist, the primary direct causes of these connection fractures were attributed to the unexpected high stress concentrations in the beam-flange-to-column connection. This rendered the complete joint penetration flange welds more susceptible under cyclic load reversals to crack propagation originating from any initial notches, inadequate fusion/penetration, or other imperfections, including the naturally occurring notch-like condition that results from a properly fused but left-in-place steel backing bar. Despite initial appearances of connection symmetry, the bottom beam flange weld was consistently more critical than its top counterpart. Several feasible explanations have been proposed for these differences: increased



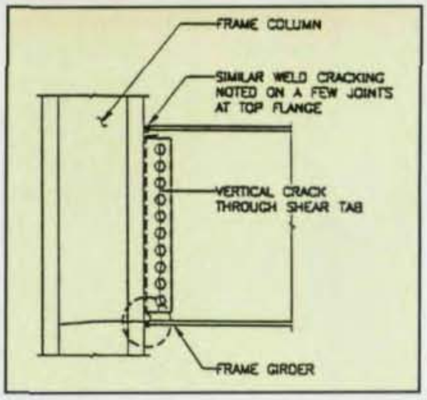
sensitivity from the weld root at the lower extreme fiber; welding workmanship difficulties, such as beam web interference (cope holes); and the additional presence of beneficial restraint at the top in the form of the floor slab and web plate connection. Indications are that the steel beam and column material complied with applicable ASTM requirements. However, A36 material overstrength factors as high as 1.75 combined with lower tensile ultimate to yield ratios could have unexpectedly limited the beam yielding to be concentrated within the welded and coped portion of the beam.

Some concentrically braced frames were observed to also experience structural damage from the Northridge Earthquake. The nature of these problems were fractures of the relatively thin-walled tubular braces, their gusset plate or column base connections. These designs were completed under a previous code. The 1992 AISC Seismic Provisions, and other newer codes and standards, require substantially thicker walls and improved gusset details for better ductility.

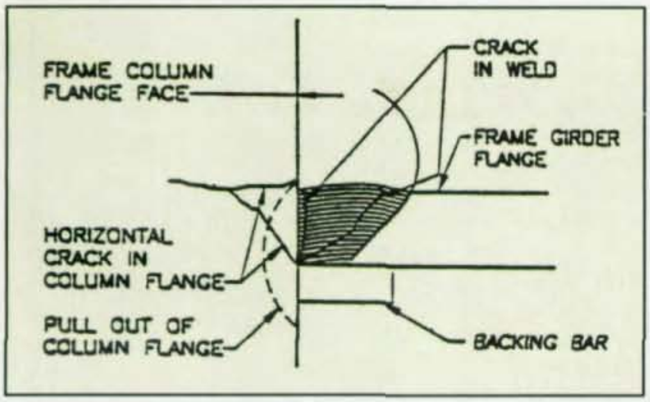
Evaluation

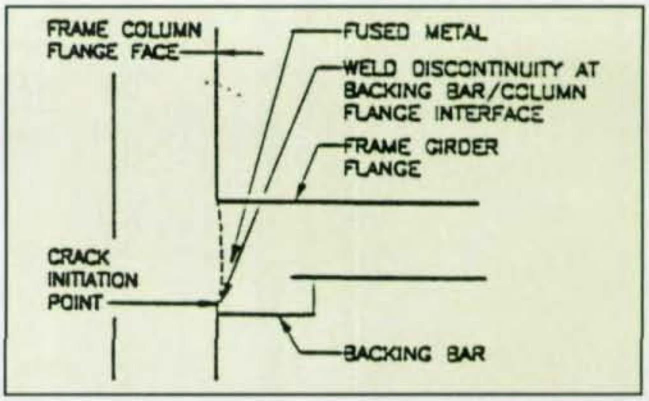
All steel frames, included those that were structurally weakened, successfully survived the Northridge Earthquake and its aftershocks without a catastrophic collapse. The residual strength of the damaged lateral resistance frames, combined with the incidental moment capacity of the shear connections of the entire gravity system, their inherent ductility, and modified frequency/response characteristics are likely to have contributed to their survival. Almost all the known damaged steel buildings are being repaired.

In many cases, external appearances or the extent of non-structural damage, i.e. glass, facade, interior partitions, and ceilings, were not a reliable indicator of the actual structural condition of existing steel buildings within the affected region of the Northridge Earthquake. Of course, buildings that suffered an inordinate amount of external or internal damage, and buildings



Pictured, from left to right: A typical joint elevation; different observed joint failures; and a schematic of fused metal at a full penetration weld.Courtesy of Englekirk & Sabol.





with permanent lateral drift (set) were carefully evaluated by qualified engineers to check the steel connections and main members within the lateral framing system that were obscured by the building enclosures and fire protection system.

Structural damage that is not identified and adequately repaired could significantly lower any building's seismic resistance, thereby increasing the hazard in future earthquakes. In areas of strong ground motion risk, more accurate technical assessments should be made of suspected buildings on an individual basis and in the context of an overall review of the potential seismic vulnerability and life safety of existing construction.

Repair Of Special Moment Frames

Any cracked steel or weld metal in the beams, columns or connections should be removed and replaced by new steel material and/or by welding. In addition, until needed research studies are performed, the following alternatives are offered as general guidelines for the repair of any damaged steel beam-to-column connections.

1. Use weld tabs at beam flange connections; after welding, remove the weld tabs and finish to a smooth contour per 3.12.3 of AWS D1.1-94; in addition, remove the backing bar after welding, back-

gouge root to sound metal, weld backgouged region and finish welding using a reinforcing fillet weld, according to 3.13.4 of AWS D1.1-94. (These provisions normally apply to fatigue prone structures but are recommended for seismic design until research is completed).

2. Weld beam web directly to column or weld shear plate connection to beam web without overstraining existing flange welds.

3. Add steel flange plates to the beam-flange-to-column connection.

The selection of one or more of these possible alternatives and their details is dependent on the extent of the damage, accessibility, and the judgement of the responsible engineer. These are intended to perform as rigid connections that will develop the supported beam's actual bending capacity (plastic moment, including any overstrength) or the column panel zone in order to enable ductile yielding of the special moment frame system under severe cyclic loads. The expected strength ratios of the beam and column material should be considered. Extra care is suggested for the repair work to ensure high quality workmanship, materials and performance. Good welding practice must be followed, including all the requirements of AWS D1.1-94, such as qualified welders (5.3), preheat (4.2), technique (section 4, parts B, C and D), inspection (section 6 and section 8, part D) and the use of electrodes capable of depositing notch-tough weld metal, such as found in AWS A5.1, 5.5, 5.20 and 5.29. In addition, any replacement or reinforcing steel should have adequate notch toughness.

New Construction Of Special Moment Frames

Based on this new evidence and pending additional research, further use of the original welded flange and bolted web special moment frame connection in the 1992 AISC Seismic Provisions Sect. 8.2.c., or equivalent, should be supplemented by the previous repair recommendations. In the absence of any revised and more definitive design criteria for new construction, these repair guidelines, together with more lateral system redundancy, can serve as reasonable provisions for future special moment frames. Fabrication should be in strict compliance with the applicable codes.

Concentrically Braced Frames

New construction, as well as replacement of damaged bracing members and their connections, should be in accordance with the requirements of the June 1992 AISC Seismic Provisions for Structural Steel Buildings Sect. 9, or equivalent, which is perceived to contain the best available criteria for this topic. Column bases should be designed to accommodate bracing system forces and deformations. Fabrication should be in strict compliance with the applicable codes.

AISC, its committees, and the steel industry are quickly responding to the public and professional needs triggered by the Northridge Earthquake. Seismic design criteria will be reviewed and addressed, as necessary, by AISC and other organizations in light of the results of ongoing research progress and project data, which is being expeditiously pursued. We are all working to retain the utmost public confidence in steel construction, which has historically provided superior life safety and structural performance during earthquakes.

By the Committee:

Egor Popov, Chairman, Univ. of Calif.—Berkeley; Roy Becker, Becker & Pritchett; Omer Blodgett, Lincoln Electric Co.; David Bonneville, Degenkolb Assoc.; A.L. Collin, Consultant; Ross Cranmer, City of Huntington Beach; Rob Culp, Culp & Tanner; Tom Culp, Culp & Tanner; Susan Dowty, I.C.B.O.; Michael Engelhardt, Univ. of Texas—Austin; Robert Englekirk, Englekirk & Sabol; Roger Ferch, Herrick Corp.; John Fisher, Lehigh University; Jerry Haaijer, AISC; Richard Holguin, LA Building Bureau; Roy Johnston, Brandow & Johnston Assoc.; Robert Lorenz, AISC; Jim Malley, Degenkolb Assoc.; Hank Martin, AISI; Duane Miller, Lincoln Electric Co.; Lowell Napper, Structural Steel Educational Council; Dave O'Sullivan, EQE International; C.W. Pinkham, S.B. Barnes Assoc.; Robert Pyle, AISC Marketing; Thomas Sabol, Englekirk & Sabol; John Shipp, EQE International; Jack Skiles, Omaha Public Power Co. (AWS-D1.1 Chm.); Raymond H.R. Tide, Wiss, Janney, Elstner & Assoc.; Fred Turner, Cal. Seismic Safety Commission; N.F.G. Youssef, Youseff & Assoc.; Nestor Iwankiw, Secretary, AISC.

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$\frac{dM}{d\theta_r} = K_r = K_i\left[1-\left(\frac{M}{M_u}\right)\right]^c$

$f = 1.57\sqrt{\frac{g}{WN_{eff}}\left(\frac{EI_tN_{eff}}{L^3}\right)}$

Redundancy in Highway Bridges

$M = P\frac{L}{2}\left(1-\cos\frac{2\pi x}{L}\right) - P\frac{L}{2} = -P\frac{L}{2}\cos\frac{2\pi x}{L}$

Design of Lifting Beams

Building Floor Vibration

Flexural Strength of WT Sections

$F_{cr} = \frac{\pi^2 E}{\left(\frac{K_xL}{r_x}\right)^2}$ $F_{cr} = \frac{\pi^2 E}{\left(\frac{K_yL}{r_y}\right)^2}$ $F_e = \frac{1}{A\bar{r}_o^2}\left[\frac{\pi^2 EC_w}{(K_zL)^2} + GJ\right]$

$K_i = \sqrt{\frac{\pi^2 EI}{P_y L^2}}$

$S = \left[\left(1-\frac{\Sigma(P/L)}{5\Sigma\eta}\right) - \Sigma\left(\frac{P}{L}\right)\left(\frac{\Delta_i}{\Sigma H}\right)\right]S_i$

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"Down the road, weathering steel

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

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

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

WHY WEATHERING STEEL?

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

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

OTHER STRONG REASONS.

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

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

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

Quite simply, weathering steel is a natural for a

will save us more than money."

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

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Bethlehem

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Design Consultant: URS Consultants, Akron, OH
General Contractor: National Engineering and Contracting Company, Strongsville, OH
Steel Fabricator: High Steel Structures, Inc., Lancaster, PA
Steel Erector: Middle States Steel Construction Company, Eighty Four, PA
Weathering Steel Supplier: Bethlehem Steel Corporation, Bethlehem, PA

Innovative Design Of Gable Frame Buildings

New design techniques allow gable frame buildings to be cost competitive with metal buildings

This article is based on a paper presented at the 1994 National Steel Construction Conference by ***Thomas Sputo****, Ph.D., P.E., a consulting engineer headquartered in Gainsville, FL.*

Conventional wisdom holds that traditionally designed gable frames fabricated from rolled sections cannot compete, from a cost standpoint, with metal building system frames fabricated from plate and bar stock. However, five recently bid projects—two of which have been built—cast doubt on that viewpoint.

Metal buildings constitute almost 60% of the low-rise, non-residential construction market and are chiefly utilized where economy is a major concern. The usual procurement for these buildings is to contract with a company that specializes in the design and fabrication of metal building systems. This may also provide erection services, though it is more common that the company simply provides building components for erection by another subcontractor.

The typical metal building system utilizes rigid steel frames as the main force resisting system. These frames support, in turn, purlins and girts, and wall and roof sheeting. Most metal building manufacturers fabricate their frames from plate and bar stock that is cut to size, then welded into I-shaped sections of constant depth, or as is more typical, of varying depth (tapered sections).

The dominance of metal buildings is a fairly recent phenomenon and is usually attributed to their cost advantages. It was only in the 1960s that lower cost metal buildings began replacing the more traditional gable framed low-rise buildings.

A gable frame fabricated from W16 sections provided an 80-ft. clear span for a gym at the Newberry Middle School at a lower cost than a similarly sized metal building.

Recently, however, a small fabricator/erector in Alachua, FL, was able to bid five gable frame buildings (frame, purlins and girts), fabricated from rolled sections and with a maximum span of more than 108 ft., at a substantially lower price than any supplier could provide a metal building system. The cost savings in this "new breed" of gable framed buildings was made possible by utilizing several innovative design and fabrication concepts:

1. LRFD design for cases where dead loads are relatively heavy.

2. Use of direct second order analysis to eliminate the need to calculate second order effects.

3. Use of a "pseudo lateral load" design strategy that allows compression members to be designed for an effective length factor (K) equal to 1.0.

4. Utilizing to the fullest extent possible standard stock lengths to minimize lengths.

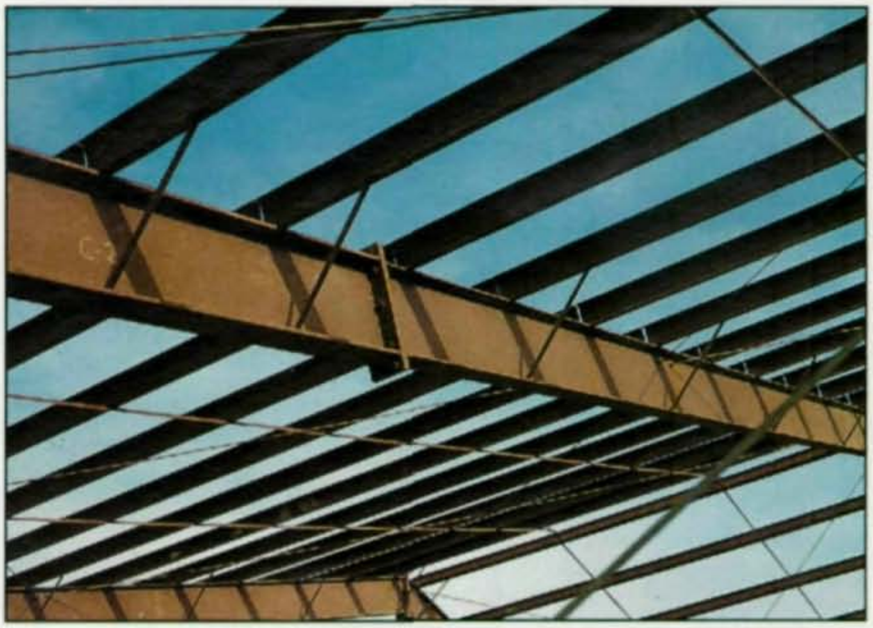
5. Innovative connection details to reduce waste and speed fabrication and erection.

Load And Resistance Factor Design

The building systems were designed using the 1986 LRFD Specification. The LRFD Specification is nominally calibrated to provide designs roughly equivalent to the Allowable Stress Design (ASD) method for live to dead load ratios of 3 to 1. In cases where the ratio is less than 3 to 1, LRFD provides some material economies. Because the buildings in question here had severe mechanical, roofing, and insulation loading requirements, the load ratio was closer to about 1 to 1, which allowed economies as compared to ASD designs. Most, if not all, metal building manufacturers are still using ASD exclusively for their buildings, and are therefore missing out on some real advantages.

Direct Second Order Analysis

Computer programs that will perform a direct geometric second order structural analysis of a planar structure are readily available today. The LRFD Specification allows designers to include second order effects directly in the analysis and to then bypass this part of the design calculation. These frames were designed using this option, which may produce some small economies. I'm currently using RISA 2D for the analysis part.



Pictured at right is a 108-ft. clear span frame, fabricated from W24 sections, for the cafeteria/auditorium at the Newberry Middle School. Shown above is a mid-rafter change in the cross section of the gable frame.

However, RISA doesn't support LRFD, so using the forces calculated by the program and perform an LRFD design by hand.

One advantage of this option is the ability to have ready access to the predicted second order deflections of the structure. A second advantage is the economy of design effort required by the designer, since moment magnification factors do not need to be calculated. The determination of these magnification factors is not as straightforward for gable frames as it is for rectangular frames.

Pseudo Lateral Load Design Strategy

The Canadian limit states code does not permit the use of effective length (K) factors for the design of unbraced steel frames. Instead, it requires the designer to apply a small perturbing force to the gravity loaded frame and to then design the frame using effective length factors equal to 1.0. (For more information on this method, see "Column Design In Gravity Loaded Frames" by Cheong-Siat-Moy in the *ASCE Journal of Structural Engineering*, Vol. 117, No. 5, 1991.)

The use of this alternate method is not prohibited by Section C1 of the LRFD Specification.

Fabrication And Erection Economies

The most innovative and exact design methods cannot overcome inadequacies that make a structure difficult to fabricate and erect. In order to exact the most

Pictured is one of the three buildings at the Prairie View Elementary School, which were fabricated from W12 sections.

economies possible, close dialogue between the fabricator and designer are necessary.

For many small jobs, such as these frames, fabricators purchase materials from steel service centers in stock lengths of 30 ft., 40 ft., 50 ft. or 60 ft. It is therefore important to consider not the total weight of steel in the structure, but instead the total weight of steel that must be purchased, including that steel which ends up as scrap. Locations of splices and changes in cross section must be considered in this light.

Connection details are an integral part of the design. After some trial and error, we developed a detail where the knee joint is welded in the shop, thereby connecting the first rafter segment to the column. This method works well from a material and labor economy standpoint. Due to the shape of the erected column and rafter segment, I refer to this method as the "Gamma Column Framing System." A similar column construction was shown in a 1940 U.S. Patent assigned to Butler Manufacturing Co.

After the first two projects were completed, additional ideas for further economies were developed. It was discovered that in some instances it may be cost effective to add cover plates to the inner flange of the column or rafter at points where the allowable moments are exceeded for a short distance.

Usually this occurs at the knee or the midspan of the rafter. By placing the cover plate on the inner flange, it will not interfere with purlin or girt placement. LRFD allows for greater capacity than ASD for monosymmetric sections, which is an additional plus for LRFD. The additional labor for attaching the cover plate must be weighed against just using a heavier rolled section for the entire length.

For the two projects constructed, all sections in a frame were of equal nominal depth and it was possible to construct a bolted moment end splice between sections of unequal depth. This would allow for lighter and shallower sections for the rafter section in many cases.

Prairie View Elementary School Addition

The first project, in August 1992, was to supply and erect frames, purlins, and partial girts for three "metal buildings" that were to be incorporated into a larger overall building in Gainesville, FL. The project specifications and contract drawings were written around a metal building system by the project architect and project structural engineer. Frame clear spans were 58 ft., 43 ft. and 30 ft. Bay widths ranged from 19 ft.-6 in. to 30 ft. Roof pitch was 4 to 12. The project architect would not allow the use of diagonal bracing, portal frames, portal beams, wind columns or sway frames for lateral stability of the building perpendicular to the plane of the frames. This requirement was dictated by architectural considerations and "daylighting".

Design loads were dictated by the 1988 Standard Building Code. Design wind velocity was 90 mph with a use factor of 1.15. The design of these frames was governed by gravity loads, not wind loads.

The "no bracing" requirements required some foundation redesign on our part.

Building lateral stability perpendicular to the frames was provided by using a fixed end condition for the column bases and using the columns as cantilevers in the weak direction. This increased the required section appreciably and required a column type section as opposed to a more usual beam type section. Since we were fixing the bases in the weak direction, we made the decision to also fix the bases in the strong direction, thereby regaining some of the advantage that we lost when a more typical bracing scheme was prohibited.

This first project was part of a learning curve. While I did perform a second order analysis and did use LRFD, I did not use the pseudo lateral load strategy that required the calculation of effective length factors using the nomographs.

Simple span W8x10 purlins were used, with purlin bracing provided by L1x1x1/8 angles applied to both the top and bottom flanges. Top flange bracing was required because the standing seam roof system to be applied by others would not provide the necessary lateral bracing. Frame flange bracing for the inside flange was accomplished using L1x1x1/8 angles welded in place as flange stays.

Newberry Middle School

The second project was to provide frames and purlins for a gymnasium and cafeteria for a new middle school campus in Newberry, FL. The design was completed in November 1992, with fabrication and erection completed by March 1993. The design loads were similar to Prairie View, but the wind resisting system was different. Frame drift requirements were set at H/300 by the specifying engineer. However, this was not listed as a requirement in the project specifications, but was later required by the specifying engineer to prevent distress to the masonry walls. H/300 relates to span/600, which is a common deflection criteria for masonry construction.

Here, as at Prairie View, wall bracing was prohibited, but masonry walls were provided to act as shear walls in the out of frame direction. Frame spans for the gymnasium were 80 ft. and spans for the cafeteria were 108 ft. Roof pitches were 4 to 12.

The purlin systems were W8 sections constructed in a continuous system using some cantilevers. This system was chosen to utilize completely the purlin stock lengths. The initial scheme was to set the purlins with a small crane, and then to make the bolted splice while working on a rolling platform. Unfortunately, this method of making the splices had to be abandoned in the field when the general contractor on the job would not allow any equipment on the floor slabs. As a result, the ironworkers had to shimmy out on the very flexible purlins to make the splices, at a high cost in additional time. In the future, we will not place splices anywhere except over frames because of the uncertainty of being allowed to work off the floor slab.

The frames for the gymnasium were fabricated out of W16 stock, using the previously referred to "gamma column" system. The frame components were laid flat on the ground outside the building perimeter and bolted together. Once connected, the frames were lifted in one piece and set into place.

Being able to make the bolted connections on the ground saved much time and effort during erection. Because of the additional lateral stiffness of these frames made of rolled sections as opposed to more weight economical tapered sections, we were able to accomplish this one piece pick. It is doubtful that this lift could be accomplished for a tapered frame of the same span due to buckling considerations.

The cafeteria frames were fabricated from W24 stock, also using the "gamma column" system. Here the spans were too big to safely lift an entire frame, so the columns were set first with wood blocking under the base plate and leveling nuts, then the rafter was set into place.

The outriggers were set atop the frame knees and welded into place. Also, welded flange stays and welded continuous angle bridging for the purlins was provided. However, for cost reasons, in the future we will recommend bolted outriggers and bolted stays.

Conclusions

By utilizing innovative design, fabrication and erection techniques, custom fabricators may find it economically feasible to custom design gable frames to compete with metal building frames on projects where the frames are a small portion of the overall job.

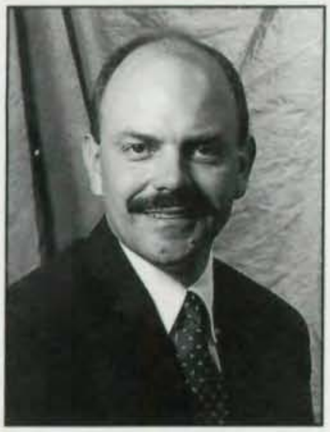
Additionally, design engineers now have the option of designing economical gable frames out of rolled sections to recapture the "classic" look for frames from the 1930s and '40s.

In the cases of both projects constructed, the design was between 5 and 10% below the lowest bid from a metal building manufacturer.

While the system described is most economical for spans up to around 60 ft., it can be economical for greater spans under certain circumstances, specifically when the roof pitch was steeper than the usual 1 to 12. Experience has shown that 4 to 12 is a good compromise, but can become unreasonable for long-span buildings for other than structural reasons.

Reducing Serviceability Concerns

Partially restrained composite beam-to-girder connections can reduce design moments and deflections while improving vibration characteristics



This article was condensed from a much more detailed paper presented at the 1994 National Steel Construction Conference by ***Clinton O. Rex****, Ph.D., a Via Fellow in the Charles E. Via, Jr., Department of Civil Engineering at Virginia Tech and* ***W. Samuel Easterling****, Ph.D., P.E., an associate professor in the same department.*

Owners consistently want more and more open space in buildings and during the past four decades steel designers have made three important changes to accommodate this demand. First, composite steel-concrete floor system technology has developed that allows designers to use the synergy of tying the two floor components (beam and slab) together in order to span longer distances. Second, the plastic section analysis and design procedures utilized in the Load and Resistance Factor Design (LRFD) methods have allowed an additional increase in span length over designs that utilize Allowable Stress Design (ASD) methods. And third, A572 Grade 50 steel is becoming more readily available at a cost comparable with A36 steel.

The benefit of these changes has been longer and shallower floors, which translates into more open space. The disadvantage is an increasing concern with serviceability issues, such as floor deflections and vibrations. In many cases, serviceability issues, rather than strength considerations, often control designs.

One method of improving serviceability characteristics is by providing partial continuity in a composite floor system. The advantages of a continuous beam over a simply supported beam include reduced design moments and deflections and, in some cases, improved vibration characteristics.

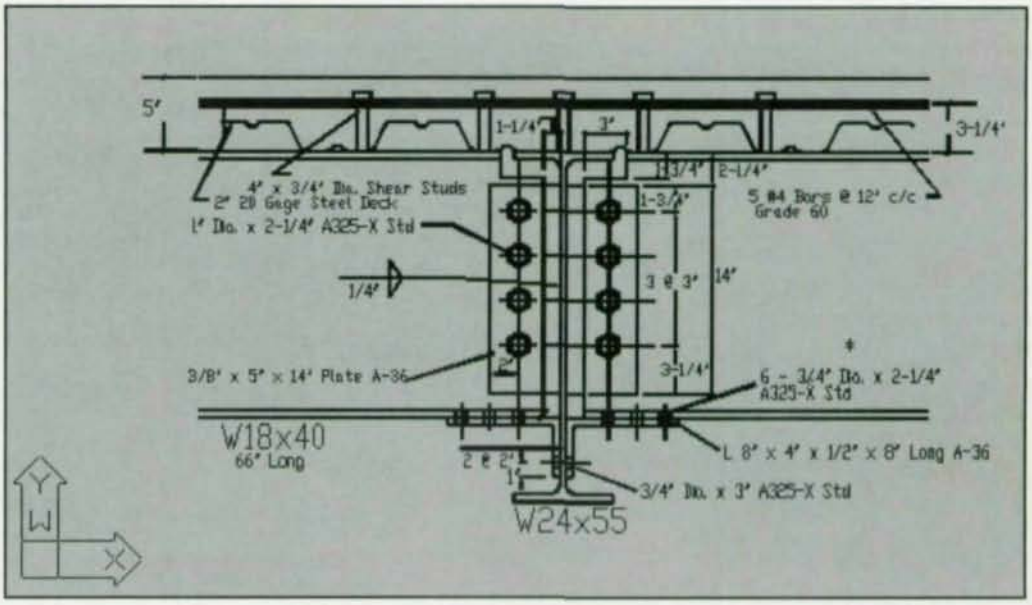
Studies have shown that some continuity can be provided through the use of partially restrained connections. While research has been conducted on beam-to-column connections, partially restrained composite beam-to-girder connections have not previously been tested. In a new test, four beam-to-girder specimens were studied.

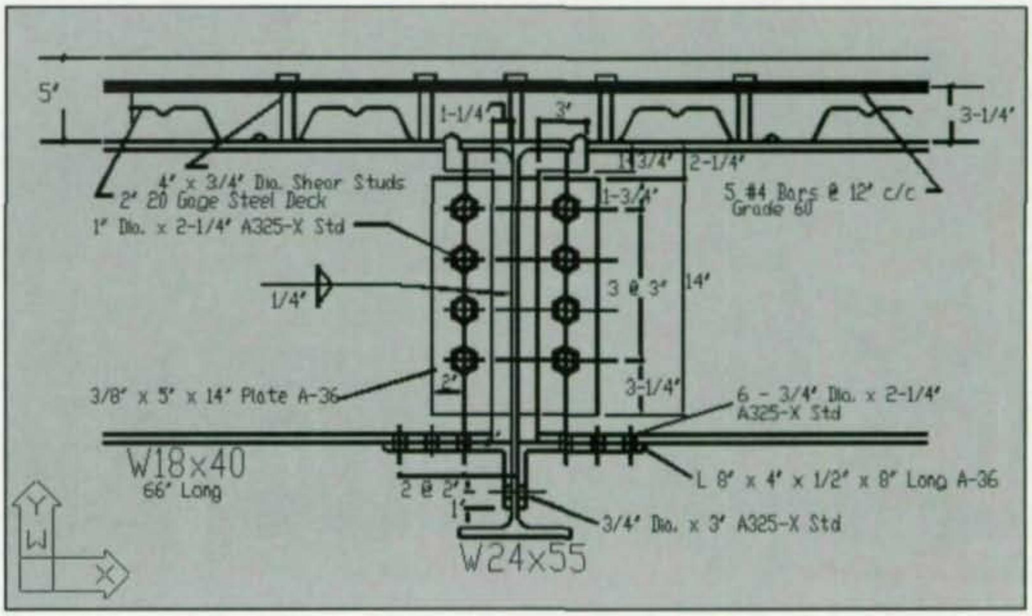
Test Results

Connection #1 was a single plate framing connection. The experimental results indicated that the bare steel connection had little rotational stiffness but that this connection became very stiff when it was combined with a reinforced composite slab. In fact, the composite connection developed nearly 37% of the plastic moment capacity of the steel beam. This connection failed as a result of lateral buckling of the bottom beam flange.

Attaching a seat angle to the bottom beam flange was shown in research on composite semi-rigid beam-to-column connections to increase rotational stiffness and provide stability for the bottom flange. With this in mind, Connection #2 was detailed similar to the first, but with the addition of a seat angle. This connection showed significant stiffness with and without a composite slab. Although the bare steel connection was not loaded to failure, it was shown to develop a moment capacity of at least 20% of the plastic capacity of the steel beam. The composite connection developed a moment capacity of approximately 80% of the plastic moment capacity of the steel beam and had a rotational capacity in excess of what would be needed in typical composite beam designs.

Connections #3 and #4 were two innovative designs developed in an attempt to increase the rotational stiffness of the beam-to-girder connections prior to placement of the composite slab. Both connections were combinations of set angle and a tension plate, which was used to attach the top portion of the beam to the girder. Both of the bare steel connections exhibited very stiff moment-rotation behavior. The composite connections were also very efficient, attaining nearly 80% and 100% of the plastic moment capacity of the steel beam for connections #3 and #4, respectively.





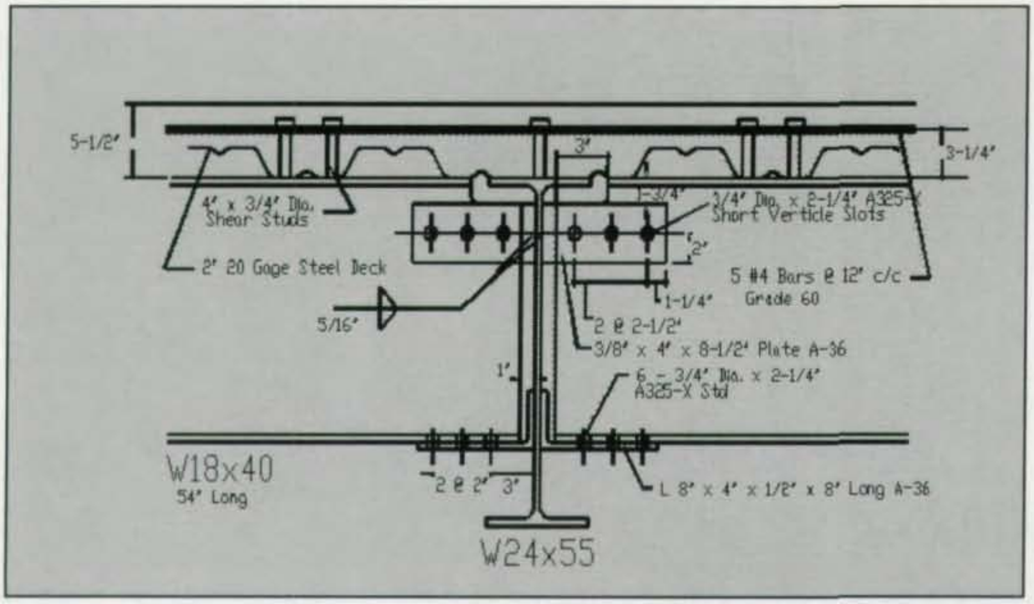
Shown above: Connection details #1 and #2.

Conclusions

The behavior exhibited by the composite beam-to-girder partially restrained connections tested indicated that simple steel beam-to-girder connections, which may not have any significant rotational stiffness on their own, can be turned into very stiff connections with the addition of a reinforced composite slab.

Additional conclusions include:

•A partially restrained connection created by combining a simple steel connection, such as a shear tab, and a reinforced composite slab can lead to insta-



Shown above: Connection detail #3.

bility such as lateral buckling of the beam bottom flange. To ensure that instability does not occur, and at the same time increase the rotational stiffness of the connection, a seat angle or a plate needs to be attached to the bottom flange. If the angle or plate is not provided, then detailing of the reinforcing steel should be given careful consideration (i.e., the amount of reinforcing steel should be detailed so that the reinforcing steel yields prior to the occurrence of any local instabilities in the connection).

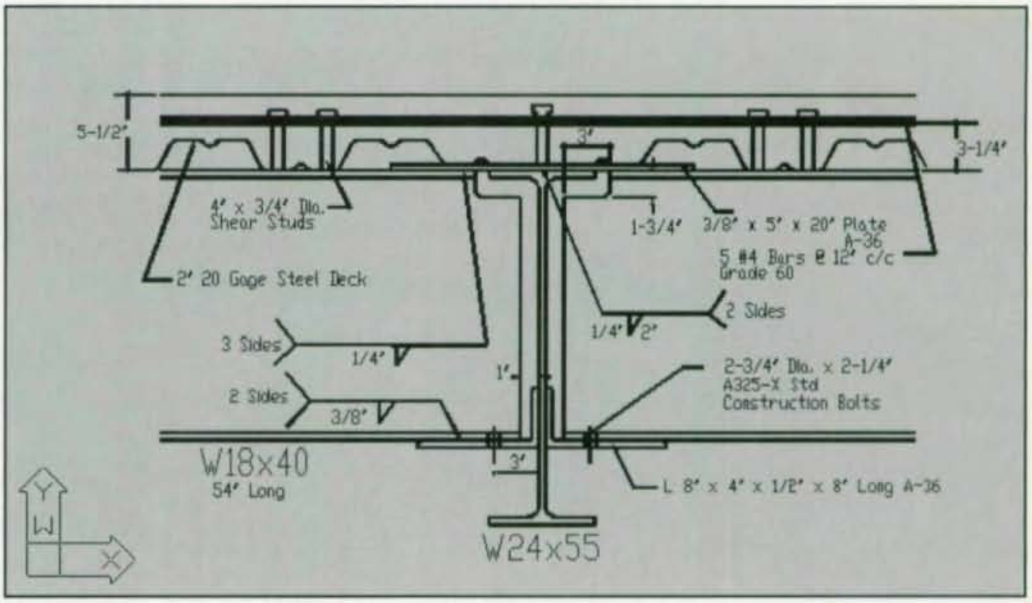
• Connections using fully tensioned bolts will typically be characterized by behavior that is stiffer and more predictable than connections that use only snug tight bolts.

• To ensure that the connection has sufficient ductility, the details of the steel connection need to be given careful consideration. If the steel connection is too stiff and does not allow the reinforcing steel to yield significantly, then the connection will likely fail as a result of local instabilities at relatively low rotations.

• Reinforcing steel should be placed within a slab width of approximately 60 in.

• Accounting for the rotational restraint provided by these connection should lead to decreased deflections and moments. This, in turn, should allow more efficient designs and possibly an eventual reduction in construction costs.

Future studies will focus on a limited number of connections, which will be evaluated and developed in great detail. In addition to engineering studies, a general economic study will be conducted.



Shown above: Connection detail #3.

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Name on card: ______

NSCC 94 "STEEL STANDS FOR THE FUTURE"

AISC brings its 1994 National Steel Construction Conference to Pittsburgh, Pennsylvania. The NSCC will be held at the David L. Lawrence Convention Center on May 18-20, 1994. Attendees will be housed at the Pittsburgh Vista Hotel—the official NSCC hotel—and the Westin William Penn. The Conference will focus on the specific interests of structural steel fabricators, engineers, architects, owners, contractors, erectors, detailers, researchers and educators. A comprehensive technical program includes seminars, meetings, product exhibitions, equipment demonstrations, optional family events, and receptions.

Designated the nation's most livable city, Pittsburgh offers many exciting options to explore. Visit the Carnegie Science Center, dubbed an "amusement park for the mind". Tour Clayton, a turn-of-the-century house museum opened to the public in 1990. Pittsburgh's Cultural District offers a variety of entertainment from the acclaimed Pittsburgh Symphony Orchestra, Broadway and Off-Broadway shows, dance and comedy performed in the revitalized historic Fulton Theater. Located in Point State Park, everyone will enjoy the Fort Pitt Museum, which brings the city's history to life or have an encounter with wildlife at the Pittsburgh Zoo's Tropical Forest Complex, where visitors can experience a fog-shrouded forest.

WHY SHOULD YOU ATTEND THE 1994 NSCC?

NSCC is the only "all steel" conference and trade show in the country. At no other meeting will you receive more information about the design and construction of fabricated structural steel.

Education—The more than 40 technical seminars will educate and inform you.

Value—Attending the NSCC will save you money and time in the future. Everything you learn at the NSCC is "take-home" knowledge to be shared with your colleagues and used to improve your business.

Networking— Meeting and mingling with your peers can create future business opportunities.

State-of-the-Art— By attending the NSCC, you'll be "up-to-speed" on the new services and products in the steel construction industry.

Fun, Of Course— Visitors and spouses will have the opportunity to experience the unique and historical points of interest in this historical industrial city. In addition a delightful evening will be spent cruising the three rivers in a paddle wheel boat. A delightful dinner and entertainment will be provided.

NSCC COSPONSORS:

American Iron and Steel Institute; American Welding Institute; American Welding Society; Canadian Institute of Steel Construction; Mexican Institute of Steel Construction; National Erectors Association; National Institute of steel Detailing; Steel Deck Institute; Steel Joist Institute; Steel Plate Fabricators Association; Steel Service Center Institute; Steel Structures Painting Council.

SEMINARS AND TECHNICAL PROGRAMS

The Conference continues to be the premier meeting place for engineering professionals and the best place to obtain the most information about steel buildings and bridges. Workshop sessions get down to the nuts-and-bolts details of designing, fabricating, and erecting structural steel. Every aspect of the construction process from concept to completion receives attention: computerized design, codes and specifications, research, shop and project management, inspection and safety, and fabrication and erection procedures.

THE EXHIBITION

The NSCC offers an ideal marketplace for those who provide products and services to the structural steel industry. In addition to display booths, exhibitors also will be given an opportunity to conduct a Product/Service Workshop where exhibitors can share, at no charge, the latest technological advances in specialized fields, conduct demonstrations, and introduce new equipments and programs. The workshops will be conducted during time periods that do not conflict with regular Conference sessions.

Who Exhibits? —manufacturers of: fabricating machinery; welding equipment; bolts; fasteners; paints and coatings; erection equipment; other associations; computer software and hardware producers; etc. Information on exhibit space is now available from AISC headquarters; call the AISC Membership Services Department at 312/670-5420.

PRE-CONFERENCE EVENTS

This year's Schedule of Events includes an Educator Meeting on those issues of interest to the teaching of structural steel design, and an AISC Professional Member Forum for structural engineers interested in current and future AISC programs. Pre-conference events are open to all NSCC-attendees. Other organizations or associations wishing to schedule a pre-conference activity should contact Lewis Brunner at 312/670-5420.

MSC TO PUBLISH SPECIAL SHOW ISSUE

Modern Steel Construction, the monthly AISC magazine, will publish a special show issue in May 1994. This four-color publication has a circulation of 35,000 structural engineers, fabricators, architects and other construction professionals. The issue will include both product information from show exhibitors and reprints of technical papers submitted to AISC.

Advertising information can be obtained from John Byrne, Pattis/3M, O'Hare Office Plaza, 2400 E. Devon, Des Plaines, IL 60018. 708/699-6030.

SPOUSES' PROGRAM & OPTIONAL EVENTS

A special program for spouses and children of registrants is offered. See the schedule of planned evening and pre-and post-conference activities in this publication.

CLIMATE/CLOTHING

Pittsburgh's average May temperature ranges from 45 to 70 degrees. A light jacket or sweater may be needed. The dress code is medium, comfortable, and casual during the day, and sportswear for the evening. However, some restaurants require men to wear jackets & ties.

LOCAL TRAVEL

It is suggested that you use a metered cab with taxi rates averaging $2.25 for the first mile and $1.30 for each additional mile. A shuttle service offers transportation from Pittsburgh International Airport to downtown Pittsburgh. Adult fares range from $10 to $25 with children's fares at $5 to $15.

NSCC OFFICAL AIRLINES

The Travel Pros, Inc. and **U.S. Air** are offering the following for this conference:

—5% off the lowest applicable fare at the time of ticketing;

—10% off the Y26 fare (for attendees not staying over a Saturday night. These fares can only be obtained by calling The Travel Pros at **1-800-537-5448** or **708-668-7510** (Illinois). Agents are available 9:00 a.m.-6:00 p.m. (CST) Monday through Friday, and 9:00 a.m.-3:00 p.m. on Saturday. Be sure to mention AISC.

PRELIMINARY SCHEDULE OF EVENTS

MONDAY, MAY 16

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

TUESDAY, MAY 17

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

WEDNESDAY, MAY 18

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

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

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

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

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

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

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

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

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

THURSDAY, MAY 19

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

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

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

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

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

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

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

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

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

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

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

FRIDAY, MAY 20

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

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

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

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

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

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

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

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

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

SATURDAY, MAY 21

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

TECHNICAL SESSIONS

1 2nd Edition LRFD Manuals—Volumes I and II

Moderator: Bill Dyker, Garbe Iron Works, Inc.
Speakers:
Nestor Iwankiw, AISC
Charlie Carter, *AISC*

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

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

2 Effective Use of High-Strength Steel in Building Construction

Moderator: William Ashton, Egger Steel
Speakers:
Abe Rokach, AISC
Chia-Ming Uang, Northeastern University

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

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

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

3 Lean Engineering

Moderator: Bill Liddy, AISC Marketing, Inc.
Speakers:
Mark Holland, Paxton & Vierling Steel Co.
Engineer—TBA

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

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

4 Electronic Data Transfer: Now and Beyond

Moderator: John Bailey, Havens Steel Co.
Speakers:
Harry Moser, DuPont
Sayle Lewis, Fluor Daniel

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

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

5 Building Innovations

Speakers:
Tom Sputo, Consulting Engineer
Neil Wexler, P.C.

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

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

6

Quality Certification: Directions for the 90's

Moderator: Tom Schlafly, AISC
Speakers:
Tom Schlafly, AISC
TBA

The AISC Certification Program is in the process of focusing on the demands of the '90s. The aim is to provide the premier certification program that is relied on by builders and owners from coast to coast.

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

7

Steel Interchange—Connections

Moderator: Robert O. Disque, Besier Gibble Norden
Speakers:
Geoffrey Kulak, University of Alberta
Omer Blodgett, Lincoln Electric Co.

The popular *Modern Steel Construction* question and answer forum is here at the Steel Construction Conference. Two connection experts will discuss and answer questions that the audience may have. The session will be moderated by Robert Disque and will include Geoffrey Kulak on bolting and Omer Blodgett on welding. These speakers will be able to help with all the design, fabrication and erection problems just like the Steel Interchange column.

Thursday at 1:30 p.m.
Friday at 10:00 a.m.

8

The Total Team: Let's Get it Together

Moderator: Jerry Milligan, Falcon Steel Co.
Speakers:
Bill Treharne, Broad, Vogt & Conant, Inc.
Eric Waterman, National Erectors Association
Bill LIndley, W & W Steel

Erectors need to advise fabricators and engineers of techniques that have and will help minimize the erector's costs. Fabricators need to press erectors for help with simplification and overall man-hour savings.

Discussion will include erector/fabricator pre-bid planning for efficient erection techniques and connection design, as well as alternate suggestions for the design engineer.

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

9

Research into Practice

Speakers:
W. Samuel Easterling, Virginia Tech
Ahmad M. Itani, California Dept. of Transportation

Much work has been done on semi-rigid beam-to-column connections, *Easterling* will discuss semi-rigid beam-to-girder connections. His presentation will discuss the research that is in progress and the design recommendations that will be the result of the research.

Seismic research will be the topic of *Itani's* presentation. Special Truss Moment Frames will be introduced and compared to the behavior of Special Moment Resisting Frames.

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

10

TQM Personnel Issues

Moderator: Sid Blaauw, Paxton & Vierling Steel Co.
Speaker: Leo Peacock, Paxton & Vierling Steel Co.

Total Quality Management is a system that can work in steel fabrication shops. Mr. Peacock is the employee of a fabrication firm and is implementing a Total Quality Management system in that firm. He will give us the benefit of his experience in tailoring this successful management scheme to our business. Topics for discussion will include problem solving, employee empowerment, and partnering.

Thursday at 1:30 p.m.
Friday at 2:40 p.m.

11

International Developments

Moderator: Theodore Galambos, University of Minnesota
Speakers:
J. W. B. Stark, TNO - Bouw
Ben Kato, Takenaka Corp.

Fabricators and designers in 1994 will be very interested in the international marketplace. Overseas development will be the topic of this session. *Stark* will discuss the Eurocode requirements for steel and composite structures. *Kato* will present the new Japanese standard in limit-states design.

Thursday at 1:30 p.m.
Friday at 10:00 a.m.

TECHNICAL SESSIONS

12

Safety = Dollars

Moderator: Terry Peshia, Garbe Iron Works, Inc.

Speakers:

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

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

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

13

Building Retrofit

Speakers:

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

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

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

14

Marketing—Getting Your Share In Good Times and Bad

Moderator: Phil Stupp, Stupp Bros. Inc.

Speakers:

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

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

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

15

Experience from Wind Damage & Design Load Requirements

Speakers:

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

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

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

16

Software Requirements

Speakers:

Souhail Elhouar, Virginia Tech
Kevin Parfitt, Penn State

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

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

OPTIONAL EVENTS/SPOUSES' PROGRAM

All tours will use a modern, fully equipped passenger bus and include licensed tour guides. If your tour requires an admission fee, this is included in the price of the tour. Note: A tour may be canceled if AISC does not receive a sufficient number of registrations by May 5. In this case, you will be notified and a full refund will be issued after the Conference.

Those registering for the COMPLETE Spouses' Program will receive a ticket to ONE tour per day. Anyone wishing to register for any additional tours may do so by selecting the events on the Conference Registration form. There will be no charge for fully registered spouses attending the AISC Welcome Reception Wednesday evening or visiting the Exhibit Hall.

1 OPTIONAL EVENT

Thursday, May 18:
7:00 p.m. – 11:30 p.m.
Conference Dinner

This year's optional dinner will be a double-barreled event from the deck of the Party Liner, one of the beautiful Gateway Clipper Fleet.

While you enjoy musical entertainment and feast upon a delicious buffet dinner, you'll view the many bridges and tunnels and other engineering feats that link Pittsburgh with its suburbs beyond the surrounding hills. The three-hour cruise will provide a unique view of this dynamic city of three rivers plus a memorable dining experience.

Transportation to and from the boat from the hotels will be provided.

Price: $39 each

A SPOUSES' PROGRAM

Thursday, May 18:
9:00 A.M.-4:00 p.m.
Pittsburgh's Treasures Tour

Discover the excitement of America's First Renaissance City and largest inland port as your knowledgeable Tourguide and private coach take your guests through the very heart of this dynamic city, rated as the nation's most livable one!

This tour begins with an unparalleled view of the Pittsburgh skyline from atop Mount Washington. Descending by way of the DUQUESNE INCLINE, a veritable museum on wheels, adds a thrilling dimension to any tour!

Within the GOLDEN TRIANGLE are shimmering new skyscrapers, charming plazas and a new...the world's shortest...subway system.

Then it's on to CLAYTON, Henry Clay Frick's Pittsburgh residence. As a Victorian time capsule displaying affluent family life, Clayton has few peers. It represents an expression of Frick's desire to build an environment for his family and friends.

Despite the fact that Frick moved his family's residence to New York City in 1905 Clayton has been maintained intact. Thus, it is one of the most complete, best preserved and meticulously documented examples of late Victorian houses anywhere. Nearby is THE FRICK ART MUSEUM which was built in 1970 in the spirit of the early Italian Renaissance.

Continuing to Oakland, the cultural, educational and medical center of the city, a stop is made at a wonderful Italian restaurant where you will be treated to a delicious meal before moving on to the world-renowned NATIONALITY ROOMS in the Cathedral of Learning of the University of Pittsburgh. Each room, a gift from one of Pittsburgh's ethnic societies, exemplifies the old-world culture of that particular nation.

A short campus walk away stands the lovely French Gothic HEINZ MEMORIAL CHAPEL. Its stained glass windows, believed to be the world's tallest, depict stories about the famous persons represented in them.

Price: $45

SPOUSES' PROGRAM (CONT.)

B

Thursday, May 19
9:00 a.m.-5:00 p.m.
Amish Country

The picturesque college town of New Wilmington, Pennsylvania, is surrounded by farmland where the "plain people" live. Explore this serenely beautiful area and learn about the Amish way of life...their religion and culture.

An experienced tourguide directs this tour to the northwest portion of Pennsylvania, nearly bordering Ohio, where horses and buggies appear from the long lanes which lead back to the simple farmhouses and immaculately kept fields. Enroute over a warm spiced apple drink and honey nut rolls, your'll learn of the House Amish or Old Amish Order Mennonite Church and the fascinating stories of its followers. The customs, lifestyles, and "ferhoodled" English of this sect separate them colorfully from today's fast-paced society.

Lunch in New Wilmington at THE TAVERN ON THE SQUARE...known throughout the Tri-State area for its impressive decor, home-cooked food and famous honey rolls. Luncheon is served family-style.

This visit among the Amish includes shopping at an Amish home where quilts made by the Amish from as far away as Wisconsin are displayed. In nearby Volant, a 19th century mill now serves as a country store containing toys, gifts, Amish quilts and furniture sharing space with old mill machinery. Five miles south, the holidays come early at the Country House Christmas Shop in a restored Victorian home.

Price: $48

C

Friday, May 20
8:30 a.m.-4:30 p.m.
Fallingwater Luncheon Tour

Drive through the magnificent mountain scenery of the Laurel Highlands by privately chartered coach to FALLINGWATER, famed architect Frank Lloyd Wright's most widely acclaimed work. This masterpiece was built as a summer home for the Edgar Kaufmann family, prominent Pittsburgh merchants.

Dramatic cantilevered terraces soar over a cascading waterfall. The living room hearth embodies an immense boulder in the setting nature created. Completed in 1939 with guest wing and servants quarters, it is as fresh today as when it was built.

This house was judged by the American Institute of Architects in 1986 to be the nation's most successful example of architectural design. It is described as the clearest expression of Wright's ideal...that man can live in harmony with nature.

A stop at the Visitors Center and Gift Shop is planned before entering the home. After the tour, your guests are welcomed back to the coach with a glass of chilled white wine or iced tea before proceeding to lunch.

THE MAIN LODGE at SEVEN SPRINGS, a beautiful Mountain Resort in nearby Champion, PA, provides a delightful prearranged luncheon before continuing on to the WEST OVERTON MUSEUMS. The museums reflect life in a 19th century rural-industrial village and feature "Old Overholt" distillery/grist mill, Abraham Overholt Homestead and springhouse—birthplace of Henry Clay Frick.

Enroute, there is a short stop at the LENOX SHOP, noted for its discount prices on crystal, porcelain and silver...where Nancy Reagan's White House china may be viewed by those not busily making their own purchases.

Price: $44

SPOUSES' PROGRAM (CONT.)

D

Friday, May 20
9:30 a.m. - 1:30 p.m.

A Shopper's Dream

The coach whisks guests from their hotel to STATION SQUARE, the lively riverfront restoration of the former P.&L.E. Railroad, now a complex of exciting shops, boutiques, historic memorabilia and fine restaurants. Guests are invited to enjoy the delightful shops for the remainder of the morning.

Your Shopping excursion begins and ends in these restored railroad buildings. This historic site houses over 65 specialty shops and restaurants, featuring goods and cuisine form around the world.

A delightful luncheon is scheduled in the Edwardian spendor of THE GRANT CONCOURSE, P.&L.E.'s turn-of-the-century terminal.

Following this pleasant respite, an afternoon of shopping awaits! Beginning with Downtown's One Oxford Centre, a collection of upscale shops, and continuing on to Kaufmanns, one of Pittsburgh's major department stores, where your guide will depart, after providing guests with return directions to the hotel.

Price: $32

E

Wednesday, May 17:
6:30 p.m. - 8:00 p.m.

AISC Welcome Reception

Hors d'oeuvres and cocktails in the Exhibit Hall.

Price: $20 (included in spouse & full registration fee)

F

Wednesday - Friday: Open Exhibit Hours Exhibit Floor pass

Price: $5 a day (included in spouse & full registration fee)

NSCC COSPONSORS:

American Iron and Steel Institute

American Welding Institute

American Welding Society

Canadian Institute of Steel Construction

Mexican Institute of Steel Construction

National Erectors Association

National Institute of Steel Detailing

Steel Deck Institute

Steel Joist Institute

Steel Plate Fabricators Association

Steel Service Center Institute

Steel Structures Painting Council.

NATIONAL STEEL CONSTRUCTION CONFERENCE

THE ONLY "ALL-STEEL" CONFERENCE AND TRADE SHOW IN THE U.S.

MAY 18-20, 1994

DAVID L. LAWRENCE CONVENTION CENTER
PITTSBURGH, PENNSYLVANIA

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

HOTEL REGISTRATION

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

HOUSING CENTER:

The Greater Pittsburgh Convention & Visitors Bureau Housing Department will coordinate NSCC 1994 hotel reservations. The Greater Pittsburgh Convention & Visitors Bureau will coordinate NSCC 1994 hotel reservations. Please use this form to request accommodations; the housing bureau will not accept reservations over the phone. You may fax this form to:

Fax 412-644-5512

RESERVATIONS:

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

CUT-OFF DATE:

The cutoff date for hotel reservations is April 10, 1994.

CHANGES/ CANCELLATIONS:

All changes and cancellations should be made directly with the Pittsburgh Housing Bureau. Your room confirmation will arrive directly from the Bureau. The housing bureau will inform you by mail or by fax of your hotel assignment. A confirmation from your hotel willl follow in four weeks.

CONFERENCE HOTEL:

Pittsburgh Vista—$104 single/$115 double per night
Westin William Penn—$112 single/double per night

The Pittsburgh Vista is the official Conference Hotel. Located adjacent to the Convention Center, it serves as the primary hotel for sleeping accommodations. All tours and optional events depart and return to the Vista Hotel. The Westin William Penn is a five minute walk from the Convention Center. Suites are available upon request at the Vista and Westin.

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

CREDIT CARD INFORMATION

CREDIT CARD COMPANY

NUMBER

EXPIRATION DATE

CARD OWNER'S SIGNATURE

SEND CONFIRMATION TO:

NAME

COMPANY

ADDRESS

CITY

STATE/ZIP

OFFICE PHONE

HOME PHONE

FAX

PHOTOCOPY AND MAIL ENTIRE COMPLETED FORM TO:

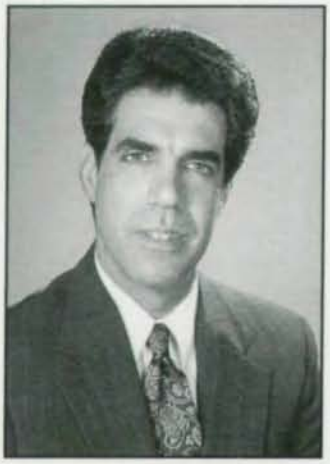
NSCC 94 Housing
American Institute
of Steel Construction
c/o Pittsburgh Housing Bureau
4 Gateway Center
Pittsburgh, PA 15222
Fax 412-644-5512

HOUSING BUREAU DEADLINE DATE:

APRIL 10, 1994

Long-Span Roof Structures

Structural steel allows the creation of cost effective, aesthetically pleasing arena roofs



The article was adapted from a paper prepared by ***Daniel A. Cuoco****, P.E., a principal with Thornton-Tomasetti Engineers in New York City, for the 1994 National Steel Construction Conference. Also contributing were Udom Hungspruke, a vice president with Thornton-Tomasetti, and Robert P. DeScenza, P.E., a vice president with TT-CBM Engineers in Chicago. The original paper included both the Anaheim Arena and the New Chicago Stadium. For information on the Anaheim Arena, see the February issue of* MSC.

Unlike the large combination sports and convention facilities built during the 1960s and 1970s, today's facilities are geared towards sports and entertainment events. Enclosed arenas, to be used for basketball and hockey games as well as other events, have recently been completed or are currently under construction in several cities, with similar projects being planned for other cities.

Due to the tight schedule and budget constraints typically associated with publicly funded arena projects, the architectural designs—though dramatic—are usually driven by structural engineering considerations. By using structural steel, designers and constructors have the opportunity to create innovative long-span roof structures that are aesthetically pleasing as well as cost effective.

A good example of this type of design is the New Chicago Stadium, which will be the new home of the National Basketball Association Chicago Bulls team and the National Hockey League Blackhawks team. It is scheduled for completion next month and will replace the adjacent 65-year-old Chicago Stadium, which will be razed to provide room for additional parking. In addition to professional sports, the new arena will be used for collegiate sports and special entertainment events. Unobstructed seating for 21,500 spectators, plus 216 luxury suites, will be provided. Architect on the project was Hellmuth, Obata & Kassabaum, Sports Facilities Group, Kansas City.

The New Chicago Stadium will be the home of the Bulls and the Blackhawks.

The foundation system consists of caissons bearing on hardpan material. The structural frame of the seating bowl is constructed of cast-in-place concrete and contains four expansion joints. Precast prestressed concrete seating elements span 16-to-36 ft. between typical concrete frames.

The roof structure spans 378 ft.-by-493 ft. It is supported by 36 cast-in-place concrete columns, ranging from 3-to-4 ft. in diameter, along the perimeter of the building. The center of the roof rises 35 ft. above the spring points at the building perimeter.

Two-Way Tied Arch System

The two-way multi-post tied arch roof system used on the project is an innovative and efficient structural system that was developed to meet aesthetic requirements and to facilitate fabrication and erection while minimizing the required quantity of steel.

The main framing elements of the roof system consist of six intersecting tied arch trusses, i.e., four queen-post tied arch trusses spanning in the short direction and two multi-post tied arch trusses spanning in the long direction. Vertical compression struts are located at the eight intersection points of the tied arch trusses. Tension tie members traverse the arena, tieing the ends of the arch trusses and connecting to the bottom of the compression struts. Thus, although the structure appears light and elegant, the effective structural depth of the system is 60 ft. The prefabricated arched top truss segments are typically 12.5-ft. deep center-to-center. Trusses of equal depth infill the central roof area and span between the tied arch trusses and the support columns along the perimeter of the roof. The out-to-out dimension of all trusses was limited to 14 ft. in order to allow for shipping of shop-assembled segments up to approximately 60 ft. long.

For arena projects,, the two-way multi-post tied arch roof system offers a number of advantages.

1. **Tied arches create** minimum outward thrust, thereby eliminating the need for a perimeter ring to resist tension (as would be required for untied arches) or compression (as would be required for air-supported and tensile structures). The roughly rectangular footprint that is ideal for arena seating layouts is not suitable for a round or oval tension or compression ring around the roof edge.

2. **Two-way spanning action minimizes** the visual bulk and enclosed volume of the building, and can easily accommodate special corner and edge conditions. In contrast, one-way spanning systems are best suited for rectangular roof plates of uniform elevation.

3. **Trussed arch members offer** both visual lightness for architectural effect and physical lightness for shipping and handling.

4. **The low arch profiles and inclined tension ties minimize** roof height, visual bulk, and enclosed volumes without sacrificing overall structural depth or obstructing sight lines.

5. **Large, efficient structural depths are** created by combining shallow, easily shipped upper trusses with simple tension ties and compression struts.

6. **Erection is** straightforward and efficient by erecting primary trusses first and then dropping in secondary framing.

Roof System Support

The design of the New Chicago Stadium is similar to that of Anaheim Stadium, which

opened last year (see "Spanning Hockey's Newest Pond," February 1994 *MSC*). However, since seismic forces did not control in the design of the New Chicago Stadium as they did in Anaheim, the roof trusses were able to be supported directly on cast-in-place concrete columns. Likewise, there was no need to perform a coupled analysis of the roof structure and concrete seating bowl. Reactions obtained from the roof structural analysis were utilized as applied loads in a separate analysis of the concrete support structure.

In order to minimize lateral displacements of the support points under gravity loads, these support points are located at the neutral axis of the trusses.

Due to large anticipated thermal ranges during construction, four expansion joints were introduced in the concrete seating bowl. Therefore, isolation of the roof structure (which has no expansion joints) from the seating bowl was required in order to avoid large forces that would otherwise develop at the truss supports due to thermal loadings. This isolation was accomplished by providing slide bearings at all 36 truss support points. In order to provide stability for wind loads, however, two supports in each principal direction are rigidly attached to the concrete support structure in order to transmit wind forces from the roof to the concrete seating bowl. All other supports allow the truss ends to slide relative to the concrete seating bowl, thereby preventing the build-up of thermal forces.

Generally, the roof beam design was controlled by gravity loadings. Wind loadings, however, did result in net uplift at the roof truss supports. The lateral loads are transferred through the horizontal top chord bracing trusses that span to the rigid support points at each edge of the arena.

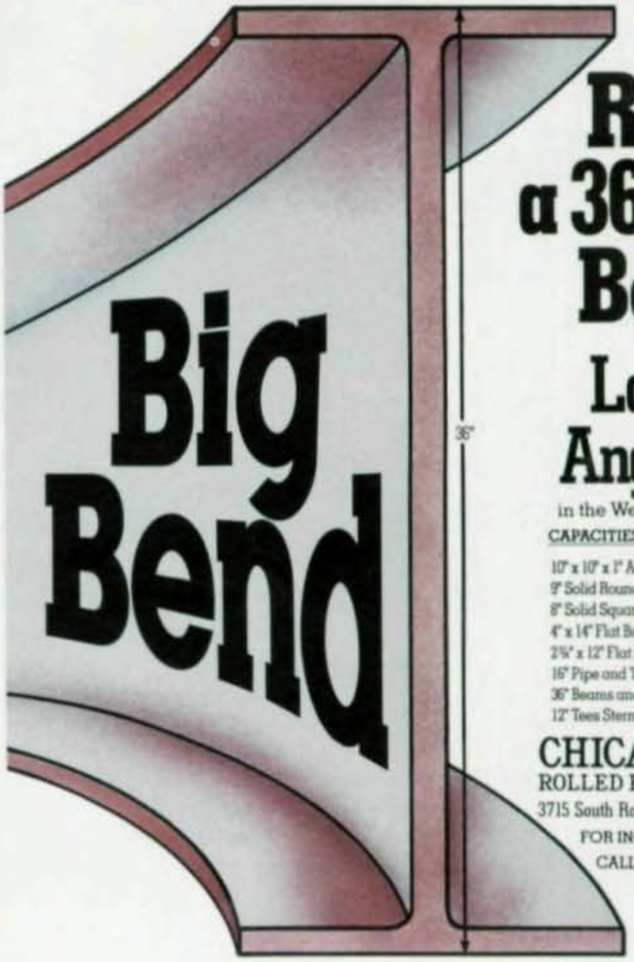
Erection Considerations

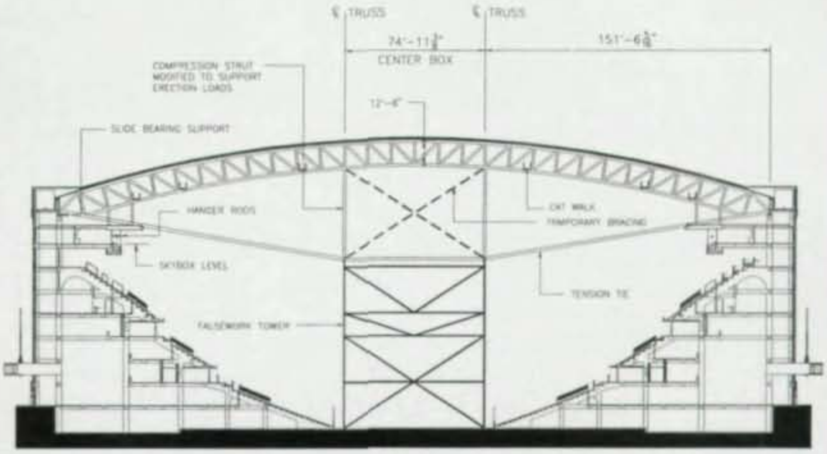
The success of any long-span structure is dependent upon its erection method. Suggested erection procedures, sequences and constraints were incorporated into the original structural design drawings.

A key design feature that facilitated erection was the location of the central truss box within the arena floor footprint. This allowed the entire roof structure to be erected without adversely affecting the construction of the concrete seating bowl.

Upon award of the steel contract to AISC-member Owen Steel, intense discussions took place among the design and construction teams in order to develop the fastest, most efficient and safest erection procedure for the roof structure that was compatible with the overall construction sequence for the project. The final erection procedure, designed by AISC-member American Bridge, the project's steel erector, was reviewed and approved by Thornton-Tomasetti Engineers.

The erection scheme used two, 100-ft.-tall, four-legged shoring towers, one at each end of the 75-ft.-by-188-ft. central truss box area. Each tower leg was located directly below a compression strut. At each tower location, the four compression struts were temporarily braced together and





were directly supported on the tower legs. Thus, the braced compression struts effectively became an extension of the shoring towers, thereby minimizing the falsework and simplifying the subsequent dismantling of the towers.

The major steps in the erection procedure were as follows:

1. **Erection of the two shoring towers** within the arena floor footprint.

2. **Erection of the compression struts**, with temporary bracing, and the truss top chords across the shoring towers.

3. **Segmental erection of the 151.5-ft.-long roof trusses** between the central truss box and the perimeter. This was performed simultaneously at both ends of the central truss box.

4. **Erection and bolting of the tension ties** between the ends of the tied arch trusses and the bottom of the compression struts.

5. **Placement and welding of the roof deck.**

6. **Incremental lowering of the shoring towers**, maintaining a maximum differential of one-half inch between towers, until the lift-off position was reached.

Six months after the start of erection, the shoring towers were lowered the expected 3.5 in. to the lift-off position.

Balancing Structural Strength And Ductility Requirements

Taking into account the effects of drift limits and the use of higher strength steel could allow designers to balance the required ductility with the available overstrength

*This article is based on a paper presented at the 1994 National Steel Construction Conference by **Chia-Ming Uang**, Ph.D., an assistant professor in the Department of Applied Mechanics and Engineering Sciences at the University of California at San Diego. A more complete version of this paper is presented in the conference* Proceedings.

Modern seismic design practice in the U.S. requires that a structure be designed for a prescribed design seismic force level that is significantly lower than that required if the structure were to respond elastically. The dual premise behind this design philosophy is that the structure has a certain amount of ductility capacity to dissipate energy and the structure's actual strength is significantly higher than that required by seismic codes.

For moment frames of low- to medium-rise construction, drift limitations as specified in seismic codes generally will not dictate the member design of reinforced concrete structures. Depending on the ductility capacity required, engineers may design a reinforced concrete frame as a Special, Intermediate or Ordinary Moment Frame. For steel construction, however, seismic design provisions provide only two classifications of moment frame design: Special and Ordinary Moment Frames (SMF or OMF). Furthermore, since steel moment frames are more flexible than reinforced concrete frames, member sizes of a steel frame tend to be dictated by drift limits.

Under such circumstances, a steel structure's actual strength may be significantly higher than that required by seismic codes; therefore, the ductility demand can be much lower than a similar structure (e.g. reinforced concrete frame) whose design is governed by strength.

Unfortunately, this unique feature of steel design is not recognized by the current seismic provisions, and the current steel design procedure is very conservative. This conservatism in steel moment frame design exists not only in high seismic regions, but also in regions of low to moderate seismicity. More efficient designs would result from the creation of an Intermediate Moment Frame (IMF) classification for steel design.

Current Design Procedures

Building code regulations in the U.S. specify design seismic forces by reducing the elastic force demand by a force reduction factor (R for LRFD or R_w for ASD). Based on the observed performance of similar structures in past earthquakes, empirical R or R_w values have been assigned to different lateral load resisting systems, with design forces for SMFs being significantly lower than those of OMFs. However, SMFs have to satisfy stringent ductility requirements. Although the R values are empirical in nature, it has been shown that this factor is primarily composed of the structural reduction factor and the structural overstrength factor. Structural overstrength

The first table shows a comparison of R*-Factor seismic design approaches, while the second table shows* R *and* R_w *values for steel moment frames.*

Seismic Provisions	R =	R_μ ×	Ω
NEHRP (U.S.A.)	specified	(not available)	
NBC (Canada)		specified	specified (constant)
ECCS (Eurocode No. 8)		specified	to compute (variable)
BSL (Japan)		specified	to compute (variable)

Building System	R (NEHRP)	R_w (UBC)	
		1985*	1988
Special Moment Frame (SMF)	8	12	12
Ordinary Moment Frame (OMF)	4½	8	6

results from structural redundancy, member oversize due to, among others, drift limits, non-seismic load effects, etc., and this factor accounts for the difference between the structure's ultimate strength and the code-prescribed design strength.

While the physical meaning of the R factor is clear, the procedure by which the R factor is implemented in the design process varies considerably from one code to another. The approach used in the U.S. does not incorporate the structural overstrength factor explicitly, while both the Eurocode and Japanese codes allow designers to take advantage of the structure's true strength in computing R and the Canadian code specifies a constant overstrength factor of 1.67. The only advantage of the U.S. approach is simplicity.

Drift Limit Considerations

Historically, the seismic design of steel moment frames in the U.S. has been to follow the seismic provisions similar to those of the 1985 UBC, which allows an R_w of 12 for SMFs and an R_w of 8 for OMFs.

Although seismic codes encourage the use of SMFs because of their excellent energy dissipation capacity, designers quickly learned that the incentive of allowing for smaller design seismic forces often disappears because drift limits as specified in seismic codes dictate the member sizes. Since cost savings cannot be achieved, some designers tend to use OMFs in order to avoid stringent (and expensive) ductility requirements. (Note that drift limit is less problematic for reinforced concrete frames.)

Frustrated by the fact that designers using the 1985 UBC were discouraged from using SMFs, starting in the 1988 edition, UBC began "penalizing" OMFs by reducing the R_w factor from 8 to 6. In other words, the design seismic force level for OMFs is twice that of SMFs. NEHRP R factors are similar.

Unfortunately, penalizing OMFs is not a rational solution; nor does this approach make the best use of steel. Instead, a new design procedure that addresses the unique features of steel moment frame design is needed.

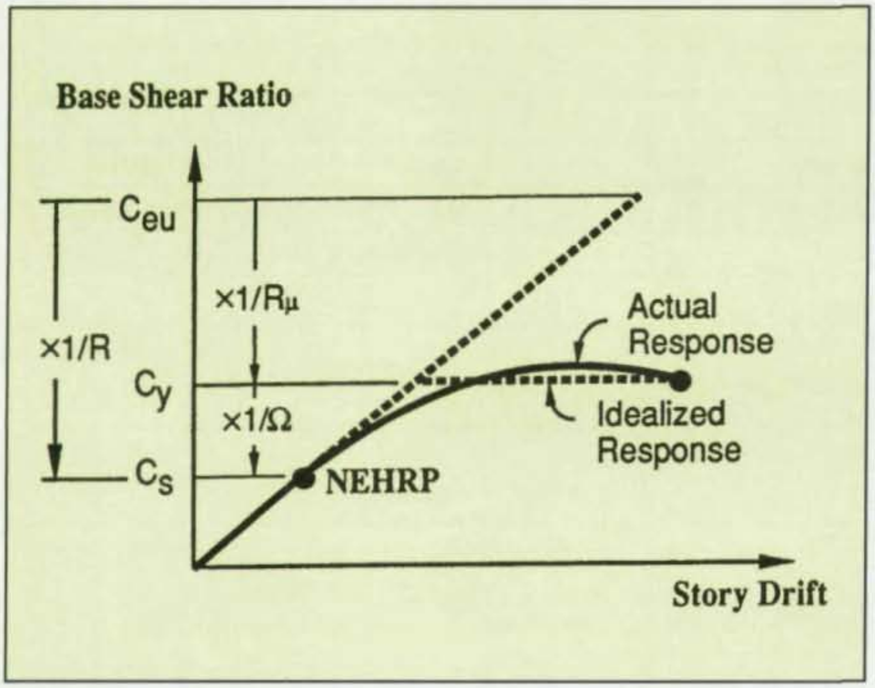
High-Strength Steel Dilemma

Significant improvements in steel making technology have been achieved in recent years and as a result the price of A572 Grade 50 steel is about the same as A36 steel.

Design engineers have responded positively to this development in non-seismic applications with the result of more economical designs. However, seismic design provisions require that plastic (i.e. Class 1) compact sections be used to ensure adequate member rotation capacity.

When A36 steel is used, this stringent requirement of using a plastic section does not impose difficulty to design engineers because most of the economic compact (or class 2) sections in the AISC Manuals also qualify as a plastic section. However, this is not true for 50 ksi steel. As a result, design engineers in high seismic regions are reluctant to use higher strength steel even though the material itself does not cost more nowadays than A36. Bridge designers have overcome this dilemma through the introduction of the effective plastic moment concept, which permits the use of Class 2 sections in plastic design at a reduced moment level.

One critical aspect that is not addressed in this issue, though, is the benefits of structural overstrength in steel frames. On one hand, it is indeed true that a steel member's rotation capacity is reduced as a result of using higher strength steel. On the other hand, seismic design provisions in the U.S. fail to recognize that, when drift controls the design, a steel structure's ductility demand is also reduced as a result of using higher strength steel. Rejecting the use of higher strength steel purely from the viewpoint of rotation capacity is misleading; a proper way to



The diagram above shows the physical meaning of the response modificationfactor, R. *The tables opposite show the classification of ductility requirements and the rotation capacity and limiting width-thickness ratio for flange local buckling.*

address the impact of using higher strength steel in seismic design is to also examine the rotational ductility demand.

Proposed Procedure

Based on the previous discussions, it is clear that what is needed is an alternate design procedure that is rational and flexible, yet is still simple for practical design.

The proposed method has two important ingredients. First, the SMF and OMF as defined in NEHRP and UBC should be treated as two distinct ductility classes, with each class associated with a set of ductility requirements (e.g., limits of slenderness ratios for local and lateral-torsional buckling limit states) specified in the current seismic provisions. Second, the ductility class needed in a specific design project should be based on the ductility reduction factor, not the R factor. The concept of tying the

Ductility Class	Special Moment Frame (SMF)	Intermediate Moment Frame (IMF)	Ordinary Moment Frame (OMF)
R_μ	≤ 6	≤ 4	≤ 2

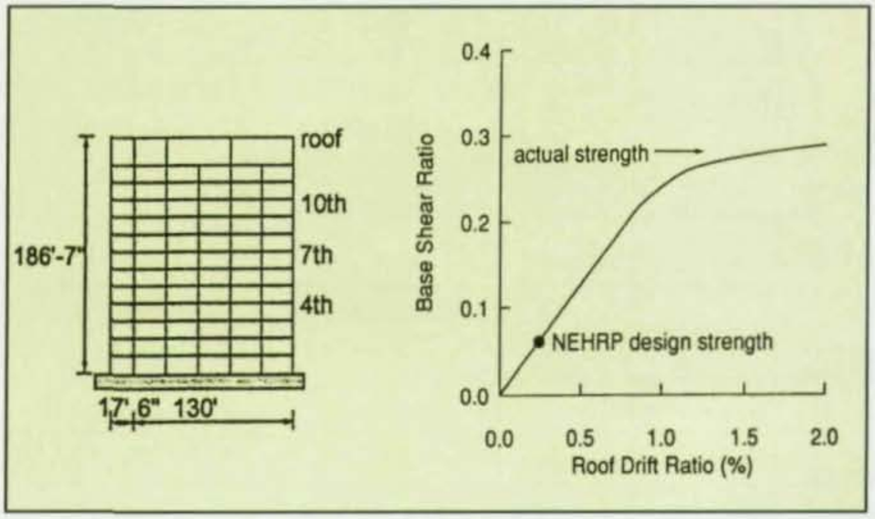
Ductility Class	Special Moment Frame (SMF)	Intermediate Moment Frame (IMF)	Ordinary Moment Frame (OMF)
rotation capacity	10	7	3
limiting $b_f/2t_f$ ratio	$52/\sqrt{F_y}$	$60/\sqrt{F_y}$	$70/\sqrt{F_y}$

ductility class (or requirements) to the ductility reduction factor rather than the *R* factor can be demonstrated by the following example.

A 13-story office building located in San Jose, CA, has been instrumented by CDMG and its seismic performance extensively studied. It is an SMF designed in 1972; the design was governed by drift limitations. Because member sizes are dictated by drift limitations, the structures ultimate strength is about five times that required by NEHRP (or eight times that required by UBC). That is, the overstrength factor is equal to 5. Stringent ductility requirements like limiting the beam flange's width-to-thickness ratio to provide the required rotational ductility capacity may be too conservative. Instead, OMFs (or even IMFs, if developed) may be sufficient. Note that the approach of using more than two ductility classes is already adopted in the Eurocode, Japanese and New Zealand seismic codes. Also, specifying three classes of moment frames in reinforced concrete design has already been used in UBC and NEHRP seismic provisions.

The required rotation capacity of flexural members is usually achieved in design by satisfying limiting slenderness ratios for flange local buckling, web local buckling and lateral-torsional buckling. An examination of the existing data on the use of limiting slenderness values to control local buckling shows that limiting the width-thickness ratios for proposed three ductility classes can be used for seismic design (SMF, IMF and OMF).

Actual strength of the structure plays a vital role for the survival of structures in severe earthquakes and the importance of the reserve strength beyond the code-specified level has been recognized recently in several foreign codes. A structure's actual strength also has to be computed in order to take advantage of the reserve strength resulting from drift requirements and the use of higher strength steel. Furthermore, quantifying the actual structural strength will discourage the use of structures with low redundancy—one- or two-bay perimter steel moment-resisting frames is one typical example. Although this seems to be a major obstacle for practical design, modern computer technology can meet the challenge. Many computer programs for



Structural overstrength of a 13-story steel moment frame.

nonlinear structural analysis are already available (for example, DRAIN-2DX, developed at the University of California at Berkeley, has been used extensively by researchers). To perform a nonlinear static analysis, only minimal additional input data is required compared with traditional elastic analysis. In addition to nonlinear static analysis programs, in this research project a PC-based plastic analysis computer program, MECHAPD, which is based on the upper bound (or mechanism) theory, has been developed for multistory frame analysis. The reduction of the plastic moment in columns due to the existence of axial force is included in the analysis, and the P-delta effect is considered by the multistory buckling concept (i.e., the B2-factor approach in the LRFD Specification). To facilitate data input, the AISC Database has been incorporated into the program so the user can specify rolled shapes directly.

Conclusions

A new design concept that recognizes the unique feature (i.e., the issue of drift limits) for seismic design of steel moment frames has been proposed. Specifically, the concept proposes that the reserve of structural strength resulting from the lateral stiffness requirement is considered in the design process. The ductility requirement is then balanced with the actual strength of the structure. The proposed design concept also is ideal for dealing with the problems associated with the use of higher strength steel for seismic design because the balance between strength and ductility requirements is considered directly.

Under the proposal, the classification of steel moment frame in current seismic provisions is extended to three ductility classes (Special, Intermediate and Ordinary Moment Frames). Design engineers can take advantage of and quantify the available structural overstrength resulting from drift limits and the use of higher strength steel; the required ductility class is then determined and balanced for a given R factor. Therefore, current practice, which bases the R or R_w factor to determine the level of ductility requirement, is abandoned. For each ductility class, the required rotation capacity at the member level is required. Efficient computer programs are currently available to compute a steel frame's actual strength. A computer program based on the plastic mechanism theory also has been developed for analyzing multistory frames.

The methodology developed in this research project also can be easily extended to braced (concentric and eccentric) frames.

The Impact Of Hurricane Andrew On Wind Engineering

Attention has focused during the past decade on protecting the building envelope

*The article was adapted from a paper prepared by **Robert J. Wills, Jr., P.E.**, the Southeast regional director of construction codes and standards with AISI, for the 1994 National Steel Construction Conference.*

Hurricane Andrew ravaged south Florida on August 24, 1991 leaving behind a total property loss estimated in excess of 20 billion dollars. While most of the physical damage related to the storm occurred in a few hours, the resulting effects on building codes and standards have been occurring steadily ever since. These developments will continue for the foreseeable future.

In the days immediately following the storm, stories circulated about Andrew being a "monster storm" with winds well in excess of 200 MPH. Actual wind speeds were difficult to ascertain since many of the anemometers in the area were damaged or destroyed. Most wind speed estimates developed since the storm have been based on analysis of structural and nonstructural damage. These estimates vary considerably and are still a source of heated debate. These discrepancies are sometimes magnified since there are several different measuring systems used to define wind speed. Some researchers express wind in terms of peak gust (3 to 5 second average), others use sustained wind (1 minute averaging period), while building code numbers are typically based on fastest mile.

While confusion over these "official" numbers continues, one basic question must be answered: Was Hurricane Andrew a superstorm with winds so far in excess of that anticipated by the building code as to make code review irrelevant?

The best available data suggests that it was not. Andrew was a very tight storm. Wind speeds at the center of the eyewall may have slightly exceeded the requirement of the building code, but as you move away from the center the wind speed quickly decreased. It is interesting to compare Andrew with Hurricane Hugo, which struck the South Carolina coast in 1989. Andrew had slightly higher wind speeds at the center, however, as you move away from the eye, Hugo's winds soon exceeded Andrew and extended for a much wider area. Even when using the most severe estimates of Andrew's wind profile, the area where design wind speed was exceeded should have been very limited. The extensive damage of Hurricane Andrew cannot be excused by it's intensity.

Andrew caused significant damage to buildings in an area of over 400 square miles. If intensity alone does not explain this result, what other factors are responsible? The region impacted by Andrew represents a large area that had been developed relatively quickly. Building department resources had been stretched with an inevitable drop in the quality of code enforcement. Poor construction practices were graphically exposed by the storm. In aerial observation of the damage, examples could be cited where all of the houses built in a particular subdivision by one contractor would show identical failure modes. Adjoining subdivisions built by a different contractor would exhibit a different "signature" type of failure. Ground investigations confirmed numerous construction lapses. Despite the evidence of construction and enforcement inadequacies, the "blame" for Andrew cannot be isolated to these factors. It is also a mistake to assume that these same problems would not arise in other parts of the country.

While there were areas of low and mid-rise commercial buildings affected by Hurricane Andrew, the majority of destruction was focused on residential housing. These dwellings were typically built by prescriptive building code requirements and are subject to the limitations of those requirements. Despite the advantages of prescriptive code requirements, the typical single family dwelling must be considered a non-engineered structure.

Hurricane Andrew coincidentally impacted the only two counties in Florida with a building code that is not based on the Standard Building Code. The South Florida Building Code (developed and used by Dade and Broward Counties) was open to intense scrutiny after the storm. Historically the SFBC has claimed to be one of the strongest codes for wind, however, in the aftermath of Andrew that claim was hard to justify. In fairness, the SFBC was at the forefront in developing prescriptive code requirements for residential construction. Unfortunately, the code had not been updated to reflect some of the recent research in wind engineering. In particular, the code did not adequately address component and cladding design. One of the first actions taken by Dade County after Andrew was to move to wind design based on ASCE 7-88.

Wind Forces: A Simplistic Review

Before we attempt to examine what went wrong in Andrew, it may be helpful to review a few basic principles that explain how wind interacts with buildings and structures. When a building becomes an obstruction in the path of the wind, either the wind must change to accommodate the building or the building changes to accommodate the wind. Since we usually try to avoid the second option, we must consider the forces imparted to the building that result from changes in wind velocity and direction.

As wind encounters the "windward" wall of a building, the forward velocity is slowed or stopped resulting in an inward pressure on the surface. However, this may be the only surface of the building that experiences a net inward pressure. The building forces the air flow to travel a longer distance as it goes around or over the structure. Wind traveling this longer distance must accelerate resulting in a lower pressure at the building surface and an outward acting pressure differential across the roof and side walls. This phenomena, known as the Bernoulli effect, is commonly used to explain the lift mechanism in airplane wings.

Since we do not make buildings as aerodynamic as airplane wings, a second phenomena is introduced. The air flow cannot negotiate sharp corners of buildings, such as wall and roof corners, ridge lines and eaves. The air flow separates from the building surface at these discontinuities and reattaches to the surface some distance downstream. The area between the separation and the reattachment point is often defined as the wake region, and it is characterized by very turbulent air patterns. In this wake zone you find fluctuating low pressure regions that can be very severe in localized areas. The engineering that defines the building loads for these areas is based primarily on evidence from wind tunnel modeling, not fluid mechanics equations. The most noticeable deficiency in the South Florida Building Code prior to Andrew was it's treatment of this localized phenomena. Unfortunately, localized failures in these areas can lead to progressive failures of other building components.

As noted in the previous discussion, the wind can cause outward pressure on all surfaces of the building except the windward wall, with the most severe pressures occurring locally at corners and ridges. There is one other condition that must be recognized to explain the interaction of the building with the wind. All the previous comments are based on the building being completely enclosed with a given ambient internal pressure. Indeed, most buildings are assumed to act in this fashion. However, openings in the exterior building envelope can reek havoc on this assumption. An opening in the windward wall will result in an increased internal pressure and a "ballooning" effect on the building. This internal pressure adds to the outward pressure on the leeward wall, the side walls, and the roof. Conversely, an opening in the leeward wall or a side wall can result in the air being drawn out of the building. This negative internal pressure can intensify the inward pressure on the windward wall. Since the wind direction typically changes during the course of a hurricane, openings in the building envelope can produce a variety of forces on the building.

What Went Wrong?

The damage of Hurricane Andrew cannot be attributed solely to the storm severity, poor construction practices, lax code enforcement, or building code deficiencies. In addition, Hurricane Andrew did not drastically change our basic understanding of how the wind interacts with a building.

Given these premises, the wind engineering community has pursued a variety of issues. Some of these issues are technically focused and related to how wind concepts are integrated in the building code and incorporated in the design process. Others issues have involved more philosophical concerns addressing our definition of what is considered to be acceptable building performance.

Technical Issues

In the aftermath of Hurricane Andrew, the mandate of the wind engineering community has been to "protect the building envelope". The vast majority of technical changes evolving in wind design are related to this concept. There are two reasons being cited for this increased level of concern.

Structural integrity—Most buildings are designed as enclosed structures. The function of the building envelope is essential to the building performing consistent with this design assumption. Current design documents imply that it only takes the loss of 5% of the windward side wall area to have the building change from completely enclosed to partially enclosed. The resulting internal pressure change is substantial, and in some instances can affect the stability of the structural system.

The "more than life safety" position—This concept is more philosophical than technical, but it hinges on the argument that building codes cannot limit their focus to life safety, but must also protect property, and to some extent, insure post-storm building function. There were relatively few people killed or injured in Andrew, but the 20 billion dollar insured loss cannot be considered acceptable. Much of this loss consisted of interior wind and water damage to contents in buildings that had apparently survived unscathed. The validity of this position has significant impact. Without this argument, the easy solution to the building envelope problem would be to require that all buildings must be designed as partially enclosed structures with full internal pressure.

As noted, the vast majority of building code changes that are being considered can be related back to the building envelope issue. Among these are:

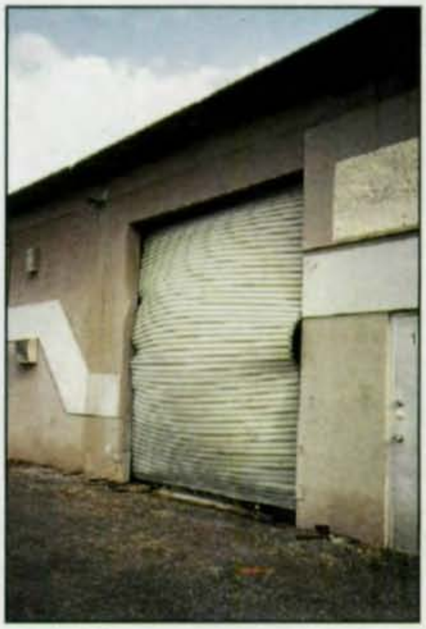
Door and window design—Most codes include wording to imply that the doors and windows must be designed for appropriate wind pressures. In reality, this intent of the code has not been strictly enforced. New wording has been included in most codes to ensure that the doors and windows will stay in place for buildings when designed as enclosed structures. In some instances, the codes also will require testing to verify performance and listing of door and window products.

Many of the failures in commercial low rise buildings during Andrew can be directly related to internal pressurization from improperly designed doors and windows. In low-rise metal buildings this pattern was epidemic. The loss of large access doors led to unanticipated internal loads which stripped the buildings of the roofing and siding. In extreme cases, the collapse of the structural frame can be traced to loss of lateral support from the cladding elements.

The same problem plagued other construction types as well. Tilt-up concrete wall panels were often blown out by internal pressure from a failed windward door. Prestressed concrete double "T" roof panels were cracked or blown off by the same internal pressure problem. Unfortunately, the failures in metal buildings were more visible and catastrophic.

If the designer is evaluating the building as an enclosed structure, it is essential that the design responsibility includes a mechanism to insure that the doors and windows are evaluated and installed to resist the proper component and cladding based wind pressures.

Windborne Debris Impact—The one component of a hurricane that has not traditionally been considered by U.S building codes involves the windborne debris created by the storm and it's potential effect on the building envelope. Research on this issue has been developed over the last 20 years, but the only code application has been in Australia. Damage from windborne debris was cited as a major factor in Hurricane Andrew, prompting several organizations to investigate standards to address the problem.



A hurricane generates missiles with a wide variety of sizes and shapes. In the current code activities, the test standards are attempting to replicate the impact of two different missile types. The typical large missile is a 9 pound 2x4 impacting at a speed of 50 ft/sec. This is intended to reproduce a variety of items such as roof tiles, garbage cans, and tree limbs. The typical small missile is a 2 gram steel ball impacting at a speed of 130 ft/sec, which is cited as being representative of roof gravel.

The large missile test is usually applied to the lowest 30 ft of the building, while the small missile test is applicable for the full height of the building. The missiles are fired by a compressed air powered cannon, however, there is a task group within ASTM working on a comparable test using a pendulum.

After the element is struck by the missile, the test specimen is subjected to a cyclic pressure loading test to replicate the wind pattern of the hurricane. The initial proposed cyclic tests required the application of 18,000-20,000 pressure cycles, however, this number has been reduced to approximately 9000 cycles. Early cyclic tests included a number of "through-zero" cycles, however, recent models include few cycles that would generate a complete stress reversal.

Two task groups in ASTM E-6 are working on missile impact standards. One uses the air cannon and the other focuses on a pendulum to produce the impact of the large missile. The BOCA Committee on Loads was presented the basic concept for a windborne debris test standard and chose not to take action itself, referring the issue to ASTM and/or ASCE.

The metal building industry needs to develop a basic understanding of how typical steel siding and roofing materials will perform in these impact tests.

Interior Non-Loadbearing Walls—As exterior building envelopes failed in Andrew, the internal pressurization of the building put unanticipated loads on interior partitions. Most codes require these "on-loadbearin walls to be designed for a minimum lateral load of 5 psf. Observed failures have led some to call for this minimum loading to be raised to 10 psf or higher. Obviously, this is not necessary if the exterior envelope performs as intended.

Unfilled Deck Diaphragm Design—Recently, the Steel Deck Institute has had to send representatives to Dade County to address code restrictions that did not allow unfilled steel decks to be used for diaphragm design. In all likelihood, the code officials are reacting to post-storm observations where a steel deck lost it's ability to function as a diaphragm as a result of unanticipated internal pressurization. This type of problem can be more accurately attributed to door and window failures.

Steel Joist Connections—A few postmortem reports have mentioned problems with the anchoring of steel joists to the supporting wall construction. Typically, the failure was either at the weld between the joist and the plate, or the plate remained attached to the joist but pulled out of the wall. Discounting the possibility of poor design or construction errors, these occurrences are usually found in buildings where internal pressure may have been a contributing factor.

Wind Design Standards

One of the first actions taken in Dade County after Andrew was the adoption of ASCE-7 1988 for wind loads. ASCE 7 is also the base document for wind loads in the three major U.S. model codes. At this time, the new edition of ASCE 7 (1995) is in draft form, and in the process of being balloted by the main committee. While Hurricane Andrew has not overtly impacted this draft of ASCE-7, there are a number of significant changes that deserve mention:

Main Wind Force Resisting System Coefficients—The 1995 draft includes MWFRS coefficients derived from research conducted at the University of Western Ontario. These coefficients are similar to the values found in the National Building Code of Canada and in the MBMA manual except that the 0.8 reduction factor has not been incorporated in the ASCE version.

Wind Speed Maps—The wind maps in the 1995 draft differ considerably from that shown in the 1988 edition. Most significantly, the 1995 values are expressed as 3 second gust speeds, while the 1988 edition is based on fastest mile wind speed values. While the wind speeds in the 1995 draft will be numerically higher for most geographic locations the resulting wind pressures should not change appreciably.

The change in wind speed classification is necessary since the "3 second gust" database is becoming the standard in this country. Fastest mile wind speed recording is currently being phased out by the National Weather Service. This change should also have a positive psychological effect since the design wind speeds will now be closer to the numbers that are reported by the news media.

The 1995 draft wind speed maps also includes a modification that results in most of the interior of the U.S having a uniform 90 mph wind speed. This adjustment is primarily caused by a change in wind speed data evaluation methods.

Serviceability—The 1995 draft contains a section that addresses the issue of serviceability under wind loads. In addition, the commentary differentiates between serviceability as it relates to occupant perception to motion, and serviceability based on failure of secondary elements, such as partitions and glazing. These distinctions can be of value since the probability of occurrence is different as it relates to these serviceabilities when compared to the occurrence probability as related to the structural capacity of the building. The increased use of limit state design methodologies may provide more opportunities to refine the load in terms of the desired overall building performance.

The following companies returned an exhibitor questionnaire as of March 21:

Company name: **American Institute of Steel Construction, Inc.**
Booth number: 333, 334, 432, 434
Address: One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001
Phone number: 312/670-2400
Fax number: 312/670-5403
Type(s) of products: Manuals and specifications; design aids; magazine & journal publishing; engineering software; fabricator certification; and design studies
Features: Newly published is the Manual of Steel Construction—LRFD (2nd Edition). Also on display are a wide range of AISC publications, including Design Guides, Modern Steel Construction and Engineering Journal. A complete publication list is available by calling the AISC Fax Information Line at 800/644-2400. AISC Marketing offers design studies and sponsors an annual technical lecture series. AISC Software features specialized computer programs for steel design, including CONX-PRT, WEBOPEN, AISC for AutoCAD, and the AISC Shapes Database. AISC offers a Quality Certification program for structural steel fabricators and metal building manufacturers. Membership in AISC is open to fabricators, manufacturers and professional engineers.

Company name: **American Welding Institute**
Booth number: t.b.a.
Address: 10628 Dutchtown Road
Knoxville, TN 37932
Phone number: 615/675-2150
Fax number: 615/675-6081
Type(s) of product: Welding software and consulting; applied research & development; robotics & automation development; computer modeling & simulation
Features: AWI is a non-profit welding institute dedicated to the improvement of welding and joining. Welding Procedure Specification software for the PC allows the storage, searching and printing of WPSs and PQRs. Consulting services cover a broad range, including but not limited to, customized software development, welding procedure development, trouble shooting, equipment recommendations, failure analysis and other welding related services.
Cost: Welding software packages range from \$95 to \$395; other prices depend on service required

Company name: **American Welding Society**
Booth number: t.b.a.
Address: 550 N.W. LeJeune Road
Miami, FL 33126
Phone number: 305/443-9353
Fax number: 305/443-7559
Type(s) of product: Welding literature and codes
Features: Publications include Welding Journal, Welding Handbook, and more than 140 standards developed in accordance with ANSI procedures, including the D1.1 Structural Welding Code—Steel. The 1994 edition of the AWS/ANSI D1.1 contains more than 400 pages of text, tables, figures, charts and illustrations amplifying 11 major sections, including qualification, design of welded connections, workmanship, technique, inspection, stud welding, and statically loaded structures.
Cost: \$132 for AWS/ANSI D1.1 (\$99 for AWS members)

Company name: **Arkansas Steel Processing**
Booth number: 120
Address: P.O. Box 129
Armorel, AR 72310
Phone number: 501/762-1000
Fax number: 501/762-1411
Type(s) of service: Structural steel processing
Features: The company offers processing services to fabricators and steel service centers. Services include: tees split from wide flange beams; cambering; milling of columns; and customized processing. ASP can fully process floor and perimeter beams and girders, including such items as cut-to-length, web and flange hole drilling, cambering, and bolting clip angles. The shop is located adjacent to Nucor-Yamato's Blytheville, AR, mill, which means ready access to raw material with no intermediate freight costs.
Cost: Depends on service

Company name: **Barasel Corporation**
Booth number: 108
Address: Bank of America Tower,
300 S. Harbor Blvd., Suite 500
Anaheim, CA 92805
Phone number: 714/776-3200
Fax number: 714/776-1255
Type(s) of product: Steel detailing software
Features: CompuSTEEL offers automation (enter data only once), ease of use (logical, on-screen menus and visual connection verification), and the automatic creation of standard material drawings, anchor bolt setting plans, erection framing plans, column detailing drawings, and beam detail drawings. It uuns on inside Windows.
Cost: CompuSTEEL is available for a \$5,900 introductory price; the required ARRIS graphics program costs \$2,800

Company name: **Behringer Saws, Inc.**
Booth number: 123, 222
Address: Morgantown Business Park
730 Hemlock Road
Morgantown, PA 19543
Phone number: 610/286-9777
Fax number: 610/286-9699
Type(s) of product: Power hacksaws; horizontal bandsaws; and plate saws
Features: The saws feature: twin column design with columns ground and chrome plated; blade guiding by double roller bearings and hydraulically tensioned carbide drives; drive wheels at approximately a 52 degree angle, which minimizes twist angle and increases blade life; the ability to cut with carbide blades as well as regular bi-metal blades; and the shortest remnant length in the industry.
Cost: $18,750 to $510,000

Company name: **Bull Moose Tube**
Booth number: 228
Address: 1819 Clarkson Road
Chesterfield, M) 63017
Phone number: 314/537-2600
Fax number: 314/537-5848
Type(s) of product: Hollow structural sections
Features: HSS are commonly used as structural columns, roof trusses and complete framing members and meet ASTM A500 in Grades B & C. Bull Moose's HSS are manufactured by a direct form process that assures an extremely tight corner radius, straighter walls and center location of weld (always on the short side).
Cost: Contact vendor

Company: **Canam Steel Corporation**
Booth number: 112, 114
Address: P.O. Box C
Point of Rocks, MD 21777
Phone number: 301/874-5141
Fax number: 301/874-5685
Type(s) of product: Steel joists, girders & trusses
Features: The company designs and manufactures metal construction components—steel joists, steel trusses, structural steel and metal decking—from its five plants in the U.S. and Canada plus related facilities in Mexico, Venezeula, France and China. Canam has developed its own design and estimation software programs to assist engineers and customers in designing efficient and economical roof and floor systems.
Cost: Contact vendor

Company name: **Dogwood Technologies, Inc.**
Booth number: 233, 332
Address: P.O. Box 52831
Knoxville, TN 37950-2831
Phone number: 800/467-0096
Fax number: 615/584-6032
Type(s) of product: Detailing Software
Features: PDS is an automated detailing system that operates under UNIX rather than DOS. UNIX offers a vast speed increase compared with DOS because of its ability to utilize a 32-bit mode of operation and to provide true multi-tasking. PDS provides automated detailing of: beams and columns including standard parts; vertical bracing of X, V and A types of single and double angle and WT members; horizontal bracing with variable drops; pan and grated stairs with breakovers and landings; automated engineering calculations for connection design; material management including material billing, sequencing, sorted cut lists, automated weights and surface area calculations and shipping tickets; fabrication control for downloading to CNC equipment; and an integrated CAD graphics module set-up specifically for steel detailing applications.
Cost: Contact vendor

Company name: **Huck International, Inc.**
Booth number: 226
Address: P.O. Box 8117
Waco, TX 76714
Phone number: 817/751-5228
Fax number: 817/751-5259
Type(s) of products: Bolts
Features: Offers wide range of bolts from $^3/_{16}$-in. diameter up to $1^3/_8$-diameter, both A325 and A490 with one-sided installation. At the show will be demonstrating the new blind fastening system, which is an A325 high-strength equivalent that can be used in place of welding.
Cost: Contact vendor

Company name: **J&M Turner Inc.**
Booth number: 403
Address: 1310 Industrial Blvd.
Southampton, PA 18966
Phone number: 800/525-7193
Fax number: 215/953-1125
Type(s) of product: Direct tension indicators
Features: Direct Tension Indicators (DTIs) are similar a direct reading load cell for structural bolts. They are the only product that, when installed with a structural nut, bolt, and washer

assembly, positively identifies that the bolt is properly tensioned. The company offers DTIs in both metric and inch sizes.

Cost: Contact manufacturer

Company name: **The Lincoln Electric Co.**
Booth number: 318
Address: 22801 St. Clair Ave. Cleveland, OH 44117
Phone number: 216/481-8100
Fax number: 216/486-1751
Type(s) of product: Welding power source
Features: The Invertec V300-Pro is a 15 to 300 amps arc welding power source at a 60% duty cycle that utilizes single phase or three phase input power to produce either DC constant current or DC constant voltage outputs for stick and wire welding. Because its lighter in weight than traditional transformer/rectifier models, it can be used on jobs that require multi-purpose capability in confined spaces. It is recommended for GMAW, FCAW, SMAW, GTAW and Air Carbon Arc Cutting.
Cost: Contact vendor

Company name: **NAPTech (North American Piping Technologies)**
Booth number: 224
Address: 851 S. Freeport Industrial Pkwy. Clearfield, UT 84015
Phone number: 801/773-7300
Fax number: 801/773-6185
Type(s) of service: Bending (pipe, tubing & structural shapes)
Features: The company offers high quality bending of structural shapes with distortion held to a minimum. The company prides itself on a large size range and for having a staff metallurgist to ensure quality.
Cost: Dependent on application

Company name: **National Institute of Steel Detailing**
Booth number: t.b.a.
Address: 300 S. Harbor Blvd., Suite 500 Anaheim, CA 92805
Phone number: 714/776-3200
Fax number: 714/776-1255
Type(s) of product: Trade Association
Features: NISD is a nationwide association of steel detailing firms and independent detailers established in 1969. At present, it consists of nine regional chapters, scores of members at large, associate member firms and

individual associate members. Its Quality Procedures Program (QPP), available to members and non-members alike, stresses the need for quality and assures clients of consistent procedures. Its soon-to-be-published "Industry Standards" guide explains the role of the steel detailing industry as part of the construction environment. The long-awaited individual detailer certification (IDC) is in its final preparation and will soon be ready for implementation.

Cost: Contact vendor

Company name: **Nitto Kohki USA**
Booth number: 400, 402
Address: 808-C North Central
Wood Dale, IL 60191
Phone number: 708/860-9595
Fax number: 708/860-0096
Type(s) of product: 1. Magnetic drills; hydraulic punches; and steel beveler
Features: The magnetic drills are totally computerized for automatic drilling of holes up to 2 in. in diameter through up to 2-in. thick steel. No adjustments are needed for size or hardness; the tool drills a hole, drops a slug and returns to the upper position and shuts off.
The light-weight, portable hydraulic punches make 1-in. diameter holes through 5/8-in. mild steel in 13 seconds.
The steel beveler produces a smooth bevel face that eliminates post finishing. It easily adjusts for depths up to 5/8 in. and for angles of 15 degrees to 45 degrees and works faster than either flame cutting or nibbling.
Cost: Magnetic drills—$895 to $3,600
Hydraulic punches—$3,800 to $4,800
Steel beveler—$2,350

Company name: **Metrosoft**
Booth number: 201, 203
Address: 332 Paterson Ave.
E. Rutherford, NJ 07073
Phone number: 201/438-4915
Fax number: 201/438-7058
Type(s) of product: Structural analysis and design software
Features: Robot V6 offers a fully intergrated graphical system for high-speed, interactive 2D and 3D frame and finite elements. The program provides static, dynamic, bukling linear

and non-linear analysis as well as sophisticated moving load generation. It can design very large structures with up to 32,500 nodes. It interfaces with CAD programs through the DXF format and supports Novell netware and other networks. A macro-recorder, as well as libraries of structures and componets, is built-in.

Cost: $495 to $8.075

Company name: **Nucor-Yamato Steel**
Booth number: 122, 124, 126
Address: P.O. 1228
Blytheville, AR 72316
Phone number: 800/289-6977
Fax number: 501/763-9107
Type(s) of product: Wide flange structural shapes, channels & miscellaneous channels, and H-piles
Features: These steel products are melted and manufactured in Arkansas in a state-of-the-art rolling mill facility that features electric arc furnace (100% scrap-based), ladle metallurgy furnace and continuous casting technology producing a beam blank that approximates the net shape of the final beam. A low carbon aim (0.06% - 0.12%) combined with continuous casting provides the end user with a product that has exceptional internal and surface quality with improved weldability, toughness and corrosion resistance. Annual capacity is about 1.6 million tons per year.
Cost: Refer to company's current price list

Company name: **Peddinghaus Corp.**
Booth number: 307, 309, 311, 313, 406, 408, 410, 412
Address: 300 N. Washington Ave.
Bradley, IL 60915
Phone number: 815/937-3800
Fax number: 815/937-4003
Type(s) of product: Cutting, bending, punching equipment
Features: Newly introduced this year is the Model 38-18 Structural Band Saw, which features a unique electronic monitoring system for enhanced blade speed (auto feed). It specifically addresses structural steel sections by optimally feeding the blade to contact the section simultaneously. It also offers mitre cutting, mist collant, and two-in. blade available.
Also newly introduced is the ABCM 1250 CNC Structural Burning Systems an automated, computerized structuring burning of common

structural shapes. It offers a 10:1 productivity gain over manual methods. There is virtually no manual set up and is easily adaptable to long and short production runs.

Cost: Model 38018: $60,000
Model ABCM 1250: $270,000

Company name: **PENware Corp.**
Booth number: 206
Address: 619 South Cedar St., Studio A
Charlotte, NC 28202
Phone number: 704/344-9644
Fax number: 704/358-1801
Type(s) of product: Pen computer interface for CadVantage
Features: PenVantage 2.6 is a leading Windows software system that runs on portable pen-based computers. The program serves as a front end tool for CadVantage (version 5.6 or higher). PENware claims to reduce CVS input time by more than 50% without sacrificing system flexibility. Users get instant graphical feedback from their input. The system makes full use of an extensive combination of both piece-by-piece and grid input detailing methods. For example, double clicking on a column grid brings up the columns input screen showing all information generated from the grid program. Detailers can input information for one grid point, and then copy the data to any other grid points. Connections can be applied by specifying beam piece marks or sizes. The system also can connect with production software systems through the CVS materials download function. The major benefits from the PENware interface are exceptional speed and the freedom for detailers of not being tied to a computer terminal. They can use the pen tablet as an electronic job sheet by: writing on the screen (PenVantage will recognize handwriting); tapping a pop-up number/letter pad; or by selecting user-defined icons. These icon buttons can represent entire end, mid-span and base/cap conditions, or shapes and dimensions. Complete beams or columns can be detailed by tapping as few as four buttons. A new set-up system allows all connections to be defined prior to beam/column take-off, thus reducing the input process to a matter of telling the program which shapes connect together. Interested parties can request a demo version of the PENware interface to be included with the demo kit for CadVantage Structural.
Cost: Not determined at press time

Company name: **Ram Analysis**
Booth number: 212
Address: 5315 Avenida Encinas, Suite M
Carlsbad, CA 92008
Phone number: 800/726-7789
Fax number: 619/431-5214
Type(s) of product: Structural engineering software
Features: RAMSTEEL is a special purpose structural engineering software package for the analysis, design and drafting of steel buildings. This software provides an advanced level of design capability for the gravity load resisting elements within structures. From conception through design calculations and construction drawings, the program automates the entire process. New for V4.0 are numerous modeling features, including enhancements for beam layout and point load layout. Scaleable text on floor maps showing beam sizes, composite studs, camber and reactions is now available. The size limitations have been doubled and the DXF format CAD drawing capabilities have been enhanced to include more user control over the drawing parameters. Also, numerous enhancements are included to facilitate connection design, including an interface to AISC's CONXPRT software.
Cost: Contact vendor

Company name: **St. Louis Screw & Bolt**
Booth number: 303
Address: 6900 N. Broadway
St. Louis, MO 63147
Phone number: 314/389-7500
Fax number: 314/389-7510
Type(s) of product: Structural fasteners
Features: The company offers 1/2-in. diameter through 1-1/2-in. diameter heavy hex structural bolts (Types I & III). The bolts are produced, packed and shipped to customer requirements and tested, certified and quality controlled at the St. Louis Screw & Bolt laboratory. All fasteners are made from steel melted and manufactured in the U.S. Larger diameter bolts are available to 3-in. diameter to meet customer specifications.
Cost: Contact manufacturer

Company name: **Southern Coatings, Inc.**
Booth number: 110
Address: P.O. Box 160
Sumter, SC 29151-0160
Phone number: 803/775-6351
Fax number: 803/775-7666
Type(s) of product: High-performance coatings for steel
Features: The coatings offer a number of benefits, including: VOC compliance; lead and chrome free; high solids; and water base technology. The company prides itself on its service, especially

its field technology service, engineering activity and preparation of specifications.
Cost: Varies depending on coating system

Company name: **Structural Software Co.**
Booth number: 401
Address: 5012 Plantation Road
P.O. Box 19220
Roanoke, VA 24019
Phone number: 703/362-9118
Fax number: 703/366-6036
Type(s) of product: Software for fabricators
Features: The company's estimating and production software is in use by more than 450 fabricators nationwide. The software is available in interactive modules, so users can build a system to match their needs. The Estimating package calculates weights, surface area, material cost and labor. It will even count shop and field bolts. The Production Control module allows the job to be released into the shop by sequence, drawing number, category, main piece or accessory piece. The user can then track pieces from station to station. This module is particularly useful for fabricators seeking AISC Quality Certification. The Inventory Control module tracks stock items as well as drops left over from previous jobs. The Purchase Orders module automatically integrates with the Inventory Control module. And the Combining module allows the user to optimize material cutting. A Nucor bundling option also is available.
Cost: Contact vendor

Company name: **TradeARBED, Inc.**
Booth number: 106
Address: 825 Third Ave.
New York, NY 10022
Phone number: 212/486-9890
Fax number: 212/355-2159
Type(s) of product: ASTM A913 HISTAR Steel Grades; WTMs Special WF (Tailor-Made) Rolled Sections
Features: ASTM A913/HISTAR Steel Grades are produced using the QST method, which results in a high yield strength (including 65,000 psi), excellent ductility and low temperature toughness, outstanding weldability due to a very low Carbon Equivalent, and weldability without preheating. HISTAR Steel Grades are available in most standard and tailer-made sizes. Many jumbo sizes can be supplied to AISC Heavy Shape impact test specifications. These sections are available at a competitive price and are ideal for high-rise buildings, hospitals and long-span structures. ARBED also produces the largest range of WF and WTM sizes of any mill.
Cost: Contact vendor

Company name: **Welded Tube Co. of America**
Booth number: 104
Address: 1855 E. 122nd St.
Chicago, IL 60633
Phone number: 312/646-4500
Fax number: 312/646-6128
Type(s) of product: Structural tubing; KleenKote Tubing & Pipe
Features: Structural tubing offers several advantages over conventional open profile sections, including: less weight per section to accommodate the same allowable load for columns and other compression members, or higher allowable loads for sections of equal weight; longer allowable unbraced lengths for members in bending and compression applications; greater torsional rigidity than open profiles; and approximately 35% less surface area to coat and fireproof. KleenKote is a mechanically cleaned, degreased and pre-primed coated section that is ready to fabricate. Many of the costly preparation, cleaning and layout stages can be streamlined with KleenKote, reducing time and cost.
Cost: HSS averages between $600 and $960 per ton

Company name: **Yamazen Inc.**
Booth number: 211, 213, 310, 312
Address: 735 East Remington Road
Schaumburg, IL 60173
Phone number: 708/882-8800
Fax number: 708/882-4270
Type(s) of product: Structural band saw machines; CNC structural drilling machines
Features: The saws feature a patented amplifying valve feed system, unique three contact point mitering method, six degree saw bow angle and carbide roller ring back-up glides.
The CNC drilling machines feature an exclusive "fixed workpiece-travelling spindles" method, which allows for faster hole location without repositioning heavy beams and results in greater accuracy. The machines also offer CNC controls, web support for thinner sections, pinch roller infeed, large diameter measuring wheels, angle drilling capabilities, web height gage and an innovative hold down system.
Cost: Varies according to model

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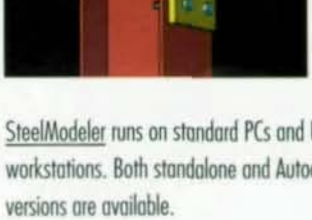
* Bills of materials



SteelModeler runs on standard PCs and Unix workstations. Both standalone and Autocad versions are available.

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For more information, contact:

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