MODERN STEEL CONSTRUCTION

October 1995

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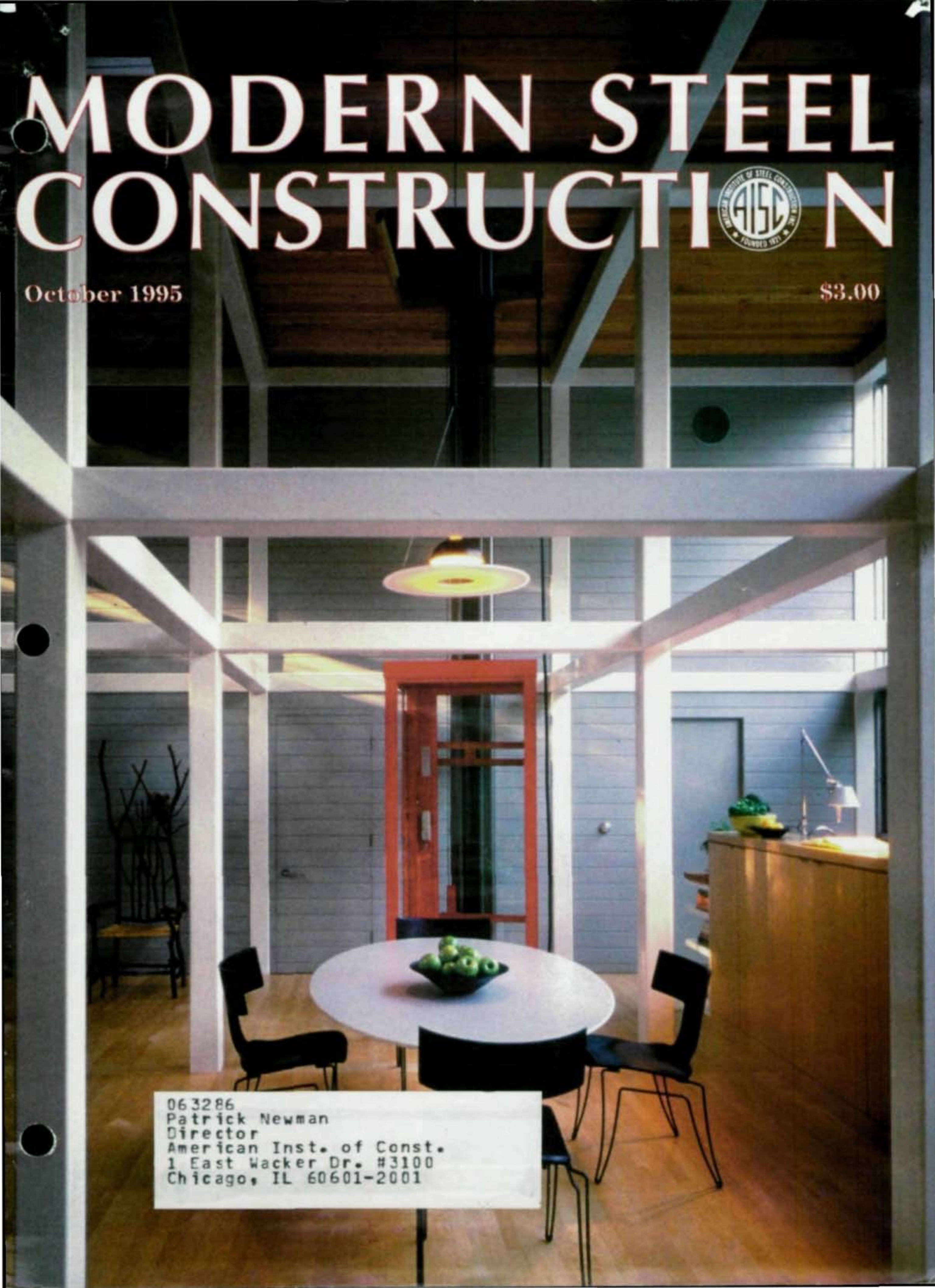

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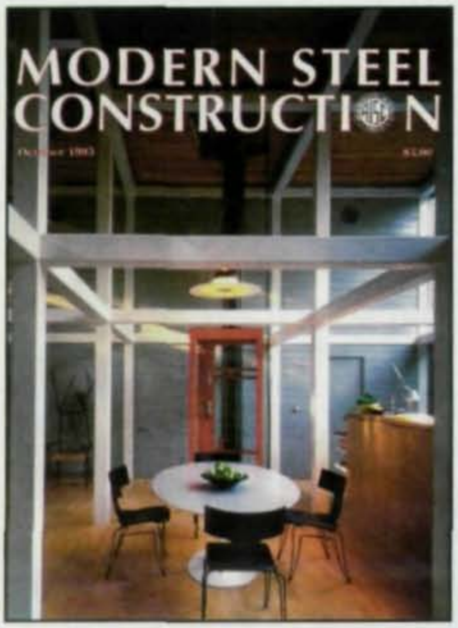
MODERN STEEL CONSTRUCTION

Volume 35, Number 10 **October 1995**



A tubular steel frame created a symmetrical 3-dimensional space in a new home home in Maine. For more information on this beautiful project, *turn to page 28.*

Modern Steel Construction (Volume 35, Number 10). ISSN 0026-8445. Published monthly by the American Institute of Steel Construction, Inc., (AISC), One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Advertising office: Facinelli Media Sales, O'Hare Lake Office Plaza, 2400 E. Devon Ave., Des Plaines, IL 60618 (708) 699-6049.

Subscription price:
Within the U.S.—single issues $3; 3 years $85.
Outside the U.S.—single issues $5; 1 year $36; 3 years $100.

Please fax any address changes to 312/670-5403 or to the address below.

Postmaster: Please send address changes to Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

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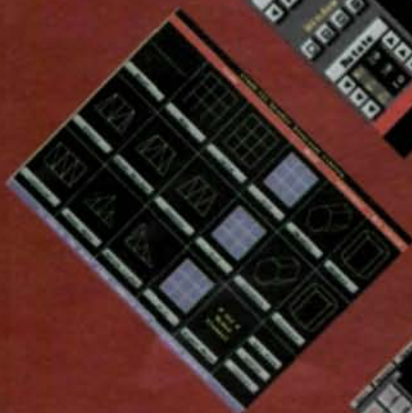

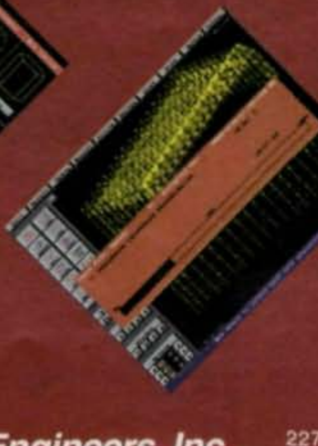

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Technologically Current

LAST WEEK, ON A DAY FILLED WITH MICROSOFT HOOPLA, ONE OF THE VICE PRESIDENTS at AISC asked me if I had purchased a copy of Windows 95 yet. As much as I wanted to be the first kid on the block with that new toy, it turns out that I won't soon be buying it. My problem is that, of the three major software packages I use on a near-daily basis, under the new operating system one program, Microsoft Word, will run just fine; the second, Adobe Photoshop, won't run properly and I can't get any information about the third, QuarkXPress.

The problem I'm having having with Windows 95 reflects the same problem many designers have. Will AutoCAD run? How about STAAD-III or ETABS? Or SDS/2 or Computer Detailing? Heck, a lot of the programs used by engineers, fabricators and detailers are still DOS-based (and, of course, there's always that handful of professionals using Macintoshes). Computer technology is advancing so fast that it is nearly impossible to keep up—especially given the financial impact that comes with purchasing new hardware and software.

While we will never reach the Utopian vision of a paperless office, it's become increasingly obvious that in the future we will use more and more electronic files. Unfortunately, the myriad vendors serving the steel design and construction industry have not yet reached an agreement on a uniform standard that will allow users of one program to read data from another. Still, it's possible to make some educated guesses as to where the industry is going and what type of equipment is needed. Even so, if you're thinking about buying a new computer system, keep some thoughts in mind:

1. Go Pentium. While you can buy a 486-based computer dirt cheap today, it's not going to effectively run the next generation of software—a generation that's due out practically tomorrow.

2. Get at least 16 megabytes of RAM. That's the minimum to run Windows 95 efficiently.

3. Get a whopping huge hard drive. I can remember buying a computer for home use five or six years ago and thinking that my 80 megabyte hard drive would be all I'd ever need. Now you need that much room just to properly load the newest operating systems.

4. Take a good look at 17-in. or even 20-in. monitors. It makes working a lot easier.

5. Buy a quad speed CD and a 28.8 bps modem. In a recent survey, almost all companies already had a modem and almost everyone was looking to get a CD. And actually, you'd be hard-pressed to buy a system today that doesn't already include these. Both will gain greater importance in the near future. CDs because more and more software—and publications—are being published on that platform and modems because bulletin boards and Internet access are becoming increasingly common and useful.

6. And finally, when you do buy software, ask the vendor when they'll be upgrading to Windows 95. Yes, the program will take up more room. But it will also be easier to run and, presumably, it will be faster. **SM**

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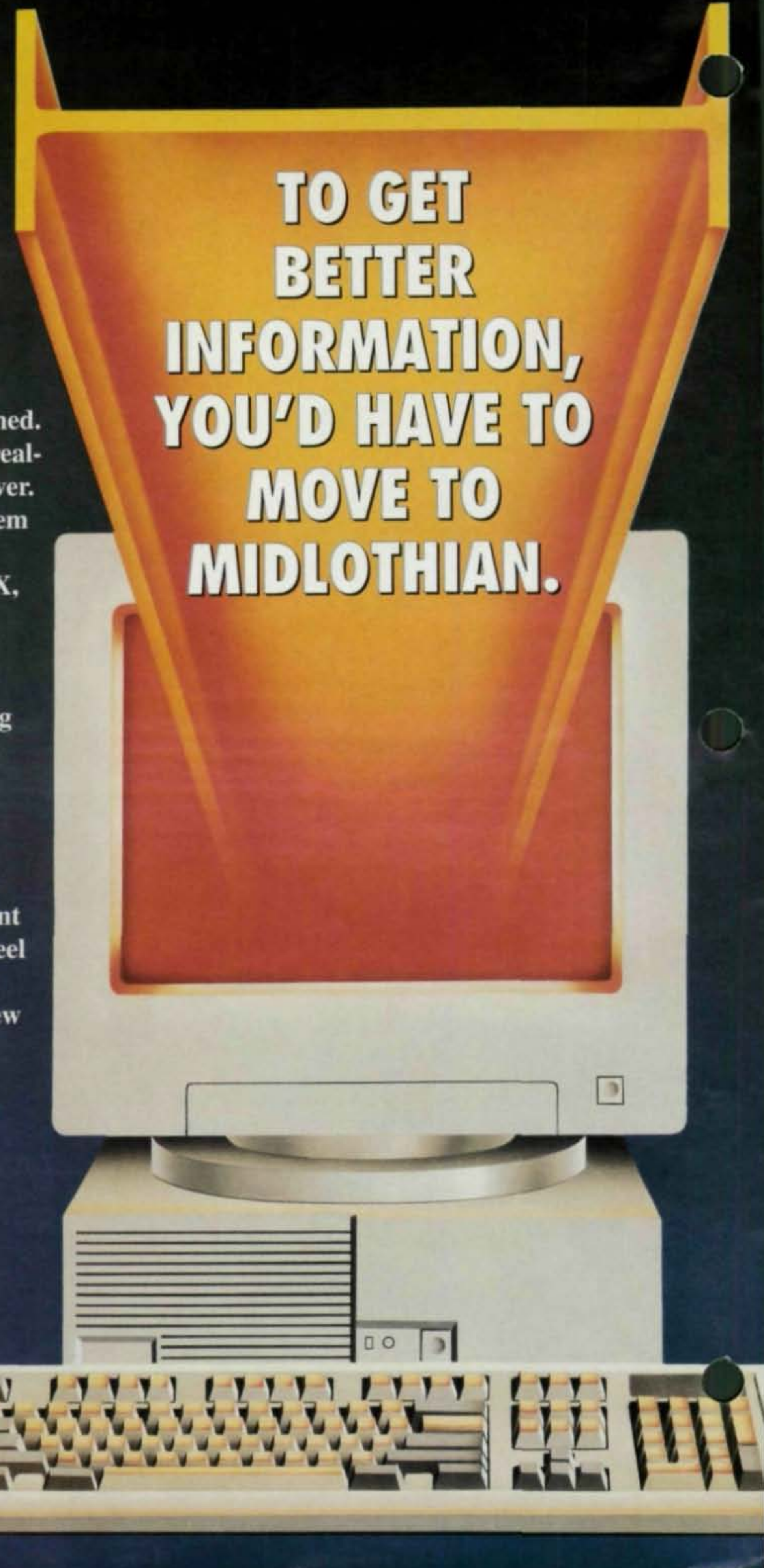

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STEEL INTERCHANGE

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

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

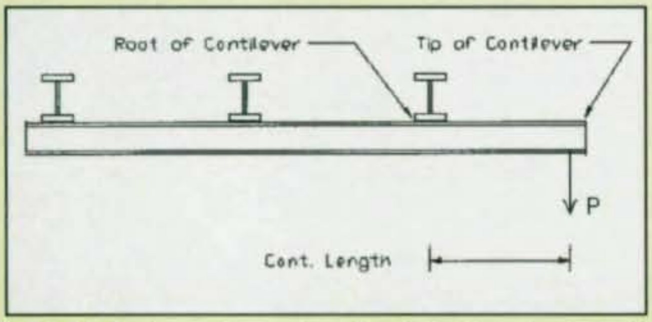
Answers and/or questions should be typewritten and double-spaced. Submittals that have been prepared by word-processing are appreciated on computer diskette (either as a Wordperfect file or in ASCII format).

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

Information on ordering AISC publications mentioned in this article can be obtained by calling AISC at 800/644-2400.

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

For a continuous trolley beam with multiple spans and cantilevered ends what is the lateral unbraced length for the bottom flange? Can the distance between points of inflection be considered an unbraced length?



We have been involved in the design of many monorail and bridge crane systems that had similar cantilever conditions. Since the number of the systems investigated was very large, we did some research in this particular problem, and we are preparing a paper for publication of the findings.

If the beams are doubly symmetric sections, such as wide flanges and S shapes, one suggested solution is given in the Guide for Stability Design Criteria for Metal Structures, edited by T.V. Galambos, section 5.2.4 page 168. The procedure gives the critical buckling moment, M_{cr}, for the cantilever to be:

where, M_{cr} is the theoretical critical buckling

$$M_{cr} = \frac{\pi}{KL}\sqrt{EI_yGJ}\sqrt{1 + \frac{\pi^2 EC_w}{(KL)^2 GJ}}$$

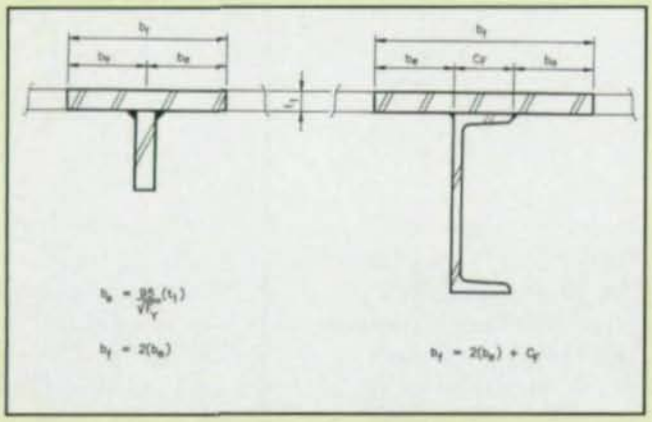
moment without any factor of safety, L is the cantilever length, K is an effective length factor, E is the modulus of elasticity, G is the shear modulus, I_y is the minor axis moment of inertia, and Cw is the warping constant, and J is the torsional constant. Both J and C_w are provided in the AISC Manuals for standard wide flange and S shapes. The value of K varies depending on the restraint conditions at the root and at the tip of the cantilever, as well as the location of the load with respect to the neutral axis (as indicated by figure 5.11 of the above reference). For the case in question, where the cantilever is continuous over the root with only top flange laterally restrained at the root, no lateral restraints at the tip, and bottom flange loading, the reference suggests a value of K of 3.0.

This will result in a value for the critical buckling moment. An appropriate factor of safety, typically in the range of 1.67 to 2, should be applied too obtain the allowable moment. In addition, this allowable moment should be limited to 0.66 of the yield moment for compact sections and 0.6 of the yield moment for non-compact sections.

In effect this method gives an unbraced length of 3.0 times the cantilever length, and there is no need to use the unbraced length to the inflection point. We should stress the fact that this method does not apply to singly symmetric beams, i.e. patented track, that are frequently employed for trolley support.

Hussain Shanaa, Ph.D., P.E.
Jehangir Rudina, P.E.
AEC Engineering, Inc.
Minneapolis, MN

Given a wall of sheet metal or plate subjected to fluid pressure and stiffened by same size parallel members spaced regularly, what section or (or width) of the wall shall be used that contributes to the section of a stiffener? The stiffening member may be a flat bar, an angle, a channel or any other section.



The effective width contributing plate section should be limited to the width thickness ratios for compression elements as found in Table B5.1 of the AISC Allowable Stress Design Specification for Structural Steel Buildings. As a bending member, the maximum b/t ratio should be limited to less than $95/\sqrt{F_y}$, to be considered as fully effective. A general rule of thumb is to consider a total plate width contribution of 32t for structures comprised of A36 steel, with a corresponding allowable bending stress of $0.6F_y$.

The figure depicts these recommended limits.

Vincent E. Kokal, P.E.
Alliance Engineering, Inc.
Richmond, VA

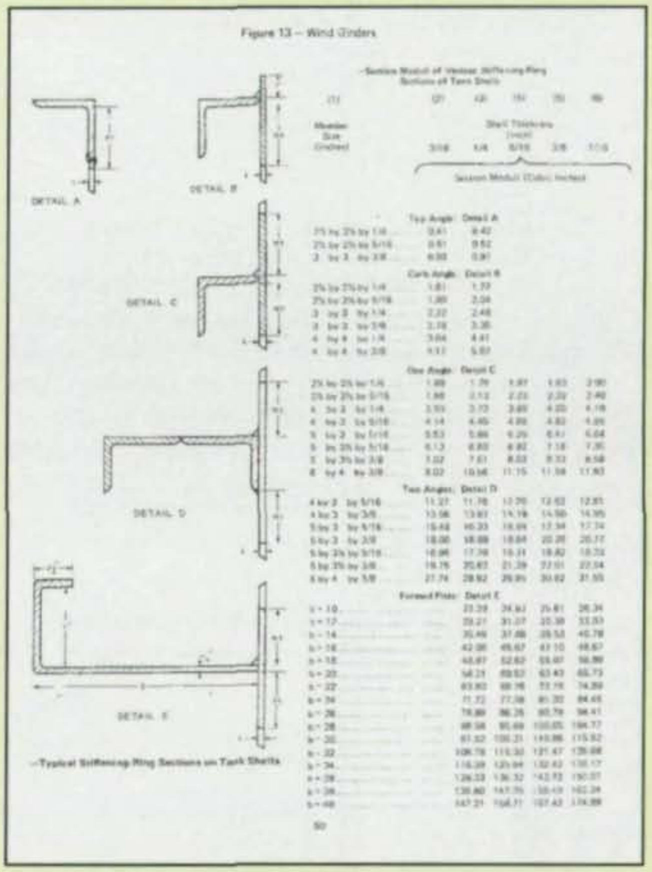
ANOTHER RESPONSE:

The effective projection of the plate on either side of the stiffener being in contact with the plate should be 16 times the thickness of the plate: Thus, $b_e = t + 32t_2$.

This common practice to obtain a transformed section has been widely used in the design of tanks for liquid storage per standards by the American Petroleum Institute (API).

The American Iron and Steel Institute in its publication entitled Steel Tanks for Liquid Storage, revised Edition 1976 provides a table for the section for the section moduli of the stiffening ring sections based on the 16t effective projection on each side of the stiffener.

Isaac Gordon, P.E.
Ang Associates, Inc.
Philadelphia, PA



Questions:

Has any engineering firm ever designed a multi-story unbraced frame using mainly semi-rigid (partially restrained) connections? Which, if any, computer programs were used to assist in the analysis and design? What are some of the major pitfalls in using partially restrained moment connections?

Michael G. Klozik
Medford, MA

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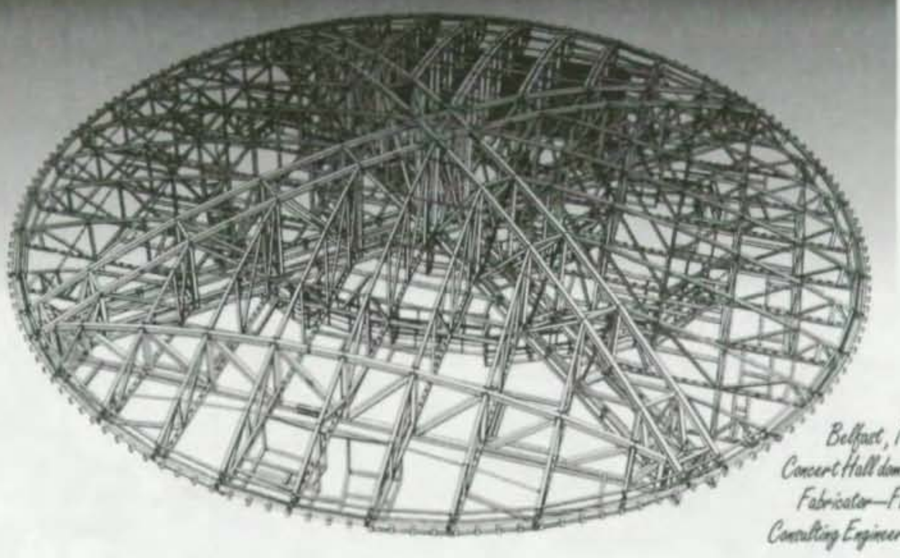

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From the solid model, it is possible to produce all drawings and material lists completely automatically. STRUCAD 2000 automatically creates field erection drawings, shop detail drawings, fitting details, full-size templates, 3-D erection drawings, and CNC manufacturing data for a wide range of workshop machinery. Data downloads from STRUCAD 2000 directly to CNC machinery to ensure accurate and consistent results.

Integration between STRUCAD 2000's design and detailing data and the production control and management data of STEEL 2000 is seamless. Structures designed in STRUCAD 2000 will flow through all aspects of estimating, detailing, multing, purchasing, receiving, production, shipping and erection with complete traceability. The estimated labor to produce any structure that is designed or detailed in STRUCAD 2000 can be printed from STEEL 2000 Estimate with a few keystrokes.

STRUCAD 2000 is interfaced to a large variety of other software packages including AutoCAD applications, STAAD-III from Research Engineers, Intergraph's Mica Plus Analysis and PDS, PDMS from Cadcentre, and PASCE from CE Automation, to name only a few.

Proven in production for six years, there are more than 400 STRUCAD systems installed throughout 22 countries. Now available in imperial units, Steel Solutions is pleased to market and support STRUCAD 2000 in the United States.

To facilitate use of this powerful tool in your plant, Steel *Solutions offers a low-cost, five-day training course for* your employees at our Training Center in Jackson, Mississippi. In addition, regular user group meetings at the Training Center will ensure that on-going requirements are constantly being integrated into STRUCAD 2000.

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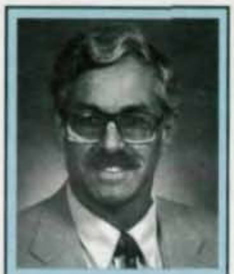
CALL FOR ENTRIES

NOMINATIONS ARE INVITED FOR THE 25TH ANNUAL T.R. HIGGINS LECTURESHIP AWARD to be presented in 1996. The award, given by the American Institute of Steel Construction, Inc., recognizes an outstanding lecturer and author whose technical paper or papers published between Jan. 1, 1990 and Jan. 1, 1995 are considered outstanding contributions to the engineering literature on fabricated structural steel. Past winners have included practicing structural engineers, fabricators and educators.

In addition to a framed certificate, the award recipient will receive $5,000 and will give a minimum of six lectures during 1996. The award will be presented based on the jury's evaluation of the paper(s) for originality, clarity of presentation, contribution to engineering knowledge, future significance and value to the fabricated structural steel industry. In addition, the nominees reputation as a lecturer will be considered.



William A. Thornton, the 1995 Higgins Award Winner

Nominees must be permanent residents of the U.S. and available to fulfill the commitments of the award. Nominations should include: name and affiliation of person nominated; title of paper(s) with publication citation (in the case of multiple authors, please identify the principle author); reasons for nomination; and a copy of the paper(s) (as well as any published discussion.

This year's jury consists of: Horatio Allison, a Dagsboro, DE, based consultant; Reidar Bjorhovde, professor of civil engineering at the University of Pittsburgh; Tim Fraser, division chief engineer, Canron Construction-Fabrication West, New Westminister, BC, Canada; John Gross, research structural engineer, NIST, Gaitherburg, MD; and William McGuire, past Higgins Award recipient and professor emeritus at Cornell University.

Recent recipients of the award included fabrication engineer William A. Thornton for his work on connection design; consultant Lawrence Griffis for his papers on composite frame construction and associate professor Roberto Leon for his work on semi-rigid composite connections.

Nominations should be sent to: Committee on Education, AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001. Deadline for submissions is November 3, 1995.

CALL FOR PAPERS

AUTHORS ARE INVITED TO SUBMIT PAPERS RELATED TO "STABILITY PROBLEMS IN DESIGN, CONSTRUCTION AND REHABILITATION OF METAL STRUCTURES" for the 5th International Colloquium on Structural Stability to be held on August 5-7, 1996 in Rio de Janeiro, Brazil.

English abstracts of 500 words or less should be submitted by November 30, 1995 to: SSRC IC/Brasil '96, COPPE/UFRJ-Program de Engeharia Civil, C.P. 68506, 21945-970 Rio de Janeiro, Brazil. Phone: (5521) 280-9933; fax: (5521) 290-6626; e-mail: SSRC_RIO@LABEST, COC.UFRJ.BR

FREE PUBLICATIONS

FIVE RECENT PUBLICATIONS FROM AISC MARKETING, INC. ARE AVAILABLE AT NO CHARGE to engineers, architects, fabricators, contractors and others involved in the design and construction of fabricated steel structures.

• ***Myths and Realities of Steel Buildings*** is a 14-page booklet that discusses such topics as: prices for fabricated structural steel; U.S. steel in the international market; why the least-weight structure is not always the most economical design; the effect of mill deliveries on construction schedules; uniform reliability and design time savings with LRFD; deflections and vibration in LRFD design; seismic design issues; connections; and painting/fire proofing of steel structures.

• ***Myths and Realities of Steel Bridges*** is a 23-page booklet that answers many questions that bridge engineers have about recent developments in steel design. Topics include: prices for fabricated bridge steel; mill delivery schedules; durability of concrete and steel; life cycle performance of steel bridges; weathering steel performance; optimization by weight and span length; jointless bridges; bearings; painting; and fatigue life of details vs. structure service life.

• ***Cantilever Roof Framing Using Rolled Beams*** is a 16-page booklet describing an alternative to joist girders and presenting sample design tables.

• ***Steel In Design*** is a six-page case study on the design and construction of the new San Francisco Museum of Modern Art.

To receive any of these publications, fax your request to *312/670-5403* or mail your request to: AISC Marketing, Inc., One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

ENGINEERING JOURNAL ANNOUNCES COVER COMPETITION

AISC IS HOLDING ITS FIRST EVER COMPETITION FOR INTERESTING PHOTOS to feature on the cover of *Engineering Journal*, the leading technical publication for the steel design and construction community. The winning photos will be used on covers in 1996 and detailed descriptions of the project, along with the names of the project team, will be included on the inside front cover of the journal.

Entries can picture either a steel building or bridge. Construction shots and photos showing connection details are preferred, but if exposed steel is used then photos of the completed project are allowed. In addition, photos showing fabrication or erection are encouraged. All submissions must be in color and the pictures should in a "portrait" mode (that is, they should be able to be presented in an $8^1/_2$" wide by 11" high format).

Pictures should be submitted either as $8^1/_2$" x 11" prints, slides, negatives, 4x5 or 2x2 transparencies, or on a Photo CD. Please include a signed photo release giving AISC permission to use the images. Deadline for submission is November 1. Please send entries to: Scott Melnick, AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

NSCC Adds Two More Tracks

COMPUTERS AND ERECTION WILL RECEIVE ADDED ATTENTION at the 1996 National Steel Construction Conference. In addition, specialized tracks will be offered on: engineering—technical; engineering—management; fabrication; and construction management.

"There was a lot of enthusiasm generated last year when we first introduced specialized tracks," explained Franklin B. Davis, chairman of AISC's NSCC Committee. "The two new tracks are designed to continue and expand on that excitement.

"There has long been a feeling in the engineering consulting community that the erection of steel buildings has not received as much attention as it should. Adding a track on erection should help bring some key issues to the forefront. And adding a computer track is a natural for the show, given its wide attendance from all segments of the steel industry. The advances being made in software have been tremendous during the past few years. The NSCC is an ideal forum where the user and the producer can get together to interface for the benefit of all parties," Davis concluded.

The 1996 Conference will be held in Phoenix on March 27-29. In addition to more than 30 technical sessions offered, there will be an exhibit hall with nearly 70 product exhibitors. The sessions qualify for CEU credit and time also is available for extensive industry networking. Also, an extensive package spouse/family activities are scheduled.

To receive an information and registration packet, please fax your request to: NSCC at 312/670-5403.

Beauty And The Beast

A FEW YEARS AGO, DISNEY'S BEAUTY AND THE BEAST'S popularity raised it above over all other movies in the minds of children and parents alike. This past summer, the Beast literally towered over Los Angeles.

To celebrate the opening of a live musical version of the movie at the Shubert Theater in Century City, CA, Walt Disney Studios commissioned a 27-ft.-tall sculpture of the Beauty and the Beast logo. The sculpture was formed from .188-in. aluminum sheet with the edges GMAW welded using 3/64 aluminum 5356 wire. The actual sculpture is 15-ft. tall by 14-ft. wide by 8-in. thick and stands on a 12-ft. base. It was cut from two sheets of aluminum, which were then welded together with 8-in. spacers in-between to create the appearance of a cast sculpture.

To keep the "free-floating" elements together, 3-in.-diameter tubing was used. To minimize the visual impact of the tubing, it was painted a non-reflective, flat back. The upright support tubing has four extension legs, each with pegs on the ends. The pegs enable the installer to add round weight plates, each weighing 100 lbs., to prevent the sculpture from falling over.

The sculpture was installed above a fountain and was designed to look as though it was floating above the water's surface.

The finish on the sculpture was achieved by random orbital sanding of the aluminum to create a swirl effect. The high polish on allows creates a lustrous effect, which is enhanced by the myriad of colors reflected from the lights shining on the fountain. Most of the sculpture—except for the rose, which was first painted with an expensive candy apple red metallic coating—was clear-coated to protect the finish. The coating on the rose is semi- transparent, which allows the finish to show through and creates an intense sparkling effect.

While the finished piece looked as though it is one piece, it was actually made three parts that could be shipped to the site and then easily bolted together. Total weight of the sculpture is 400 lbs. Fabricators was AISC-member FTS, Inc. and the sculpture was designed by Sculptures and Props Unlimited (Todd Jones, owner).

The sculpture was removed on July 8 and is currently in storage, awaiting another performance.

THE DE
IS STACKED IN

CREATIVE MOVEMENT

Shifting two columns in an occupied apartment building with no existing drawings required careful monitoring of a series of strain gages

By Gary Steficek, P.E.

WHAT DO YOU DO WHEN YOU DON'T HAVE SUFFICIENT INFORMATION on which to base your engineering decisions? In one recent design, we made conservative assumptions, tested them with strain gages and proceeded very cautiously.

The project involved the renovation and conversion of a floor of an apartment building into a single residence. The architectural design, by Johnson and Wanzenberg, New York City, involved rearranging all of the interior walls and also necessitated relocating two columns to create an enlarged living room with symmetrically located columns. Adding to the problem, the size of the columns needed to be minimized so they could be architecturally clad. Fortunately, however, there was space available to add a third slender column. All three of the new columns are double extra strong (XXS) 6-in. round pipes welded to the existing structure.

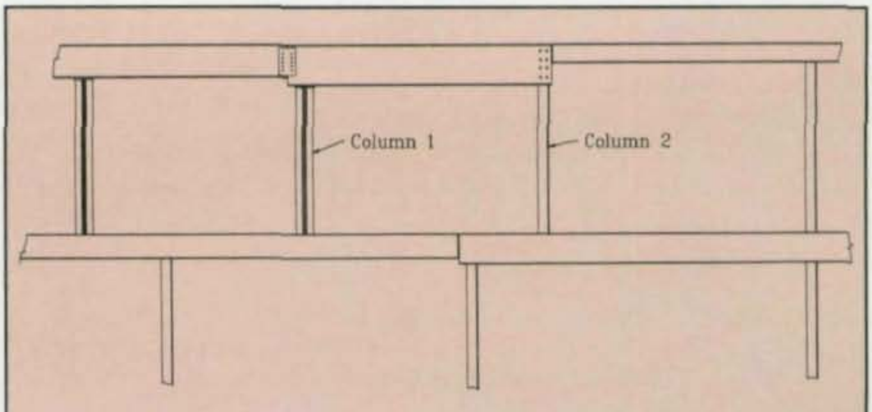
The project was in a 25-story apartment building on Central Park South in New York City. Allegedly, it was the last building constructed in New York City prior to the end of World War II. The lower floors were constructed of reinforced concrete slabs and columns while the upper floors were constructed of structural steel columns and beams and reinforced concrete slabs. However, the sizes of the steel beams seemed excessively large. This may have been the result of steel shortages as WWII began, which could have resulted in the contractor using whatever larger sized members were available. The structural objective was to relocate the two columns so as not to increase the load on any of the columns below and to reinforce the beams above and below as needed.

As with most renovation jobs, nothing is ever simple. The first problem to crop up was the lack of structural drawings of the existing building. Building department searches came up empty. Next was the design of the building itself. The building stepped back in plan on each of the four floors above the apartment, which meant the beams were already picking up a transfer load. Additionally, our design needed to align the columns, even though the beams they were supported by were not aligned and the beams they were supporting were not aligned. Additionally, there was a 150 kip water tank on the roof. And finally, as is often the case with renovation work, the work couldn't interfere with the occupied floors above and below.

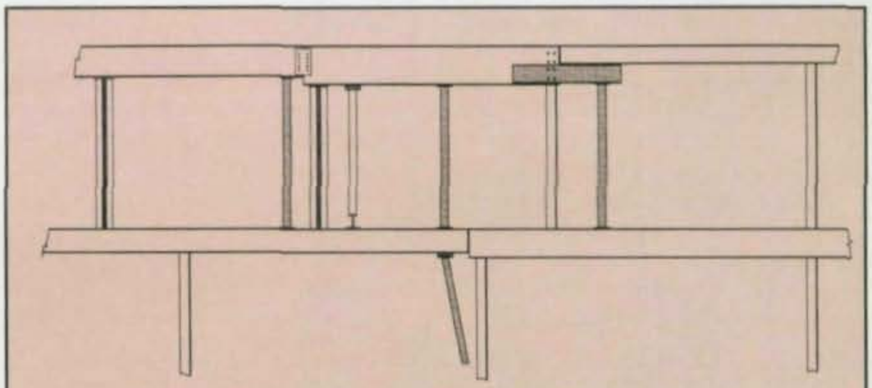
Our design had three phases. First, we made conservative assumptions about the possible load that was being carried in the columns to be transferred. We also made as many probes as we could in an effort to determine the actual connection configuration and determine as much as possible about the framing above and below.

Second, we made an initial test of our assumptions by installing strain gages on the existing columns. We then fluctuated the water level in the water tank on the roof while taking readings on the strain gages, which gave us confirmation of the distribution of the water tank load.

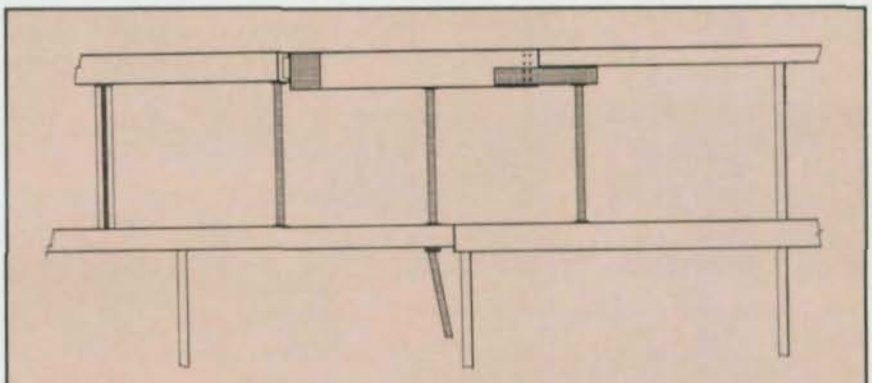
Lastly, we made a final verification of the load as we removed the columns. Initially, temporary shores were installed around the columns, with some of these shores later acting as the final tranferred columns. Strain gages were installed on the columns and shores, which then recorded the load released from the columns as they were cut loose from the structure and the load carried by the shoring members after the columns were removed. Having both of these numbers allowed us to cross check the gages and would give us the actual load in the columns for comparison with our assumed design load and give us a measure of added safety. If the actual load had not compared favorably with the design load, we could have re-attached the columns while we reconsidered our design.



1. Existing conditions



2. Shoring in place, prior to cutting columns



3. Final transferred condition

More Surprises

Not surprisingly, other problems were encountered. To start with, various non-structural work was going on in the apartment at the same time and construction workers from other trades were constantly knocking off or damaging the strain gages. Also, the existing columns were double angle and single angle columns, which complicated the interpretation of the strain gage results and also required the placement of extra gages. At the same time, however, the number

Column Number	Load in Column				
	Using Full Capacity of Element	Assumed Load Takedown			Actual Load From Gages
		Total Load	Live Load	Dead Load	
1	248 kips	206 kips	20 kips	186 kips	188 kips
2	187 kips	156 kips	17 kips	139 kips	126 kips

of gages that could be placed and monitored was limited since there was a fixed number of gages that could be monitored simultaneously by our equipment.

The placement of the columns also presented a slight problem. Since the new columns could not be placed at the center line of both the upper and lower beams, an eccentricity was introduced, perpendicular to the plane of the transfer, which was removed with kickers to adjacent beams. However, the location of these kickers had to be coordinated with the proposed ceiling coffers. In addition, one of the column relocations placed it over an adjacent beam, causing a reversal in the force through the existing riveted connection, as well as requiring a large increase in the capacity of the connection. This was finally resolved by adding a sloped column under the new location, which brought the load back to the original location at the base of the column below, without a need to modify the connection between beams. General contractor on the project was Clear Cut Construction, Inc., West Milford, NJ.

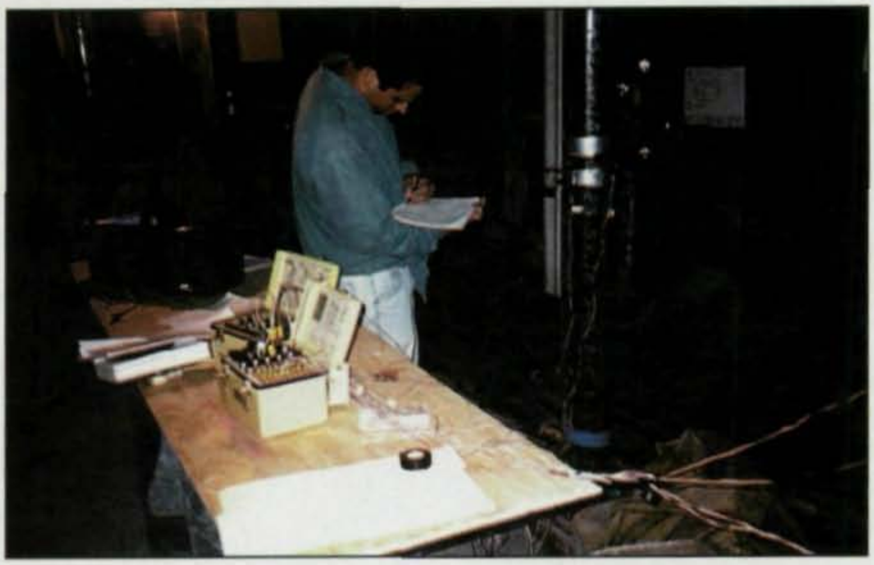
Of course, there were also positive side effects to working on a full renovation project. The major bonus was that since all the partitions were removed from the floor on which we were working, we were sure that the load was not being shed into adjacent partitions. Had partitions been in place, the reliability of the strain gage results would have been compromised.

Our first approach at a conservative number for design was to take the existing single and double angles and calculate their maximum capacity, using the code in effect at the time of their design. We assumed they were

The sizes of the existing steel beams were excessively large, which may have been the result of steel shortages as WWII began. The shortages might have resulted in the contractor using whatever larger sized members were available.

concentrically loaded, which we knew they were not, and came up with a conservative maximum load. This was the load we used in our design.

Our second check was an assumed load takedown for the building. This gave us a lower number, but still in a similar range of load. We then installed the strain gages and had the water level lowered in the roof tank. The results were encouraging, and we assumed that 25% of the water tank load would be carried on each column to be transferred. Consequently, we learned that the maximum load was on column 1 and it was carrying only 14% of the water tank load. We did not, however, change our design, but continued using our more conservative assumption. Whatever savings may have been had by reducing member sizes would have been insignificant compared to the cost of increasing the sizes after



Pictured at top is exposed existing column and beam, while shown above is the new column with strain gage monitors attached. Shown at left is the exterior of the vintage building.

they were in place.

Ultimately, after our testing during phase three, we found that our assumed load takedown was very close to the actual load.

Gary Steficek is a partner with the structural engineering firm Gilsanz Murray Steficek in New York City.

SOMETIMES A CON
CALLS FOR THE UN

It's not every day that bridge building techniques are used to expand a convention center. Yet, when the growth of Bartle Hall Convention Center in Kansas City was hemmed in on three sides by buildings and on the fourth by Interstate-670, there was but one way to go: over the highway.

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Please circle # 37

Bartle Hall Convention Center, Kansas City.

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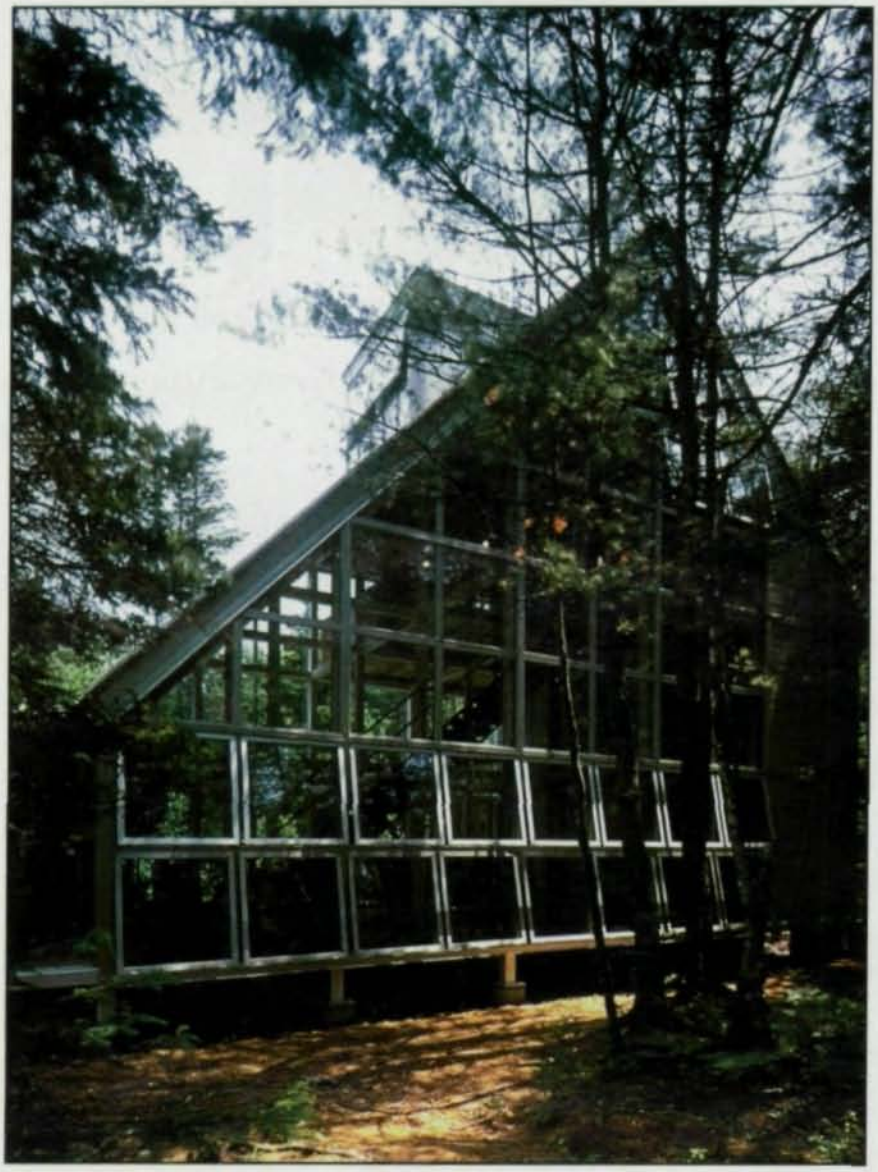


NATURAL SURROUNDINGS

A client's allergies necessitated an innovative design solution for a new home

WHILE A CLIENT'S REQUEST FOR A HOME THAT WOULD ONLY MINIMALLY INTERFERE with the surrounding natural beauty was not unusual, an additional requirement to minimize the use of petrochemical products created a unique design challenge.

The 2,200-sq.-ft. residence is located in a densely wooded hillside on Mt. Desert Island in Maine with view of the mountains of Acadia National Park. "The design concept was to interfere as little as possible with the natural surroundings, both by limiting the physical intervention and by the transparency of the building," stated Peter Forbes, FAIA, principal of Peter Forbes and Associates, Inc., an architectural firm with offices in Boston, New York City and Southwest Harbor, ME. The design also needed to accommodate the health requirements of the owner, a woman in her 60s who is afflicted by numerous allergies, primarily to petrochemical-based products. "Her allergies precluded using many common house construction materials," Forbes explained, "such as plywood (formaldehyde glues), wood windows (customarily impregnated with fungicides), most paints, stains, varnishes, polyethylene vapor barriers, foam insulation, etc."

INERT MATERIALS

Since she is allergic to neither glass nor steel, those two materials became the logical choice for constructed her new house. "She came to us in part because of the steel structured house we had designed on Deer Isle, ME—a house that won an AISC Award in 1987," Forbes said. "In the process of our research into materials, we discovered that the highly volatile paints used on steel 'out-gas' so quickly and completely that they did not present our client with the long-term health hazards that conventional house paints do." As a result, the "steel" theme was continued throughout the house in the stairs, balcony railings and elevator enclosure. "All of these elements could be fabricated off site, painted and then installed. The aesthetic consistency that this gave is quite wonderful." A zinc coating from Tnemec was ultimately used on the project.

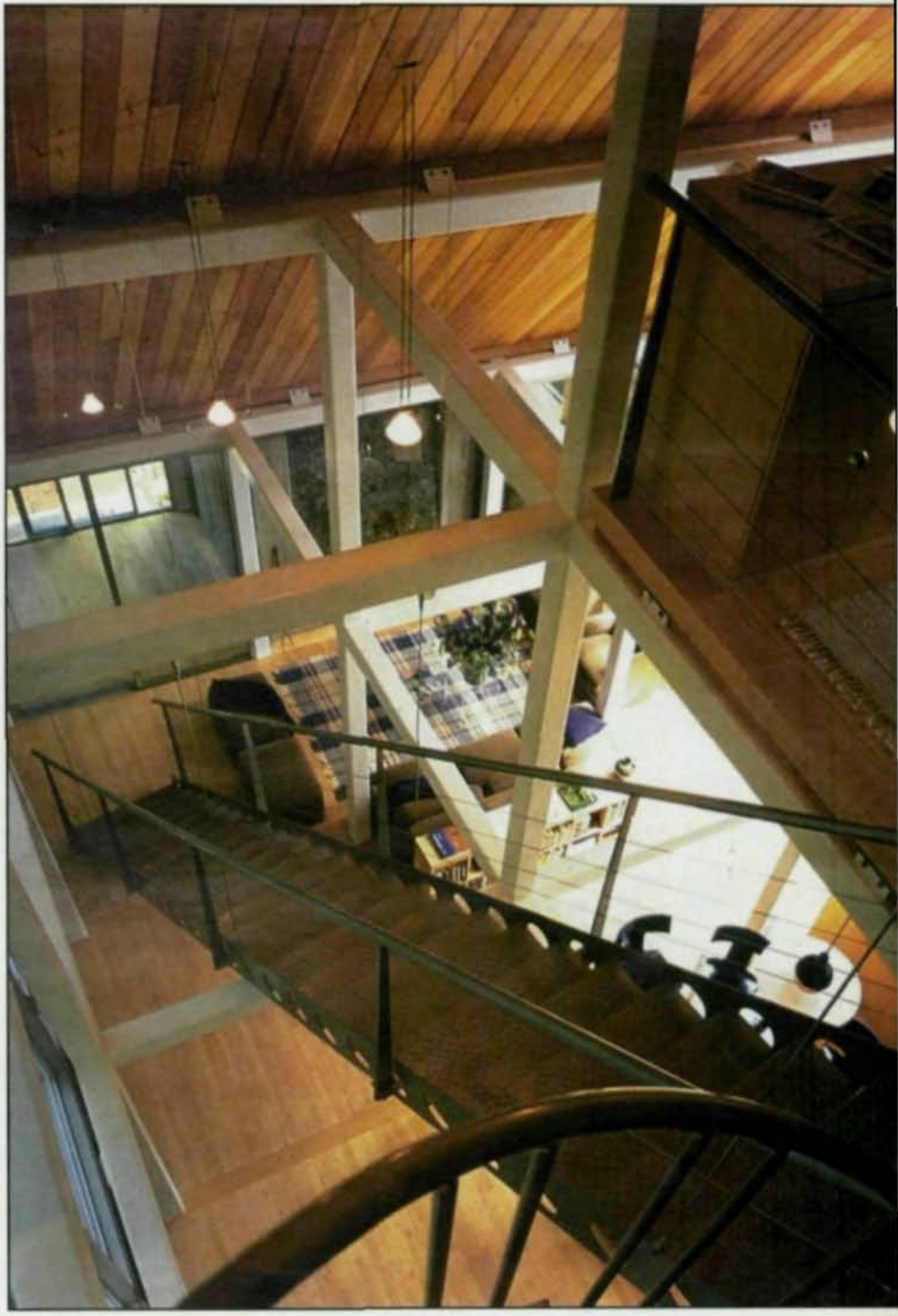
To achieve the best views the house had to be quite tall. Usually that requirement would conflict with the desire for a "transparent" structural system since a tall structure would require a larger-than-desired lateral bracing system. Instead, the home was constructed from a rigid welded frame of 6-in. steel tubes, 8-ft. on center both vertically and horizontally. "All of the other elements of the building—window wall, roof, floors, wall panels—clip to the three-dimensional steel grid," Forbes explained. "This had the additional advantage that we were able to construct rooms without walls, another requirement in a house where 100% air exchange is necessary to eliminate static polluted air."

3-D Symmetry

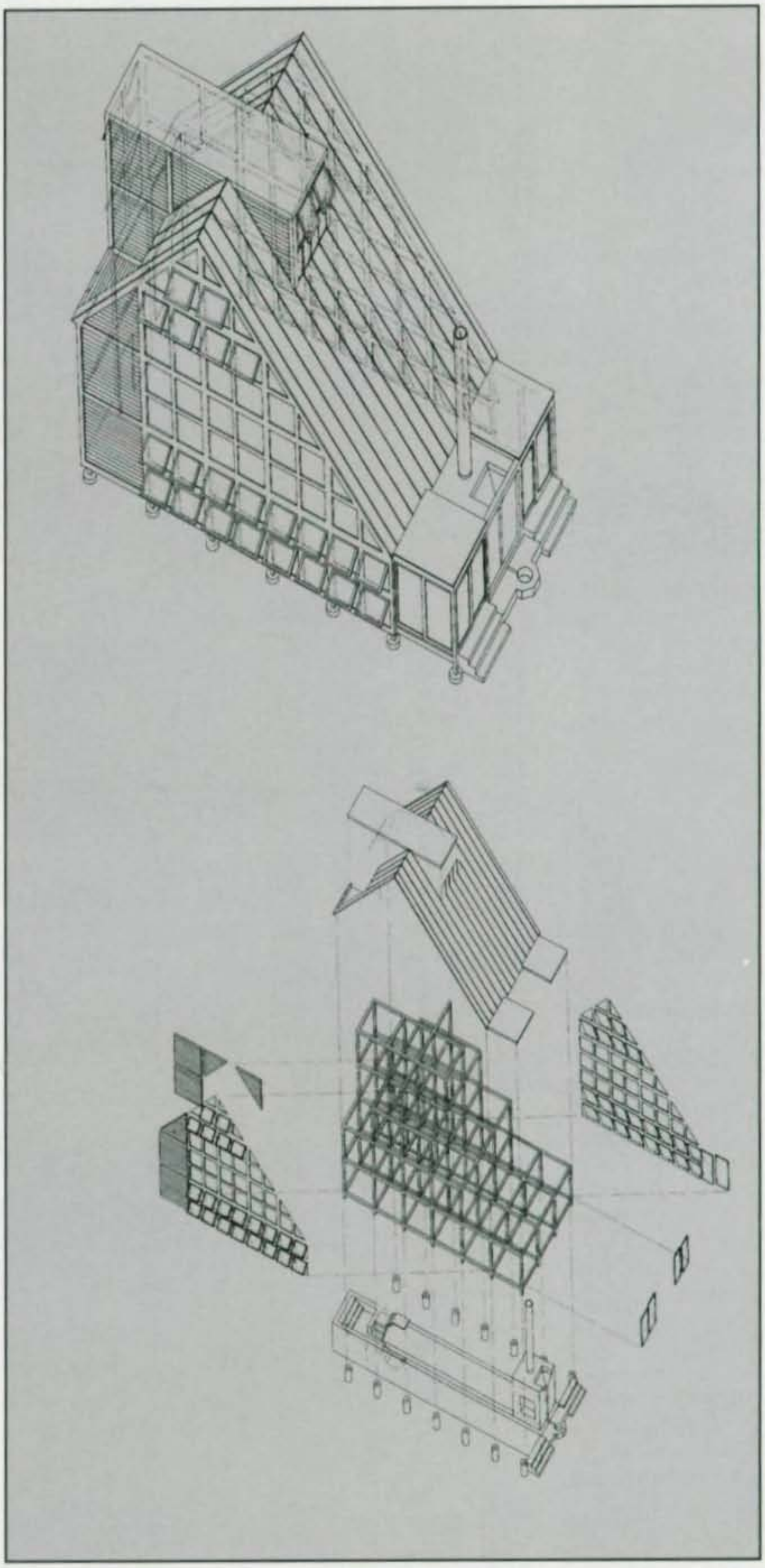
The steel grid created a 3-D symmetry about the structure, according to Michael Jollife, president of Zaldastani Associates, the project's Boston-based structural engineer. "Anything but a tube would destroy the symmetry. At any point you can turn a corner and be exposed to either a horizontal or a vertical plane."

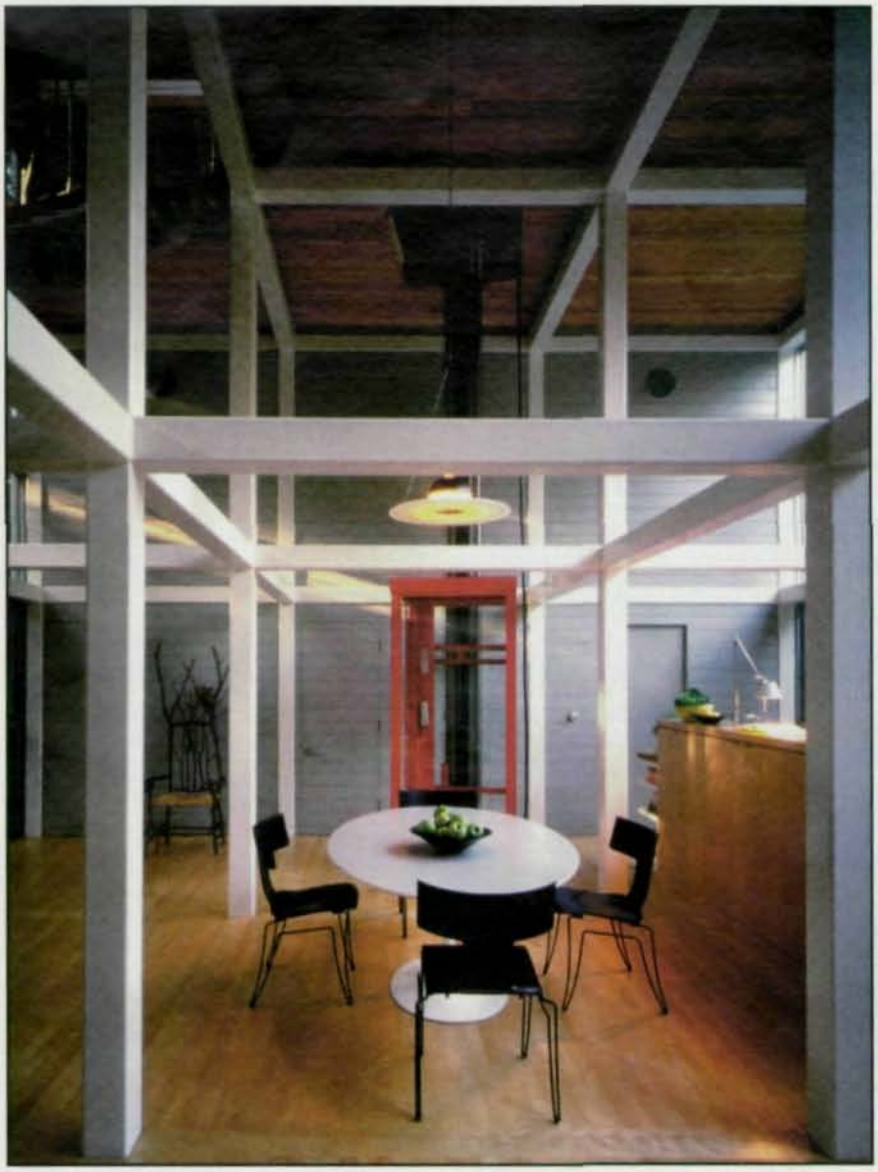
The effect is remarkable. In the spring, summer and fall, the occupants are literally surrounded by virtually uninterrupted nature. But perhaps even more wonderful is the winter scene, when you can stand inside the home and be enveloped in the falling snow.

Careful fabrication and erection was crucial to the success of the project, according to both Forbes and Jollife. Forbes required that the tubes be joined in such a manner that it was impossible to tell whether the



The small, symmetrical tubes create a wide open space. Photos (above, opposite and on second page following) by Paul Warchol





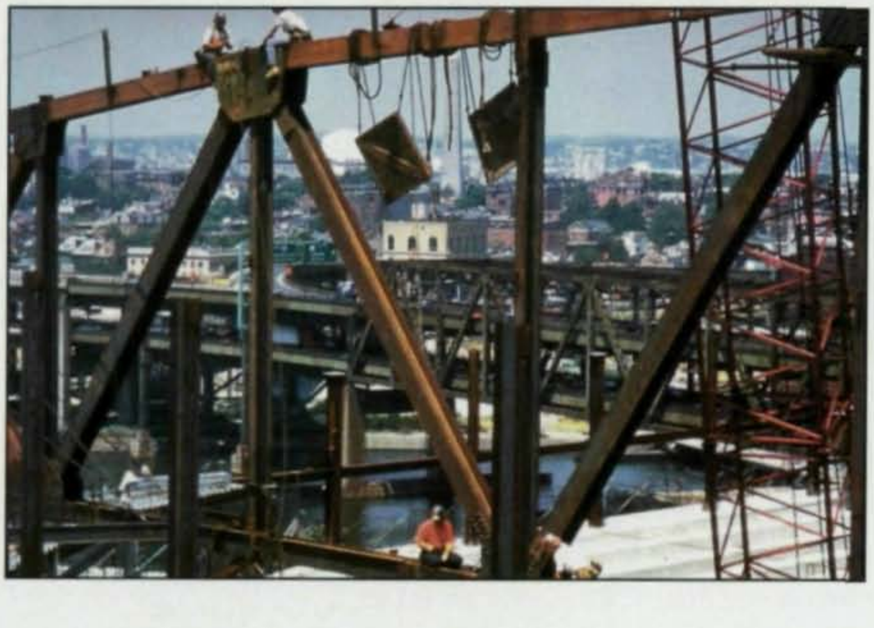
columns or the beams were continuous, a necessary requirement to maintain the symmetry of the structure.

"We checked and double-checked the drawings and worked to a much tighter tolerance—one-sixteenth of an inch—than we normally would," explained John C. Yohe, president of AISC-member Megquire & Jones, Inc. "We detailed the project in-house, surveyed each anchor bolt as it was placed, and every column was oriented individually on the drawings." Field erector on the project was Atlantic Builders.

Welding was also crucial. The tubes were prepared for full-penetration welds in the shop. The vertical pieces came in one piece from the fabricator and the horizontal members had clip angles attached. After welding, the clip angles were then removed. "All of the welds—flat, vertical and overhead—had to look identical," explained Rick Newman, president of Superior Welding & Fabrication. To achieve the uniform look, Newman used 7024 welding rods for the flat sides, 7018 rods for the overhead welds and the uncommon 7048 rod for the downhill welds. "Few people know about the 7048 rods, but they're terrific for downhill welds," he stated. The welding rods were supplied by Lincoln Electric.

TIGHT FIT FOR BOSTON GARDEN REPLACEMENT

The designers of a new home for the Celtics and Bruins faced numerous constraints, including an incredibly tight site and the necessity to build over an extensive network of railroad tracks



ONE BY ONE, FAMED—BUT DATED—SPORTS VENUES are coming down. And while fans may gripe at first, that reaction usually turns to praise. The new stadiums hold more fans while presenting better sightlines with greater amenities. Fleet Center, the new 20,000-seat home of the Boston Celtics and Bruins, should prove to be no exception when it opens its doors this month.

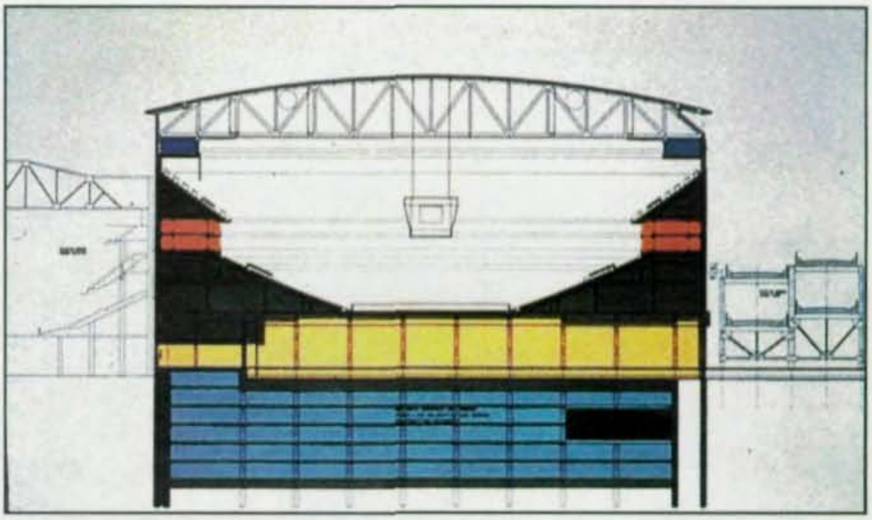
Inside, the design pays attention to those aspects of the old Garden that Boston fans loved most. "We were sensitive to the great love for the original Garden," explained Thom Greving, AIA, senior architectural designer with the Kansas City office of Ellerbe Becket. "It was a very intimate arena, and the shape and configuration of the new building is designed to bring the viewer as close as possible even in this much larger space. We approached this project the way we always do, by thinking about the user. Boston sports fans are there to see the sporting events, not other activities. So our design doesn't have additional entertainment activities. Instead, the design makes references to the teams. For example, the lighting reflects the team colors."

While the concept was straightforward, the execution was very difficult. To start with, space was very tight. Within inches on one side is the old

Boston Garden and only a few feet away on a second side is the Central Boston Artery, a large elevated roadway scheduled to eventually be replaced by an underground highway. The other sides are tightly hemmed in by existing buildings. Further complicating the design is the existence of a series of passenger railroad tracks running through the site.

According to Peter J. Cheever, P.E., vice president at the Boston-based structural engineering firm of LeMessurier Consultants, the new building is really three structures in one. Below grade is a 1,200-car parking garage (constructed using slurry walls and a top-down construction process) and the Orange subway line. On the ground level is a large room through which passes much of Boston's passenger train traffic. And finally, on the "table top" of that structure is the new Fleet Center.

The changing uses as the structure rises greatly complicated the structural design of the facility. While the columns for the ground level train facility are based on an orthogonal 40x30 grid dictated by existing train track layout, the stadium columns are radially arranged to suit the needs of an arena. "We arranged the layout of the arena columns according to the bowl configuration, sightline considerations, the number of seats and for ease of entry and exit," Cheever said. And while the underground garage normally would have been built with a regular grid of columns, the existing subway, which curves through two levels of the garage, disrupted some of the layout and necessitated several transfers. Nearly 200 caissons—each 5-ft. in diameter and attached to a W14x250 column—were used for the garage. The 90-ft.-high A572 Gr. 50 columns were delivered to the site in three sections each weighing 30 tons and welded on-site. The columns were designed with cap plates at the ground-



One of the difficulties in designing and constructing the new Fleet Center was working with the site constraints. The drawing at top shows the existing Boston Garden on the left and the Central Artery on the right. Pictured above is the Fleet Center under construction alongside the Central Artery. Shown at left is the close proximity of the new building to the old structure.

floor level to support and provide the connection for the arena columns, clip angles at each garage-floor level and shear studs welded to the bottom 10-ft.

General contractor on the project was the Boston office of Morse Diesel and steel erector was Dorel Steel, Quincy, IL.

The roof of the train area (the floor of the arena) consists of more than 100 built-up plate girders, which transfer the load from the arena columns above to the garage columns below. The arena columns also are W14s. Bolted to the columns are W18 and W24 raker beams, which support the precast concrete seating bowl. "We introduced as many columns as possible to minimize the depth of the beams, but we also had to consider the weight of the columns since the load had to be transferred," Cheever explained.

Another consideration in designing the column locations was the creation of a marshaling area to bring trucks onto the arena floor. "We actually went down to the Sun Coast Dome and used their large parking lots to lay out columns markings and then drove semi-trailer trucks through the arrangement to verify that the layout was practical," Cheever said. "It was expensive, but it was the only way to be certain there wouldn't be any problems in the future." Added Greving: "Only a handful of the columns actually extend the full length of the facility."

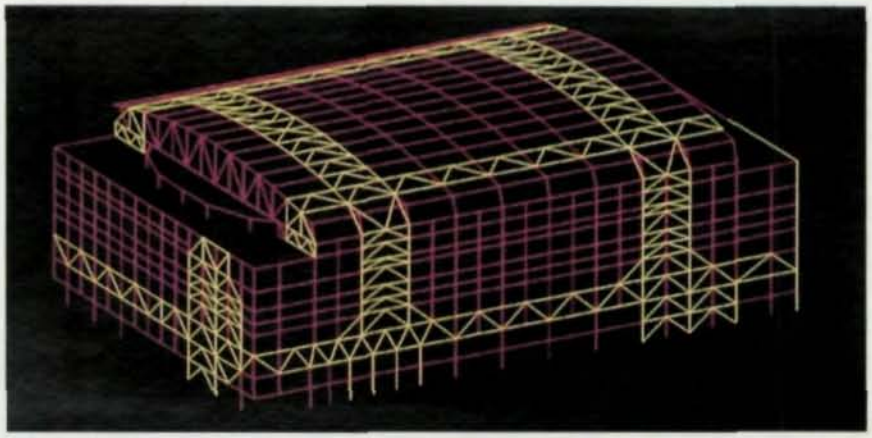
The arena is a perimeter braced frame with the load delivered to the bracing through the concrete floor diaphragm. Typically $^3/_4$-in. A325 bolts were used, though some 1-in. A490 high-strength bolts were specified. "We wanted to minimize field welding and the only moment connections are for some cantilevers for suites and some roof areas."

Vertical transportation was an important design consideration since the arena floor was 45-ft. above ground level and the main concourse was 60-ft. up.

"We used a lot of escalators and did a lot of signage studies," Greving said. "We conducted study after study to see how people could move most efficiently in this structure."

Roof Design

The Fleet Center has an overall plan of 300-ft. by 460-ft., with a convex curved roof with a high point 160-ft. above grade and eaves 18-ft. lower. "The roof shape is a low-rise barrel vault having a uniform radius of 634-ft.," Cheever said. Most of the roof is supported on 10 30-ft. deep segmented top chord trusses, 40-ft. on center, spanning 296 ft. Steel purlins spaced 10 to 10-3/4-ft. span between trusses and carry a 3-in. deep acoustical galvanized steel deck, insulation and fire proofing. "The bottom chord level of the main roof trusses is 96-ft. above the arena floor. Deep gutters are provided near the eaves at each side of the roof, and special measures are designed to prevent snow sliding, which would be very hazardous to motorists on an adjoining expressway and to pedestrians."



The roof design loads include dead loads of permanent construction, design live loads, special loads for the grid/catwalk level and roof snow load. A catwalk live load of 30 psf was used, plus allowances for lighting, speakers and spotlights. The press ring live load is 80 psf, including a partition allowance. An allowance of 3 psf was provided, as an overall hung load, to allow for banners, miscellaneous piping and ducts. A central scoreboard weighting 40,000 lbs. was assumed. The gird/catwalk level framing is designed to resist a hierarchy of possible loads from such sources as circus rigging and special concert or show hung lighting and speaker grids. For larger areas, a total hung live load of 20 psf was used. For individual beams, concentrated live loads were applied—up to 5,000 lbs., in a number of possible directions.

Snow loads, wind loads and seismic loads are all in accordance with Massachusetts Building Code. While earthquake loads did not control the design of the purlins or main trusses, some parts of the inplane roof bracing system, the grid/catwalk level lateral bracing, and the longitudinal bracing trusses were controlled by load combinations with earthquake. Most of the purlins were W18 and W21 rolled shapes, ranging in size from W18x46 to W21x68, with the heavier sizes used for the purlins supporting the hung catwalk loads.

The top chords of the main trusses are made up of straight segments following the roof curvature. Truss depth at each end is 14.2 ft. and spacing of the vertical web members is 21.2 ft. with some diagonal web members more than 36-ft. long. The top and bottom chords are rolled W-shapes oriented with the flanges in vertical planes, as are the web members.

Truss chord sizes vary from W14x99 to W14x370, while web members vary from W14x30 to W14x211. At the north end, the top chord must cantilever to support a wing-like architectural feature that extends the full length of the roof.

A prominent architectural feature is an 80-ft. tall mast near the edge of the roof, which was mandated by fire code that required an entry to the roof and also provides a needed overrun to accommodate a freight elevator. "The exterior of the structure makes no architectural reference to the old Garden," Greving said. "It was an odd building to design because we were designing for conditions that will exist in the future after the old Garden, two nearby buildings, and the Central Artery are taken down. From the exterior the roof is designed to make the building look smaller than it really is. In Boston, there are not too many large footprint buildings."

WHY A CHRISTMAS TREE?

Whether the custom began in Scandinavia or with Native Americans, topping out parties are today an important custom in the steel construction industry

HOW DID THE TOPPING OUT CEREMONY ORIGINATE? More than a dozen readers wrote responses to that question in *Modern Steel Construction's* August editorial.

One of the most detailed responses came from James A. Newman, fabrication division manager with AISC-member Art Iron, Inc., who sent an article that appeared in *The Ironworker* (December 1984) and an excerpt from which follows:

"No one seems to know exactly when or how it started, but the tradition of 'Topping Out' has become a cherished custom of Ironworkers whenever the skeleton of a bridge or building is completed. Topping Out is a signal that the uppermost steel member is going into place, that the structure has reached its height. As that final beam is hoisted, an evergreen tree or a flag or both are attached to it as it ascends.

"The nice thing about Topping Out is that no two ceremonies are exactly alike. For some, the evergreen symbolizes that the job went up without a loss of life, while for others it's a good luck charm for the future occupants. For some, the flag signals a structure built with federal funds, but for others it suggest patriotism or the American dream.

"We do know that as early as 621 B.C. the Romans celebrated the completion of the Pons Sublicus over the Tiber River—by throwing human beings into the water as sacrifices to the gods. In ancient China, the ridgepole of a new structure was smeared with chicken blood, as a substitute for human blood, in hopes of fooling the gods. It was widely believed that evil spirits may have occupied the structure, and that is why, through the Middle Ages, the local priest or rabbi had a special blessing for new homes, ships, churches and public buildings.

A topping out ceremony—with a tree AND a flag—as pictured on the cover of The Ironworker *in 1984.*

"By 700 A.D. in Scandinavia, the custom of hoisting an evergreen tree atop the ridgepole was a popular way of signaling the start of a completion party. The roots of this custom may also be mixed in with fertility symbols. Saplings, eggs, flowers and sheaves of corn are long-standing customs in European home building, presumably as a wish to the newlyweds for a productive and long life together. While the Teutonic tribes may have tried to appease the tree spirits for killing trees and using up that lumber, the Germans in the Black Forest seem to have invented the Christmas tree custom to celebrate the nativity of Jesus Christ, and hardly a structure goes up in Germany without an evergreen to signal the birth of a new building. The Swiss, also, lay claim to the custom of a fir tree to signal Topping Out.

"As iron and steel replaced timber as primary building materials, ironworkers naturally would carry on the custom of Topping Out. Strangely enough, none of the early photoengravings of ironworkers show the evergreen in Topping Out ceremonies. Perhaps, due to the exceedingly high fatality rates, such a symbol would not be appropriate.

"When the last strands of cable were laid for the Brooklyn Bridge a hundred years ago, the wheel operated by the ironworkers was decorated with American flags. By 1920, ironworkers were again draping their work with American flags, this time while driving the first rivet on the Bank of Italy in San Francisco. By the end of the decade, the tradition of flags in Topping Out was fully established.

"Why an American flag? Probably because the so-called "American Plan" launched in 1919 did not include unions. In fact, the single largest potential employer of ironworkers, Elbert Gary, chairman of U.S. Steel, contended: 'The existence and conduct of labor unions, in this country at least, are inimical to the best interests of the employees, the employers and the general public.' The American Plan—promising the destruction of unions, starvation wages, deadly hours, hopeless safety conditions and the dreaded 'yellow-dog contract' swearing never to joint a union—suggested that unions were somehow un-American during the post-war Red Scare. Thus, the American flag became a natural symbol to protest the American Plan and to demonstrate the ironworker's loyalty to flag and country.

"The two traditions of flag and evergreen converged only a couple of decades ago, perhaps to balance out the final beam."

Going back another decade, *The Ironworker* reported the following in December 1974 issue:

"The symbol is rooted in an old Scandinavian custom. The Norsemen venerated the evergreens—cedars, spruces and pines. The trees were plentiful throughout the frozen reaches of northern Europe and thus provided building materials and firewood for the inhabitants of those wintry regions. In addition, the evergreens retained their color throughout the years and provided welcome relief from the dull hues cast by snow and ice.

"Those hardy Vikings challenged the seas of Europe and the New World in long ships of seasoned spruce, with tall masts carved from towering pines and steering oars of cedar. Returning from a particularly successful raid on hapless southern neighbors, Viking chieftains often constructed huge homes—called mead halls. Upon completion, these chieftains hoisted an evergreen tree to the ridge pole in celebration. So, when the topping out beam rises aloft with its customary symbols, the flag and the tree, it offers a link with history."

Persian Origins On Bridges

Scott A. Bustrum, field operations manager with AISC-member Junior Steel Co. provided information from his company records that he says originally came from Bethlehem Steel. In addition to talking about early Chinese and Roman customs, his data adds: "Bridges posed special problems and goaded the fears and superstitions of the ancients. Xerxes, the famed Persian military leader, blamed recalcitrant river gods for the collapse of a pontoon bridge over the Hellespont. To punish and shackle these gods, the water was given 300 lashes and a pair of manacles was thrown into the river."

Concerning the Scandinavian roots of the topping out ceremony, Junior Steel's information included that "In later times in these same Scandinavian countries, and also in the Black Forest, it was customary to fasten a sheaf of corn to the gable. The corn was believed to serve as food for Woden's [the chief Norse god] horse and as a charm against lighting. In more recent times, garlands of flowers or sheaves of corn were duplicated in wood, stone or terra-cotta on Gothic buildings. Such agrarian decoration is perhaps a survival of the ancient custom."

Many others wrote in with similar answers. Curt Zeigler of Stewart-Amos Steel, Inc. and Ron Montes of Bay Drafting Service, both cited *Why Do Clocks Run Clockwise? And Other Imponderables*, a wonderful book by David Feldman, which contains essentially the same explanations presented in *The Ironworker*. Three readers, Adam S. Bangs, P.E., of Hudson Engineering Corporation in Houston, Erol J. Aydar, P.E., of Hankins Anderson, Inc., in Richmond, VA, and Eric Bjorklund of Freese-Nichols in Fort Worth, TX, referenced Jack C. McCormick's "Structural Steel Design", which states:

"The Christmas tree is an old North European custom used to ward off evil spirits. It is also used today to show that the steel frame was erected with no lost time accidents to personnel."

Thomas C. Shaeffer, P.E., of Stanley D. Lindsey & Associates, Inc., in Nashville quoted from *Reader's Digest*, which, in turn, was quoting from the book *Ever Wonder Why?*:

"In ancient times, people would attach plants thought to be inhabited by good spirits to the top of their new structures. Builders still observe this superstition in a custom called topping out of the new building."

Kim Stanfill-McMillan, P.E., with the USDA Forest Service, wrote: "The Christmas tree atop the last beam is an old timber-framer's tradition (sorry). Here is a quote from Tedd Benson's book entitled *Building the Timber Frame House—The Revival of a Forgotten Craft*: 'To signify a safe and successful raising, to pay respect to the wood that has given life to the frame, a traditional pine bough is attached to the peak of the building. Some of the old timers mark this occasion further by breaking a bottle of rum at the ridge and delivering a few lines of verse composed for the occasion.'

"Usually a dance is held on the floor after the frame is raised, a tradition that also continues to this day. Steel erectors and others have borrowed the tradition of a pine bough, but since the scale of these buildings is often larger, the pine bough has evolved into a Christmas tree, which is more readily seen."

Renaissance Roots

A variation on the ancient theme was submitted by Sheila Shaw, marketing director with Bread Loaf Construction, a design/build firm in Middlebury, VT. "The first known ceremony with the use of a tree was in the Third Dynasty, about 2700 B.C., in Egypt. This first appeared when the first stone building of Egypt, the Step Pyramid of King Zoer at Sakkara, was completed. The slaves placed a live plant on the top of the Pyramid for those slaves who had died during the construction so they too might have an eternal life.

"It later appeared in the early Renaissance Era, during the period of the Gothic Cathedrals. An evergreen tree was placed on the highest point to signify the completion of the building. A large festival, lasting sometimes for weeks, was held in the town for this honor. From the Italian Renaissance, it was carried through the countries of France, England, and Spain, as they, too fell into the Renaissance Era."

Bread Loaf's account then adds information about Scandinavian and German traditions.

Gordon Wright, an editor at Building Design & Construction magazine, sent along a copy of an article from Morse/Diesel's newsletter, which printed the history of topping out as presented by Scioto Erectors Inc. of Columbus, OH:

"Scandinavian mythology suggests that man originated from a tree and that the soul of man returned to the trees after death, giving each tree a spirit of its own. Man began constructing his shelter with wood. Before cutting a tree, he would formally address the forest, reminding it of the consideration he had always shown toward the trees and asking the forest to grant use of a tree for construction of his home. When the house was complete, the topmost leafy branch of the tree would be set atop the roof so that the tree spirit would not be rendered homeless. The gesture was supposed to convince the tree spirit of the sincere appreciation of those building the home.

"As time passed, the early conception of tree worship gradually changed. The individual tree spirits merged into a single forest god who could pass freely from tree to tree. Trees were no longer placed atop the home to appease the spirits, but rather to enlist the blessings of the forest god. The tree branches on top of the home insured fertility of the land and the home. Gradually, ribbon, colored paper, painted eggs and flowers were added to the tree as a symbols of life and fertility.

"The custom of placing a tree on a completed structure came with immigrants to the United States and became an integral part of American culture in barn raisings and house warmings."

Carpenter's Tradition

A similar explanation was presented by Frank Lundy, P.E., of Lundy Construction in Williamsport, PA, who explained that "This tradition may spring from the Carpenter's tradition of nailing a free tree branch to the ridge (rafter) board to entice the "wood spirits" to bestow good fortune on a house. If you look in the attic of older houses, you may find such a feature." Blair Hanuschak of Walter P. Moore and Associates in Atlanta sent along a similar explanation from the program given out during the topping out ceremony of the Florida Aquarium. And Robert J. Susz, a design engineer with Garrett Engineering in Geneseo, NY, gave much the same explanation but added that "I believe the flag was first used when steel framing became popular. It was in dedication to good old U.S. made steel beams. The signing of the last beam or girder by the laborers has similar traditional roots."

Native American Origins

Some people offered different interpretations, however.

Barry P. Chepren, E.I.T., of Oquossoc, ME, wrote:

"At my first topping out party for a 10-story building in Tampa, FL, I asked the same question when a large pine tree was hoisted to the top of the building. The answer that I was given was that the tradition originated around the time when high-rise construction became necessary in most major cities. During this time, many of the contractors employed many American Indians on their construction crews. According to my source, American Indians believed that no man-made structure should be taller than a tree. This belief became enough of an issue at the time to prompt someone to place a tree at the top of a topped-out building. This practice caught on and is still performed today at most high-rise building projects.

"During the Vietnam War, many people perceived construction workers as unconditional supporters of U.S. government policies in Southeast Asia or "hawks" as they were called. This impression was made popular when the news media broadcast footage of clashes between war protesters and construction workers during a rally in New York City. Many construction workers as well as police officers began to wear the American flag on their hard hats and uniforms to show support for American soldiers in Vietnam. It is around this time I am told that American flags became popular at topping-out events.

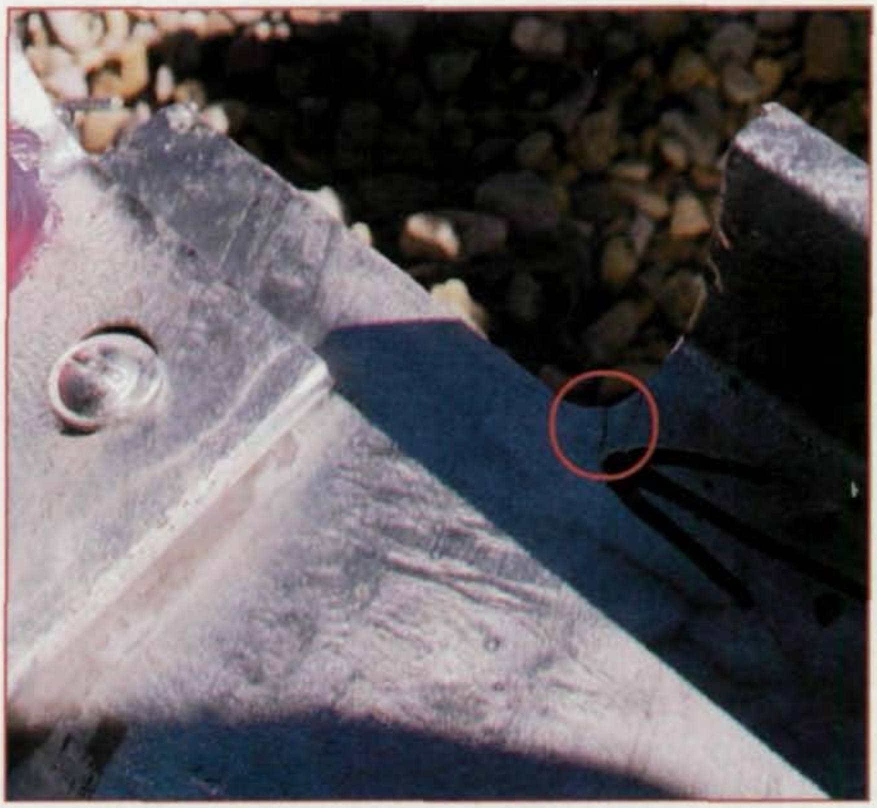
Several other writers supported the Native American origins of the topping-out ceremony. The final word, however, may be a novel interpretation from Harvey G. Johnson at Bittner Engineering, Inc. in Escanaba, MI:

"During World War II it was a custom for a submarine returning from a mission with all of its torpedoes used to tie a broom to the periscope to signify a "clean sweep" or completion. How, or if, this ever translated to the tree/flag, I have no idea."

COPE CRACKING IN STRUCTURAL STEEL AFTER GALVANIZING

There have been isolated reports of cracks in the flame cut radius of copes of galvanized steel

By Thomas J. Langill, Ph. D. and Tom Schlafly



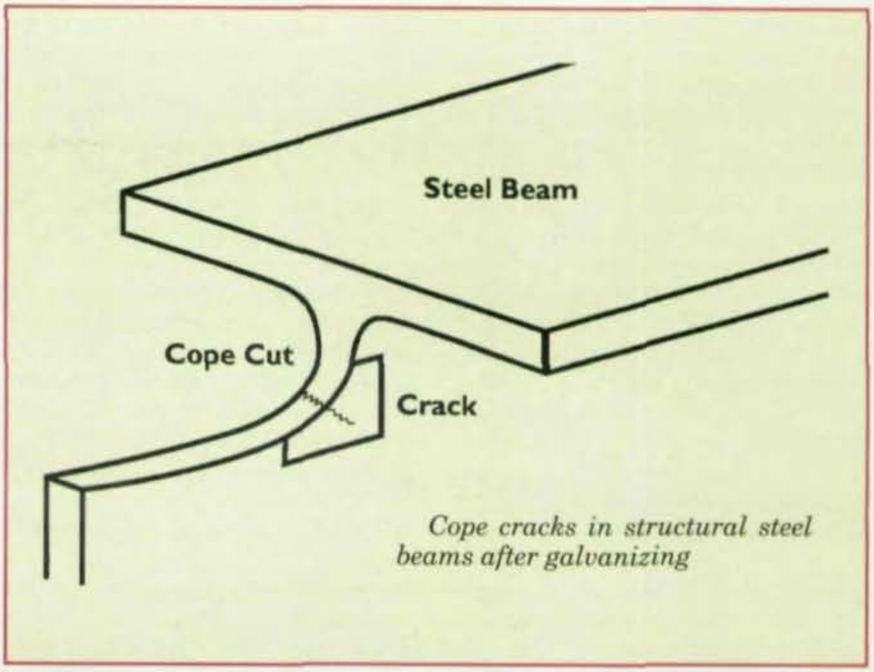
Pictured above is an example of cracking in an oxygen cut cope in a structural steel beam.

HOT-DIP GALVANIZING, THE PROCESS OF DIPPING STEEL PARTS IN MOLTEN ZINC has been successfully used for over 140 years as a corrosion inhibitor for steel structures. As the zinc coating weathers, it forms a very stable protective film on the surface of the material. This film provides many years of corrosion protection under all but the most severe environments. An attractive feature of galvanizing is the property of protecting small areas of exposed steel. Zinc acts as an anode in a galvanic couple between the zinc and the steel. The zinc is consumed before the steel rusts. During the galvanizing process, the zinc and iron form a metallurgical bond, a series of intermetallics. The intermetallics provide good abrasion resistance and a high adhesion strength of the coating to the substrate.

Since 1991 there have been isolated reports of cracks found in galvanized structural steel wide-flange sections. Many of these reports are not well documented. Commonly, the problem is found and repaired before it is reported and investigated. Repairs destroy the evidence and indications that could be used to determine the physical cause for the cracks. There have been common threads through most of the reports. The cracks are commonly found in the flame cut radius of copes and extend radially away from the copes at 45 degrees. They are visible on both sides of the web indicating they extend through the material. The cracks have appeared in A36, A572 and probably in A588. Cracks have been found in some material from a given heat and not in other material from the same heat. There have been reports of cracking in material made by the blast furnace method as well as the currently prevalent electric arc furnace method. Fabricators and engineers who have witnessed the problem report cracks are more prevalent in sections around 21 inches deep but they have been found in material as small as 10 inches. Members without flame-cut copes show no signs of cracking. Reports to date indicate that cracks can be identified by visual examination. Visual inspection of copes prior to galvanizing has not revealed any 'pre-galvanized' conditions leading to cracking. At least one fabricator alleges a difference in the incidence of cracking between galvanizing companies.

There are a number of areas to investigate in searching for probable causes for cracking and developing appropriate procedures to minimize cracking problems.

- The galvanizing temperatures are below the critical transition temperature for steel, but variations in cooling rate and the presence of molten zinc result in metallurgical reactions effecting steel properties and changes in residual stresses.
- The flame cutting causes high tensile residual stresses (at or near yield point) along the flame cut surface of the cope. The galvanizing temperature can create stresses at a lower level, but throughout the member as evidenced by warping in some sections.
- The reaction of steel to the presence of liquid metal can cause an effect called liquid metal embrittlement. Galvanizer's pickling baths and handling techniques can vary to some degree.
- ASTM A143-74 Standard Practice for Safeguarding Against Embrittlement of Hot-Dip Galvanized Structural Steel Products and Procedure for Detecting Embrittlement (ASTM VOL 1.06) states that the duration of pickling can effect embrittlement. The standard further states that heating to 300 deg F. between pickling and galvanizing in most cases results in expulsion of the hydrogen absorbed during pickling. Variation in galvanizer practices such as bath temperature, quenching, bath additions and pickling time have not been found to have a measurable effect on probability of crack occurrence according to the American Galvanizers Association.



Cope cracks in structural steel beams after galvanizing

- The processes of steel making have changed significantly over the last twenty years with the possibility of varying amounts of trace elements and different mechanical properties across sections than those which were typical with past practice.
- The mechanical properties of the steel including the lower toughness in the area of the fillet between the flange and the web can also be effected by differences between cooling rates in the different thicknesses of material, segregation, and less grain refinement due to mechanical work by the mill rolls. Any of these factors could contribute to the problem.

Because of a number of instances of cracks that emanated from weld access openings of full penetration welded splices of jumbo sections subject to applied tension in the early 1980s, a special task committee of the AISC Committee on Specifications was appointed to study the problem and develop recommendations for specification provisions to minimize the potential for such occurrences. A review of the investigations which had been conducted on the reported incidents and study of related research clearly revealed that the area of the web-to-flange junction of heavy rolled shapes were often characterized by large grain structure of low toughness, and further, that the flame cutting operation resulted in a very thin layer of brittle untempered martensite on the flame cut surface. It was further noted that magnetic particle or dye penetrant inspection of the cut surfaces revealed the existence of microscopic cracks in the brittle surface material which could not be detected by unaided visual examination. These observations were confirmed by research by Kennon and Kohler, Australian Welding Research, 1979 and Alexander Wilson, AISI 1987 which showed that flame cutting a variety of structural steels including A36, A572 and A588, resulted in microcracks along the edges of cut surfaces. The AISC Specification task group therefore recommended that weld access openings for full penetration welded splices of ASTM group 4 and 5 shapes (jumbos) which were to be subject to applied tension should be subject to supplementary notch toughness testing requirements, preheating before flame cutting, grinding to bright metal and dye penetrant or magnetic particle examination prior to welding(Ref. ASD 9th ed., J1.8 & M2.2; LRFD 2nd ed., J1.6 & M2.2). Similar provisions have been incorporated in the AWS Structural welding Code—D1.1. Preheating is intended to reduce the thickness of the brittle martensite layer, and the grinding is intended to remove the brittle layer and roughness of the flame cut surface and to facilitate magnetic or dye penetrant examination. The sections exhibiting problems after galvanizing are not necessarily group 4 or 5 shapes, but the same mechanisms which are responsible for cracking in jumbo shapes may also be initiating micro-cracks in the copes of smaller beams. If such beams are then subject to liquid metal embrittlement and the thermal cycles of the galvanizing process, the pre-existing invisible micro-cracks may be driven to visible size. Grinding of flame cut copes and magnetic particle or dye penetrant examination prior to galvanizing may be warranted as one possible precaution.

Ongoing research which started in the galvanizer industry and is now sponsored by the International Lead/Zinc Research Organization (ILZRO) at Metals Technology Laboratories of the Canada Centre for Minerals and Energy Technology (MTL/CANMET) is examining some factors that could contribute to the formation of this cope cracking. This research indicates that room temperature yield strength has an effect on the propensity to crack and combines with high residual stresses and liquid zinc reactions on steel to make the micro-cracks unstable until they propagate out of the tensile residual stress region. The effect of liquid zinc on steel and the reaction called 'liquid metal embrittlement' are being reviewed. Photomicrographs of cracked surfaces show zinc on the fractured surface indicating the crack occurs before the piece is removed from the kettle. Whether the zinc enters micro-cracks before they propagate or after while the piece is still submerged has yet to be determined. In studying the effects of the galvanizing temperature it was found that cracks did not form when samples were cycled through process temperatures without actually being immersed in liquid zinc. Trace elements from the steel have been found in concentrated levels at the grain boundaries near fractured surfaces. Elements such as copper, tin and nickel have been identified. To date there is no conclusion available linking these elements to cracking. The surfaces of the cracked section indicate a brittle behavior which

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is consistent with the theory of a martensitic layer and residual stresses.

Some fabricators have elected to grind the area of the flame cut section near the re-entrant corner and report a significant reduction in the incidence of cracking but not a total elimination.

ILZRO will continue to fund work to determine the relationship between microstructure, stress and other factors such as steel chemistry. Remedial measures will also be explored. AISC will recommend to ILZRO that they explore the effectiveness of using a combination of pre-heat, grinding and magnetic particle or dye penetrant inspection to avoid cracks. We will report results of this exploration when it becomes available.

Near Term Practices

Unaided visual inspection of flame cut surfaces may not be effective in detecting microcracks. Until ongoing research determines more appropriate criteria, it would be prudent for flame cut surfaces of copes of structural members destined to be galvanized to be ground to bright metal and the surface examined by the dye penetrant or magnetic particle methods. After galvanizing, the area of the copes should again be visually inspected to assure that cracks are not left in the final structure.

Another method that may be effective in minimizing the potential for cracking problems is to pre-drill (not punch) the beam to form the radius portion of the cope as shown in ASD Commentary fig. C-J1.2 pg. 5-162 or LRFD Commentary Fig. C-J1.2, pg. 6-218.

The American Galvanizers Association has reported an alternative to avoid cracking that involves placing a bead of weld along the edge of the re-entrant corner on both side of the web. The weld is extended one inch in either direction from the corner of the cope. It is pointed out that a low silicon filler metal considered compatible for galvanizing is used. No cracks have been found in copes treated this way. While this may be effective, it appears to be an expensive solution.

The galvanizing can be repaired using methods described in ASTM A780, Standard Practice for repair of damaged Hot-Dip Galvanized coatings.

Thomas J. Langill, Ph.D., is techncial director with the American Galvanizers Association in Aurora, CO. Tom Schlafly is director of fabricating operations and standards at AISC.

FINITE ELEMENT ANALYSIS AND TESTS OF BEAM-TO-COLUMN CONNECTIONS

On-going seismic research is using finite element models to determine why some steel moment connections failed

By Ralph M. Richard, Ph.D., James E. Partridge, P.E., Jay Allen, S.E., and Skip Radau

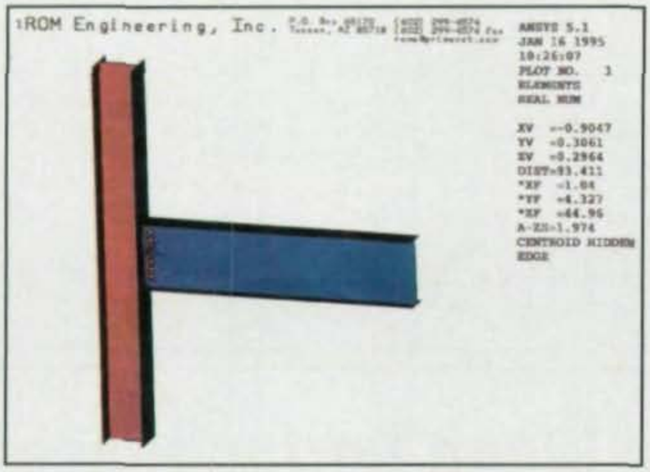

Fig. 1: W14x176 column & W27x94 beam

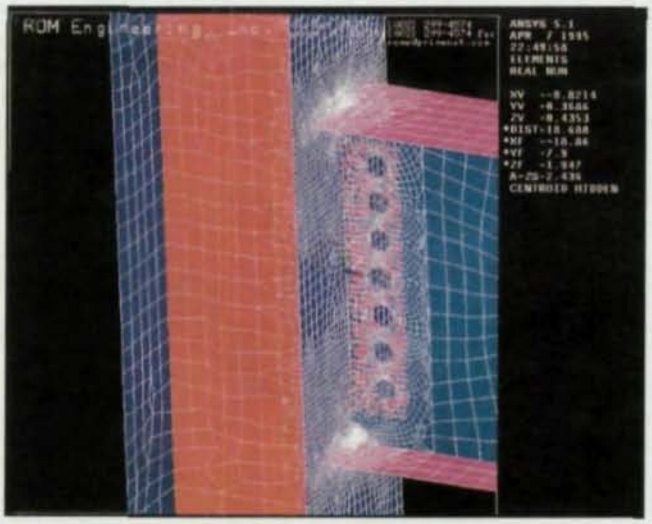

Fig. 2: Finite element connection model

THE JANUARY 17, 1994, NORTHRIDGE EARTHQUAKE UNEXPECTEDLY FRACTURED a large number of steel moment frame connections that were specifically designed to respond in a ductile manner under seismic loading.

These joint fractures in the popular bolted web-welded flange moment connections included weld failures, beam flange fractures at the weld access hole, and divots of steel pulled from the face of the column. These failures, which occurred in connections with and without column stiffeners, were almost always in the proximity of the bottom flange, and indicate that the presence of the floor slab significantly strengthened the connection at the top flange. Moreover, case studies in the AISC Earthquake Report indicate that even if the welds are properly made, stresses at and in the welds appear to be quite high.

To evaluate the stress and force distributions in these connections and provide improved designs and retrofit procedures, extensive analytical and experimental studies were made. Finite element analyses were used to design full scale static and dynamic tests that were recently conducted at the Smith-Emery Company laboratory in Los Angeles on a connection identical to the type that fractured (bottom flange weld) in the Northridge earthquake. This test connection assembly comprised a 10-ft. W27x94 beam with the flanges welded to a 14-ft. W14x176 column as shown in Fig. 1. A $^1/_2$-in. shear tab with seven $^7/_8$-in. A325 fully tightened bolts was the web connection. The weldments were identical to the field weld material, which was NS-3M-0.120-in. Three Charpy V-notch specimens of this weld tested at low values of 4, 5, and 7 ft.-lbs. and had an average tensile yield strength of approximately 63,000 psi.

The assembly was "pin" supported at the top and bottom of the column and loaded with a concentrated load at 9.5 ft. from the column flange.

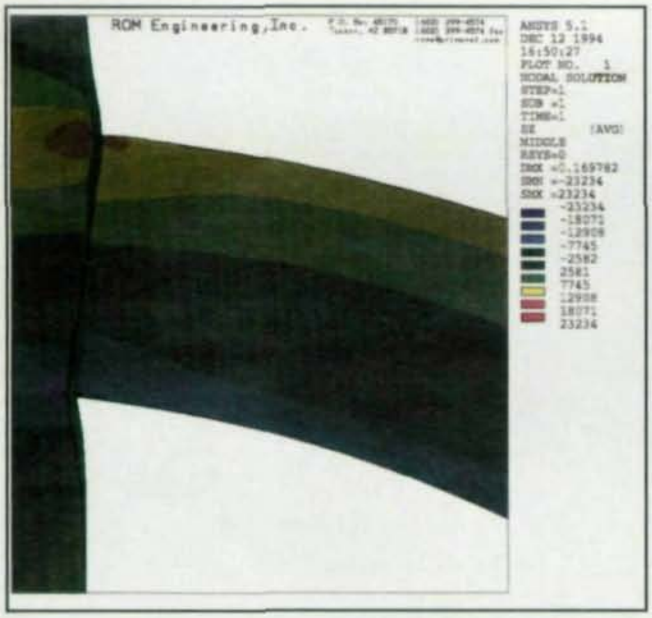

Fig. 3: Fully welded connection SYMM abt column centerline

Fig. 5: Top flange bottom surface longitudinal stress contours; nominal flange stress = 11,000 psi

Fig. 4: Top flange top surface longitudinal stress countours; nominal flange stress = 11,000 psi

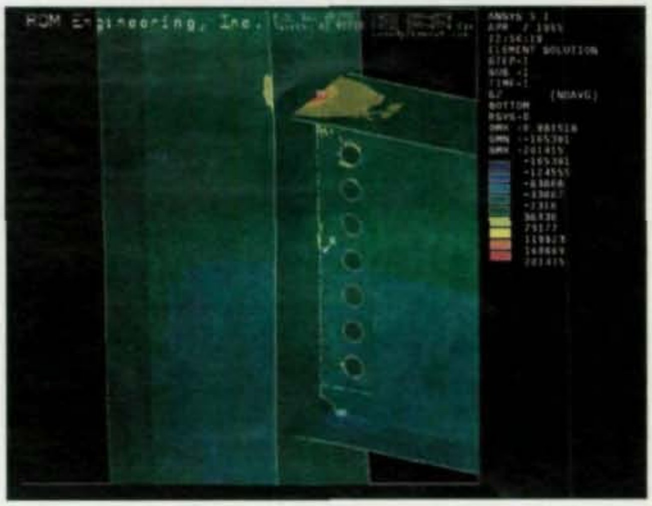

Fig. 6: Stress contours Web-bolted flange-welded connection

Shown in Fig. 2 is close-up view of one of the finite element meshes used in the analyses. Several mesh sizes were studied to ensure that the high stress and strain gradients in the weldment were properly obtained. This high fidelity model consisted of about 30,000 four node plate bending elements having about 40,000 degrees of freedom. Solid element sub-models used to evaluate internal stress and strain distributions in the beam and column flanges and welds comprised 80,000 elements and 100,000 degrees of freedom.

Shown in Figs. 3 through 5 are the results of this connection study with both the column flanges and web fully welded to the column flange. To clearly demonstrate local bending effects and stress distributions the beam was subjected to a pure moment that caused a maximum nominal stress at the weldment of 11,000 psi. The deflections due to this loading shown in Fig. 3 are magnified 250 times. Note the flexing of the column flanges of this connection which according to current AISC specifications does not require continuity plates. The stress concentration factor in the column web near the weld is approximately 2. Shown in Figs. 4 and 5 are the longitudinal beam flange stress contours. In Fig 4 the flexural stress contours on the top surface of the beam flange are shown. At the weld line these stresses vary from 1,244 psi at the flange tips to 41,405 psi at the center resulting in a stress concentration factor of

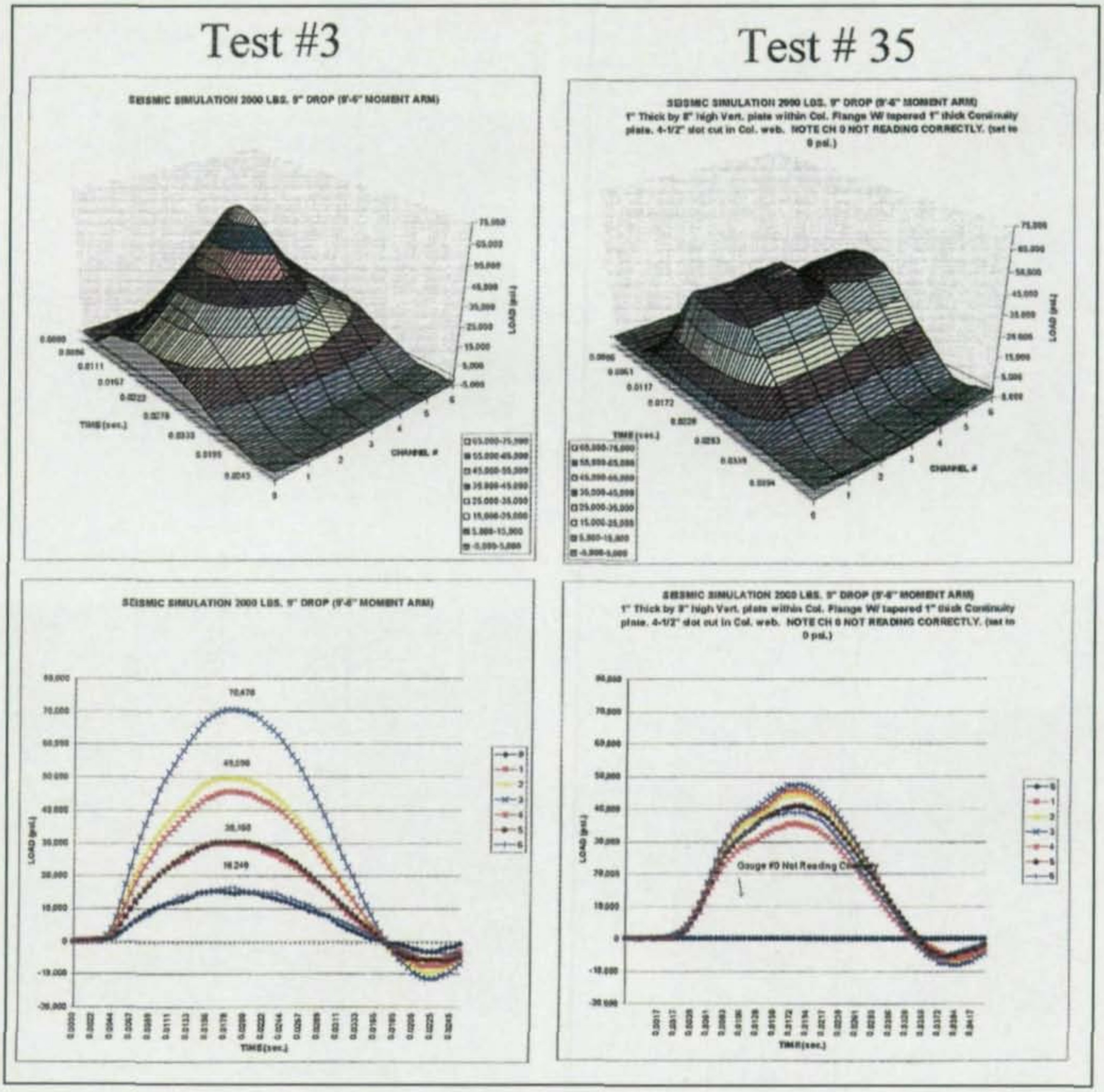

Fig. 7: Experimental top flange stress distributions

about 4. In Fig. 5 are the longitudinal stress contours for the bottom of the top flange where the maximum stress occurs at the edge of the weld access hole. Note that when the loading on the beam is reversed the stress "hot spot" in tension is at the weld access hole on the bottom flange where it is the most difficult to weld. Shown in Fig. 6 are stress contours for the web bolted connection with the beam subjected to 100 kip end load. This results in a nominal beam flexural stress at the column flange of 44,400 psi whereas the peak stress at the column flange is 201,000 psi which gives a stress concentration factor of 4.5. The flange stress at the weld access hole is 165,000 psi. These contours show that the high flexural stresses in beam top flange, which is 10 inches wide, are very local and that the nominal beam flexural stresses exist only several inches from the column flange. The stress concentration factor of 4.5 indicates that the weld must be ductile and of uniform quality to accommodate the high strains in the proximity of these high stresses. These analyses also show that the shear tab picks up about 22% of the connection moment.

Shown in Fig. 7 are plots of the experimentally obtained top flange stresses vs. time for a 2000 lbs.

drop test of 9 in. This dynamic loading resulted in generating strain rates ranging from approximately 1.00 in./in.-sec. at the center of the beam flange to 0.20 in./in.-sec. at the edge near the flange welds. These high strain rates are representative of the effect of impact loads in regions in structures with high strain gradients. These experimental results, which were obtained from the top flange strain gages located at 0.55"-in. from the column flange as shown in Fig. 8, confirm the large stress gradients across the beam flange at the face of the column as predicted by the finite element models. In Fig. 7 the Test #3 results are for the connection without any column reinforcement whereas the Test #35 results are for the column reinforced with both horizontal and vertical stiffeners and a slotted column web opposite the beam flanges as shown in Fig. 8. With the column modifications, the peak stress at the strain gages has dropped from 70,000 psi to 48,000 psi and the stresses are nearly uniform across the beam flange at the strain gage locations. By reinforcing the column flanges with both vertical and horizontal stiffeners and softening the column-beam flange interface with the column web slot opposite the beam flange as shown in Fig. 8, the rigidity of the connection is significantly more uniform.

A detailed examination of two of the connections tested at the at the University of Texas by the Swinden Laboratories, Moorgate, England, indicated final fracture in the one specimen initiated in the heat affected zone (HAZ) of the column flange. The initiation region, which was at the high stress location shown in Fig. 4, was in the column flange and was characterized by a flat fracture appearance and consisted of a mixture ductile microvoid fracture and fine grained cleavage.

To analytically model and accurately evaluate the states of strain and stress through the thickness of the column and beam flanges solid finite element models are required. Shown in Fig 9 is a typical finite element sub-model comprising 10 node linear strain tetrahedrons which was used to obtain three dimensional inelastic states of stress and strain in these connections and represents this current on-going seismic connection research effort.

Ralph M. Richard, Ph.D., is Professor Emeritus at the University of Arizona, Tucson, AZ; James E. Partridge, P.E., is President of Smith-Emery Company, Los Angeles, CA; Jay Allen, S.E., is President of The Allen Company, Los Angeles, CA; and R. E. Skip Radau is President of ROM Engineering, Tucson, AZ.

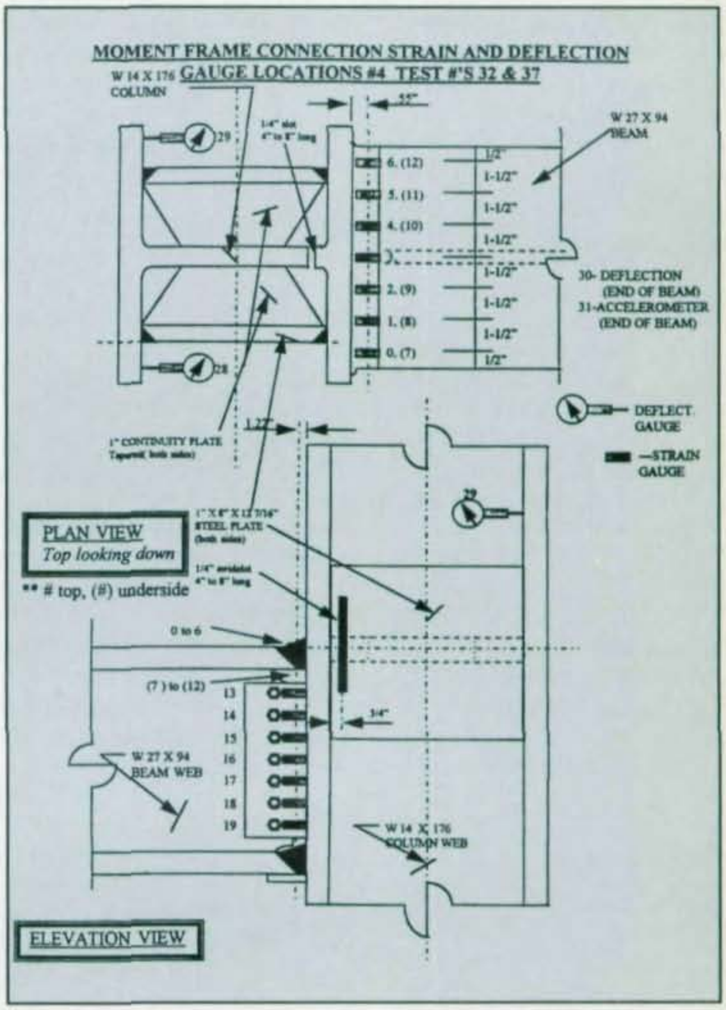

Fig. 8: Connection stiffeners and slot designs

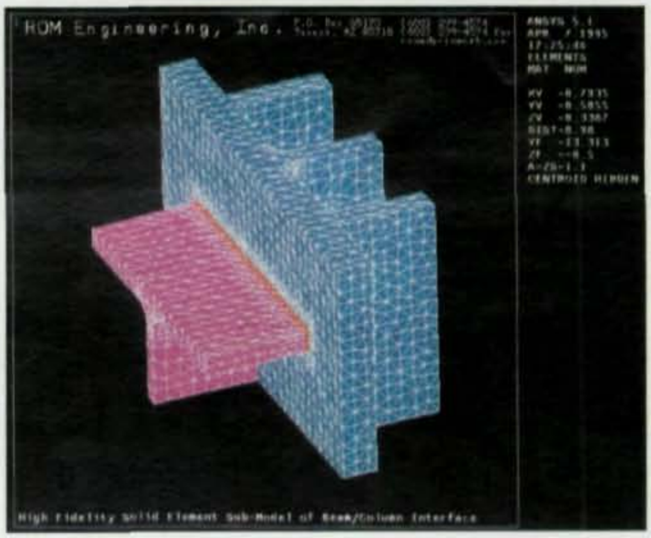

Fig. 9: Solid element sub-model of beam / colum interfaces

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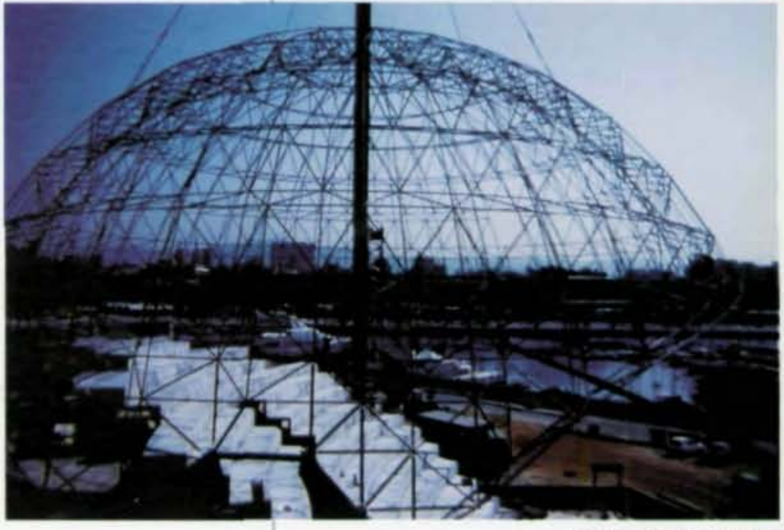

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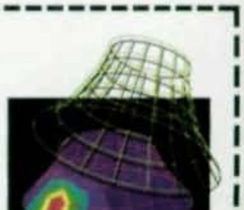

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