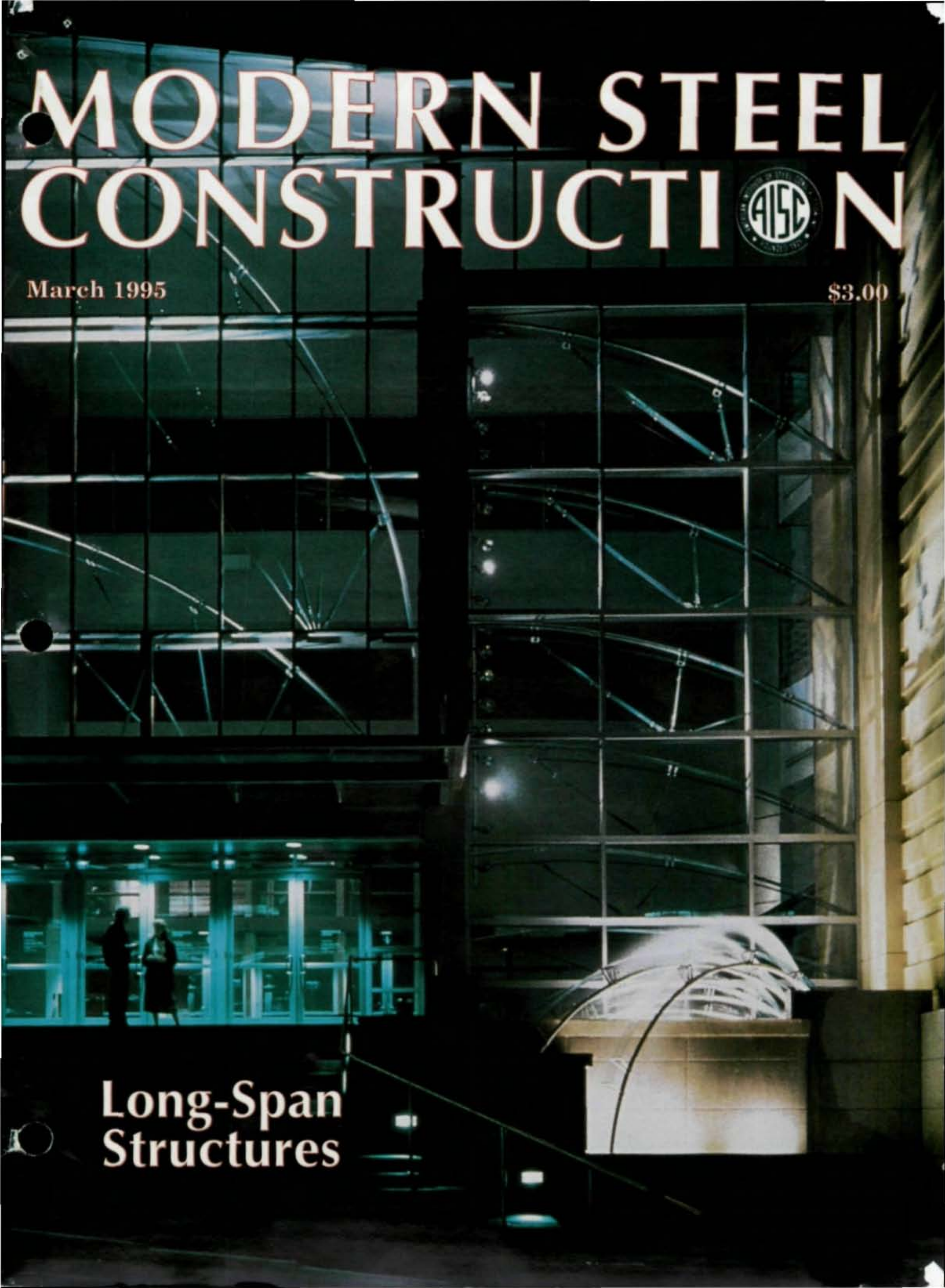
MODERN STEEL CONSTRUCTION

March 1995

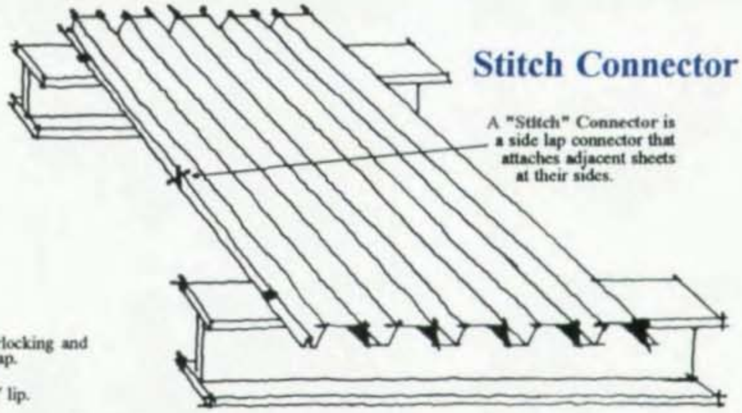
$3.00



Long-Span Structures

deck connection terms

Stitch Connector

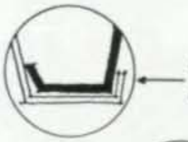


A "Stitch" Connector is a side lap connector that attaches adjacent sheets at their sides.

Side Lap Types

- *Common names include:* interlocking, button punch, and bayonet side lap.
- Can be button punched, welded, or screwed.
- Horizontal return on "female" lip.

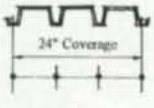
- *Common names include:* interlocking and button punch and bayonet side lap.
- Can only be button punched.
- No horizontal return on "female" lip.



- *Common names include:* overlapping, barge and foot, and nestable.
- Can be welded or screwed.

- Provides adjustment when required.

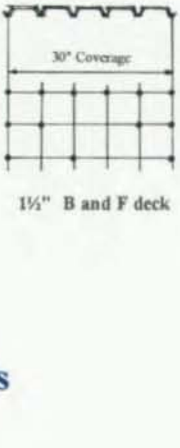
Typical Frame Fastener Layouts
for roof deck



24/4 pattern

3" N deck

- **Note:** In general the nomenclature is "cover width"/(number of welds).



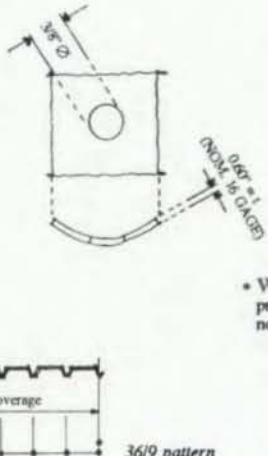
30/6 pattern
30/4 pattern
30/3 pattern

1½" B and F deck

36/9 pattern
36/7 pattern
36/5 pattern
36/4 pattern
36/3 pattern

1½" B and F deck

Welding Washer



- Welding washers are to be used on deck products lighter than 22 gage (0.028"). Do not use welding washers on side laps.

MODERN STEEL CONSTRUCTION

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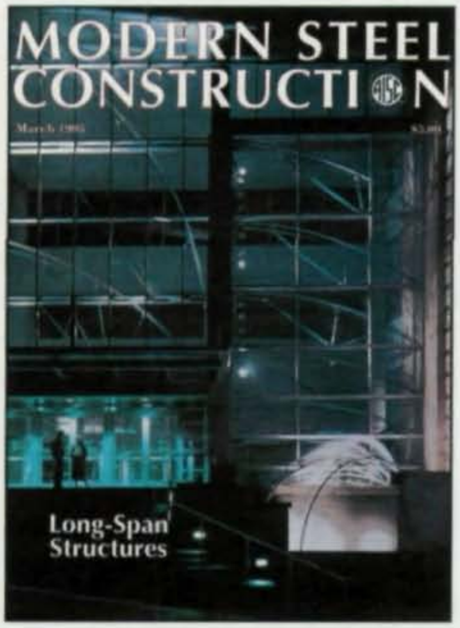

The multi-use Meydenbauer Center in Bellevue, WA, includes both convention space, offices and a dance studio, necessitating careful consideration of vibration issues. The story behind this innovative project begins on page 36. Photo by James F. Housel

FEATURES

DEPARTMENTS

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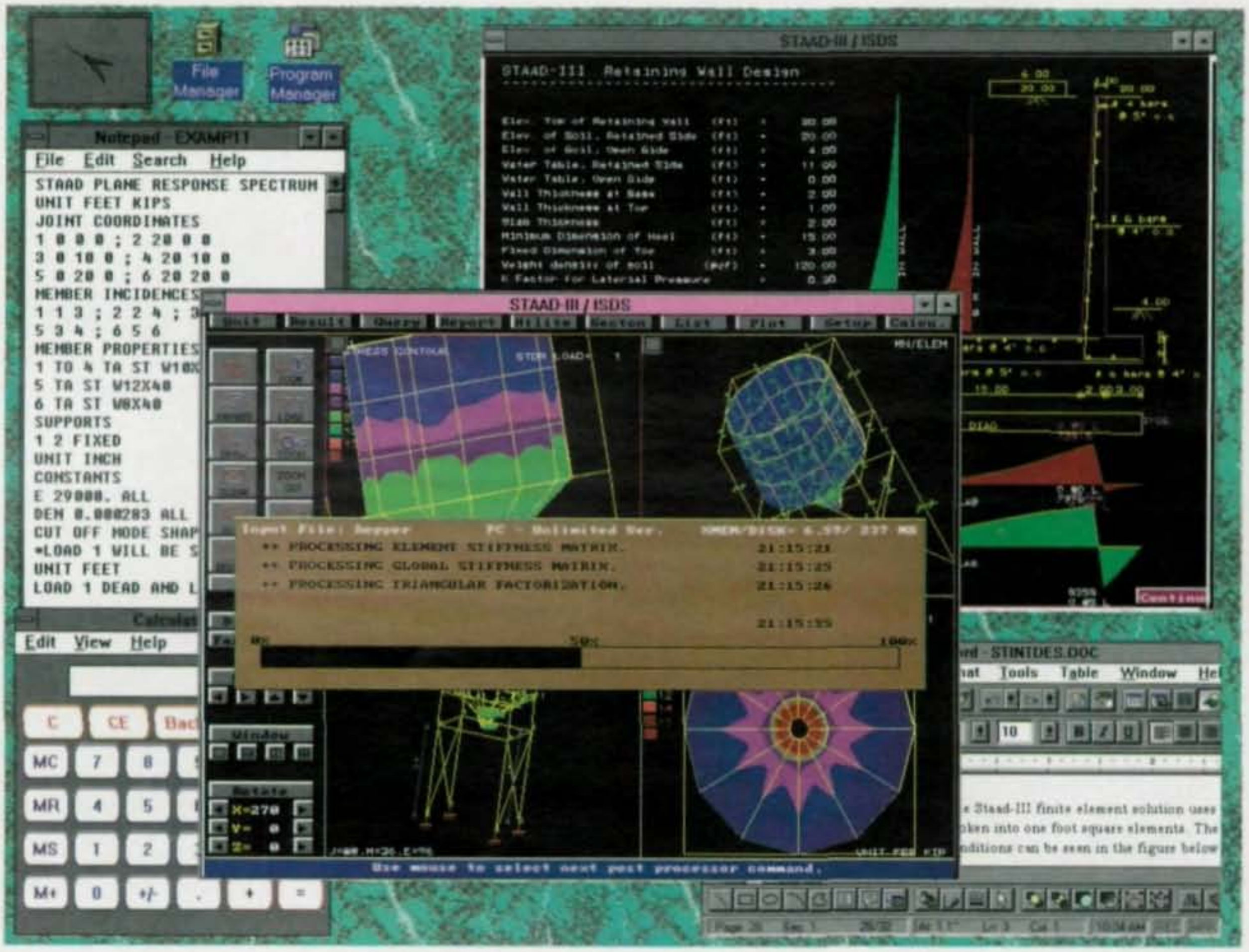

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Reaching AISC

I'M KNOWN AROUND THE OFFICE AS A BIT OF A CURMUDGEON. And when you add to that my oft-times stated dislike for poorly executed voice mail systems, it came as a wonderful surprise to the powers-that-be at AISC that I actually like our new voice mail system.

Why haven't I vilified this new technological intrusion? Because, quite simply, it's a well executed design that is actually an improvement over the old system. Unlike many other voice mail systems, this one is clear and simple. Too often, companies have general numbers that are answered with long-winded messages requiring the caller to punch in a long list of replies before being transferred into some black hole with no easily accessible method of reaching a live person.

AISC's system is different. Everyone has their own phone number. If you don't know an individual's number, simply call the AISC switchboard (312/670-2400) and a LIVE operator will take your call and transfer you to the correct person. Of course, sometimes the switchboard is busy. In that case, an answering machine will ask for your name and phone number and someone will call you back. If you do know an individual's phone number, the system is equally simple.

For example, if you call me directly (312/670-5407) and I'm on the phone, you'll reach my voice mail, where you have ONLY three options: hang up and call back; leave a message; or dial zero to reach the AISC switchboard. No fuss, no muss.

The same procedure holds true for all other AISC departments, such as: LeAnn Schmidt in Membership (312/670-5432); Patrick Newman (312/670-5417), Charlie Carter (312/670-5414), Jacques Cattan (312/670-5430), Fred Beckmann (312/670-5413), and Nestor Iwankiw (312/670-5415) in Engineering; Mike Moffitt (312/670-5444) and Abe Rokach (312/670-5416) at AISC Marketing; and Tom Schlafly in AISC Quality Certification (312/670-5412).

There has been one additional change, however, and this one is even better. AISC has introduced a new, toll-free number direct number for publication sales: 800/644-2400. While this number won't allow you to reach any other department at AISC, it will allow you to reach the publications department at no charge (for AISC Software, however, call 312/670-5411).

So if you have questions about *MSC*, call 312/670-5407 (or 708/699-6049 for advertising information). If you need information about AISC, call one of the numbers listed above. And if you want to order an AISC publication or request an AISC Publications list, call 800/644-2400. **SM**

STEEL INTERCHANGE

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

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

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

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

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

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

When making a wide flange section out of three plates, can you weld on only one side of the web?

A solution to the problem requires a tee joint. This partial penetration joint is prequalified by the American Welding Society. One criteria for its use is that the web plate be greater than ¼" thick. Preparation of the web plate requires a single 45 bevel. The effective throat thickness is determined by the depth of the bevel minus ".

Ray Schork
Bayer-Becker Engineers
Fairfield, OH

Another response:

Yes. The weld between web and flanges can be on only one side and need only be large enough to transfer the horizontal shearing stress between the web and the flanges under static loads. In some cases, if the web of this built-up section is thin enough that the full penetration can be achieved with a fillet applied on only one side. Especially, this can be commonly found at main framing members of a pre-engineered steel building on which the majority of main frames comprise three-plate (built-up) sections.

However, if the built-up members are subject to dynamic loads, the notch which is sometimes produced by welding on one side only may lead to fatigue problems. In such a case, fatigue stresses should be checked according to the procedures defined in the AISC Manual and other applicable codes.

Kunming Gwo, P.E.
HCI Steel Building Systems, Inc.
Arlington, WA

Another response:

The weld between the flange and web of a fabricated wide flange section that is a flexural or compression member should be determined by the computed shear between the flange and web. If the computed shear can be carried by a weld on one side of the joint there is no reason to arbitrarily weld both sides. Millions of welded members in rigid frames with one-side welds are performing well all over the world and have been for over fifty years. As with any production process, tolerances and quality of workmanship must be monitored. The angle between the flange and web should not be allowed to deviate significantly from 90 degrees, for instance.

There are cases where one-side welding is not appropriate, for example, crane beams and beams that are going to be hot-dip galvanized. Beams with high tension loads perpendicular to the flange (i.e. underhung crane attachments and moment connections to the flange) should be reinforced with a second weld in the affected area.

Donald L. Johnson P. E.
Principal Engineering Consultant
Butler Manufacturing, Research
Grandview, Missouri

What is the most efficient and cost-effective way to connect a steel wide flange girder to a concrete column?

Assuming moment does not have to be transferred through the connection, the simplest and most efficient connection would probably involve the installation of a plate, with appropriately sized anchors to transfer shear, flush with the face of the column. The plate would be secured to the inside face of the column formwork to prevent its dislocation during concrete placement. After the formwork for the column has been stripped, either a shear tab, seat angle or double angle could be welded to the face of the steel plate so as to produce a "standard" simple connection.

This concept is frequently used in precast concrete construction and design information for the embedded plate can be found in the PCI Design Handbook or in various ACI publications.

Stuart K. Jacobson
Stuart K. Jacobson & Associates, Ltd.
Northbrook, IL

New Questions

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

If you have any questions, please send them to: Steel Interchange, c/o 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.

Are there any limitations on the span to depth ratio of beams required by AISC Specification for Structural Steel Buildings?

The Manual of Steel Construction includes many items that are used along with structural steel frames, this is very convenient for structural engineers. However, some of the tables do not provide all of the information needed by engineers. One of the tables that AISC includes covers the dimesioning of cotter pins. What is the strength of cotter pins listed in the Manual of Steel Construction? Where can these items be obtained?

The AISC Code of Standard Practice in Section 2.1 lists items that are classified as "strutural steel" while Section 2.2 lists those items not classified as "structural steel". The rules is the AISC Specification and Code of Standard Practice apply to sturctural steel members. One of the items classified as "not structural steel" is stairs, catwalks,handrail and toeplates, what criteria is used to design these members?

In a partially cover-plated column, as shown below, how would you analyze the column for governing l/r ratio to calculate F_a?

Vijay P. Khasat, P.E.
Clinton, OH

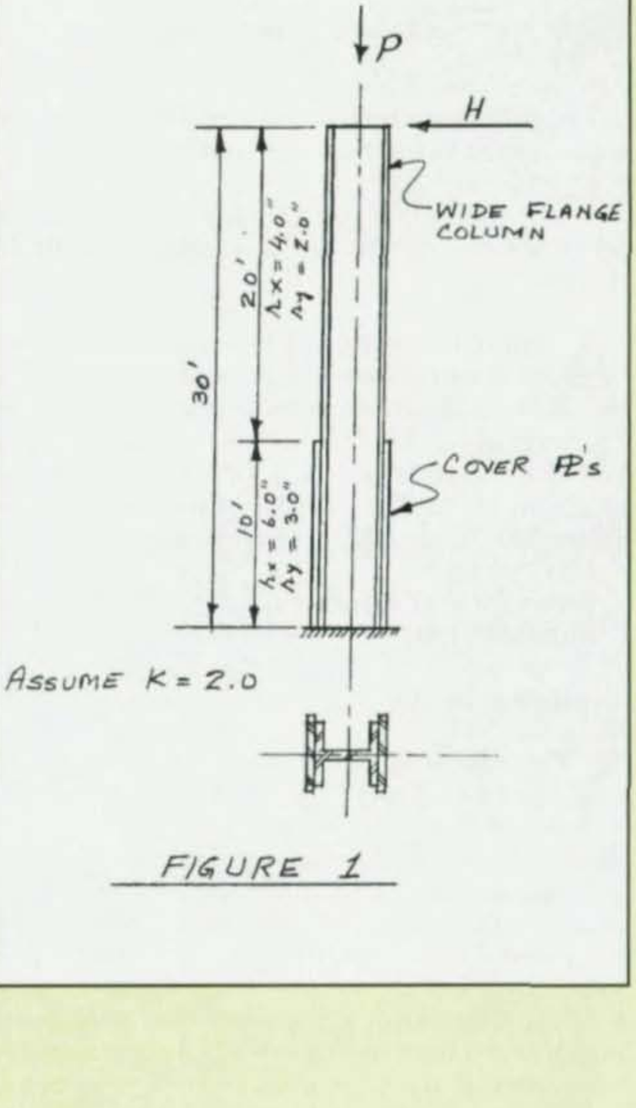
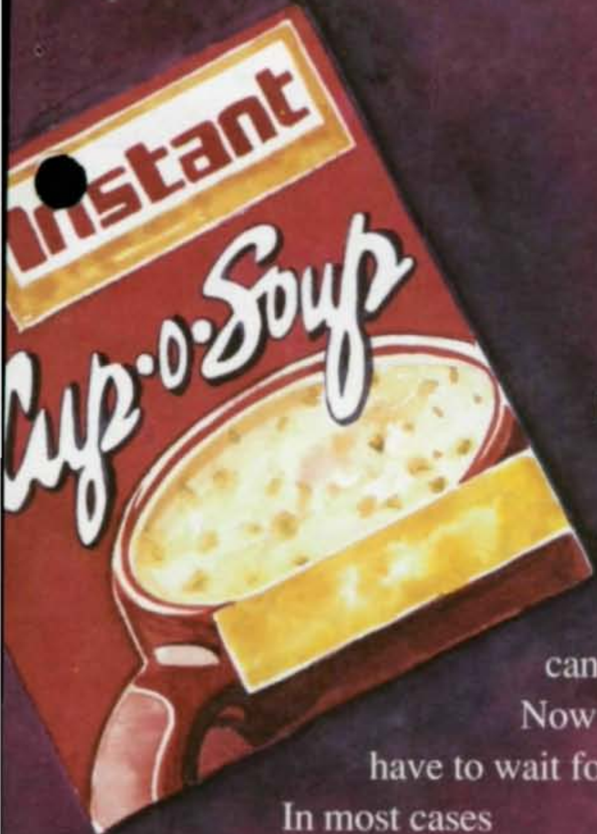


FIGURE 1



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CHAPARRAL STEEL

STEEL CALENDAR

April 4 - 6: AWS International Welding & Fabricating Exposition, Cleveland

More than 20,000 attendees are expected at the 75th Anniversary Show. Keynoting the show will be Miller Electric 's president, Dennis Martin, discussing important issues facing the welding industry. Also, this year's Comfort A. Adams lecture will feature Trevor Gooch, technology manager for the Materials Department at TWI in Cambridge, England, discussing "Corrosion Resistance Of Stainless Steel Weldments." In addition, more than 90 other papers will be presented, along with an extensive product exhibition and a symposium on thermal spraying. For more information, contact the AWS Conventions & Expositions Department, American Welding Society, 550 NW LeJeune Road, Miami, FL 33126; ph: 800/443-WELD.

May 17-19: National Steel Construction Conference, San Antonio, TX

The premier industry event for structural engineers and steel fabricators will feature more than 20 seminars on such topics as: Reducing Structural Steel Costs; Interpreting the Mexican Market; New Concepts in Industrial Building Design; Weldability, Fracture Mechanics, and Metallurgy; Seismic Design Solutions After The Northridge Earthquake; Connections For Hollow Structural Sections; Effective Project Specifications; Inspection Of Welded & Bolted Joints; Economical Painting; OSHA Training For Fabricators; Fabricating Steel Tubes; Constructability Issues; Flame Straightening Technology; and Team Building/Partnering. This year, the NSCC also has added sessions of special interest to contractors and construction managers. In addition, the Conference offers a product exhibition with nearly 100 booths. For more information, contact AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001; ph: 312/670-2400; fax 312/670-5403.

June 19-21: 12th Annual International Bridge Conference & Exhibition, Pittsburgh

Sponsored by the Engineers' Society of Western Pennsylvania, the IBC features numerous seminars on the design and construction of bridges, as well as an extensive product exhibition. For more information, contact: Engineers' Society of Western Pennsylvania, Pittsburgh Engineers' Building, 337 Foruth Ave., Pittsburgh, PA 15222; ph: 412/261-0710; fax: 412/261-1606.

Partnering Tops ADR List

PARTNERING AND MEDIATION ARE CLEARLY THE PREFERRED AND MOST EFFECTIVE forms of alternative dispute resolution (ADR) within the construction industry, according to a 1994 survey of 2,400 attorneys, design professionals and contractors by the Construction Industry Dispute Avoidance and Resolution Task Force (DART), Washington, DC. Architects and engineers clearly viewed project partnering "as a superior method" for achieving desired results, concluded survey director Thomas J. Stipanowich, alumni professor of law at the University of Kentucky College of Law. The AGC, continued Stipanowich, was "extremely favorable toward prospects of project partnering and tended to view it as a highly effective vehicle for achieving a host of goals on construction projects."

The survey asked respondents to rate various forms of ADR on their relative effectiveness in achieving goals. Partnering was rated most effective by both DPIC-insured design professionals and the AGC members for:

- Reducing dispute resolution time
- Reducing related costs
- Minimizing future disputes
- Opening channels of communication
- Preserving or enhancing job relationships
- Meeting job budgets and schedules.

(Partnering will be the subject of a seminar at this year's National Steel Construction Conference. For more information on the NSCC, call 312/670-5420 or fax 312/670-5403.)

OTHER RESULTS INCLUDE:

- Nearly two-thirds of surveyed contractors were familiar with partnering; 35% had been involved with partnered public projects and 22% with partnered private projects.

•More than eight out of 10 of both AGC members and DPIC insured design firms predicted future increases in the use of partnering, as did 72% of the ABA Forum on the Construction Industry and 71% of the ABA Public Contracts Section members.

•85% of the ABA Forum members predicted a future increase in the use of mediation, as did 84% of the ABA Public Contracts Section members, 71% of DPIC insured design firms and half of the AGC members.

•Partnering was the ADR technique that the ABA, DPIC and AGC all felt should be most "encouraged" in contract language.

•Combined, DPIC and ABA respondents reported that favorable experiences with partnering outnumbered unfavorable experiences by a nearly five-to-one margin. Favorable experiences with mediation outnumbered unfavorable experiences by two-to-one. Both groups reported more unfavorable that favorable experiences with binding as well as non-binding arbitration.

Russ Chaney, DPIC's senior vice president of loss prevention services, says the DART survey results were very encouraging to the insurance company and reflected the successful efforts of all sectors of the construction industry to bring change to how parties resolve disputes.

"The trend toward more and more litigation is finally reversing," Chaney said. "Maybe it's time that ADR stands for 'appropriate' dispute resolution, and litigation becomes the last chance alternative to resolving construction industry disputes."

DPIC Companies recently announced that 30% of its open claims file are now in some form of ADR—primarily mediation—up from 10% in 1991. It encourages the use of mediation and partnering by its policyholders through financial incentives.

Field Of Dreams

A design study showed that structural steel frame was cost competitive with a basic metal building for the new University of Kentucky Field House

By Kurt D. Swensson, Ph.D., P.E. and Douglas W. Robinson, P.E.

THE OPENING OF THE UNIVERSITY OF KENTUCKY FIELD HOUSE IN MARCH OF 1993 was the realization of the Wildcat athletic program's dream for a premiere indoor athletic facility. This state-of-the-art facility serves the football, baseball, track and gymnastics programs, as well as the university population at large. The 250-ft. by 450-ft. facility encloses almost six million cu. ft. of space. The approximately 132,000 sq. ft. of floor space includes:

- 108,000-sq.-ft. field that contains a full-size football field or two full baseball infields with 65-ft. maximum clearance to structure
- Six-lane 290-meter track with nine-lane straightaway
- Area for field events, including high jump, pole vault and shot put
- 9,000-sq.-ft. gymnastics area with 19-ft. clearance to structure

Miscellaneous storage, office, lobby, locker and observation areas.

The facility also includes a motorized netting system suspended from the structure that can separate the field area into isolated areas so various activities may occur simultaneously. Inverted channels spanning between joist bottom chords support the lighting system, allowing indirect, even lighting that supplies approximately 120 footcandles without creating glare.

Approximately 751 tons of rolled steel sections and 130 tons of joists define the bulk of the structure. Structural steel was erected in approximately four months at a cost of $1.2 million, while the entire cost of the project was approximately $8.5 million.

DESIGN STUDIES

As in any project, a major goal of the design team was to provide the most building for the least dollar. The athletic department's program required a minimum 450-ft. long by 250-ft. wide facility with a 65-ft. clear height at the center of the field. In response, the design scheme developed four schemes, varying from a no-frills scheme to a quality finished facility scheme for the preliminary pricing.

Scheme A — Base level metal building with metal side panels

Scheme B — Gable roof supported by exposed concrete piers

Scheme C — Flat roof supported by steel columns

Scheme D — Steel arched roof supported by exposed concrete piers

The team eliminated Scheme C because of the significantly larger volume of conditioned space compared to Schemes A, B and D. Comparison of the remaining schemes resulted in the selection of Scheme B. The architect and university decided that the quality and flexibility of design provided by Scheme B more than made up for the approximate 10% cost premium compared with Scheme A. The gable roof shape, while not as exciting aesthetically as the arched shape, provided more workable space than the arched roof shape of Scheme D.

The initial cost estimates only suggest the order of magnitude difference in cost between the systems. Since the site is underlain by rock, the foundation costs were not sensitive to the structural system weight. Therefore, the team used the same foundation costs for all systems.

The final floor plan includes a 450-ft. by 250-ft. column-free practice field section and two ancillary spaces. The practice field houses a playing field and

track. Lobby, lockers and office space and a 90-ft. by 120-ft. column-free area for gymnastics and lockers are housed in the north ancillary area, while storage space is housed in the south ancillary area.

The total cost of the structural system, excluding wall and roof panels, was approximately \$2.2 million, or \$16.6 per sq. ft. Structural engineer on the project was Stanley D. Lindsey and Associates, Ltd., Atlanta.

Roof Structure

The roof system above the practice field and south ancillary space, known as the high roof, consists of a standing seam metal roof on 3 in. of rigid insulation. The slope of the high roof is 5-in. rise in 12-in. horizontal. A $1^1/_2$-in.-deep Type B painted metal roof deck that spans approximately 6-in. parallel to the slope supports the metal roof system. A cost comparison of purlins to joists resulted in the use of open web joists to span the approximately 30 ft. between the trusses in the practice area. The main structural steel trusses have straight chords and are kinked at the peak of the roof. Typical trusses over the practice area are approximately 10-ft. deep and weigh approximately 250 plf.

Concrete piers support the trusses at a height 24 ft. above the practice surface. The concrete piers are 7-ft.-4-in.-long by 2-ft. wide at the base. A column capital at the top of each pier provides a length of 10-ft.-4-in. at the truss bearing point.

The roof system of the north ancillary area consists of a traditional built-up roof placed on a flat structure. Sloped insulation provides the needed slope for drainage. The offices, lobby and locker facilities are located in a triangular area with a roof height of approximately 20 ft. above the finished floor. The north ancillary roof structure includes $1^1/_2$-in. Type B metal roof deck, open web joists and wide flange beams. A typical bay is 30-ft. by 30-ft. in this area. Above the 90-ft. by 120-ft. clear span gymnastics area the structure consists of 3-in.-deep Type N painted roof deck on 48-in.-deep long-span joists spaced at 10-ft. on center and spanning 90 ft. The roof over the gymnasium is approximately 24 ft. above the finished floor.

Walls & Foundation

The wall systems consist of a combination of masonry walls near the ground, a band of precast panels from 16 ft. to 32 ft. above grade, and metal wall panels above 32 ft. Along the practice area and north ancillary area, the precast panels are supported by the concrete piers at approximately 30 ft. However, along the 150-ft. diagonal that forms the face of the south ancillary area, the roof trusses carry the 16-ft. width of precast.

The foundation consists of shallow spread footings bearing directly on shale. Forces transmitted through the practice area trusses to the piers required rock anchors to be added to the design.

Lateral Load Resisting System

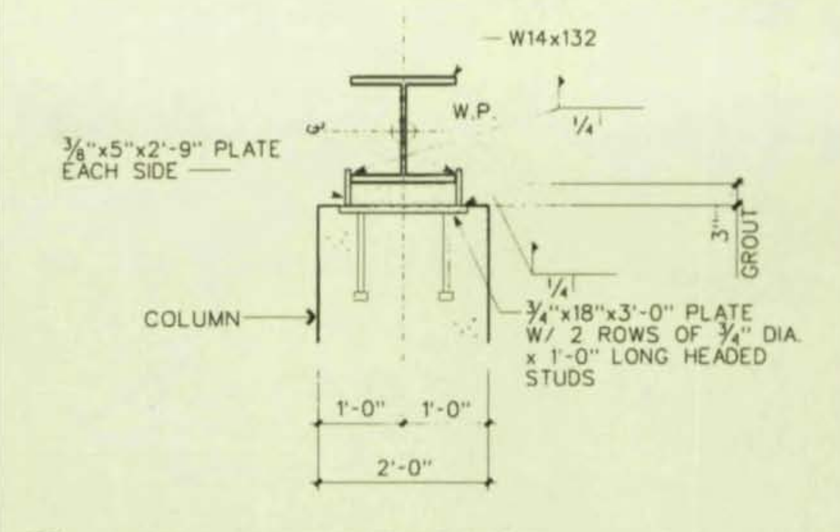
Lateral load resistance is provided by the cast-in-place concrete piers. The piers act in combination with the structural steel trusses to create frames that resist both gravity and lateral loads that act perpendicular to the ridge line. Lateral loads that act parallel to the ridge line are transferred through the metal roof deck diaphragm to the exterior line columns. Vertical X-bracing between the piers transfer the lateral loads from the top chord level of the trusses to the top of the concrete piers. For lateral loads parallel to the ridge, the concrete piers act in the weak axis as cantilevered columns to transfer the loads to the ground.

Flexible Connections

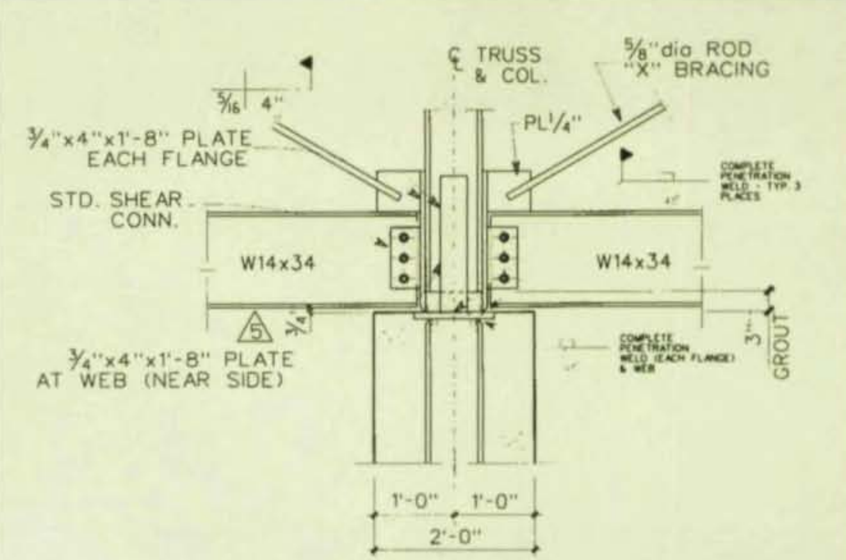
One key to the success of any project is for the engineer to take advantage of any structural properties inherent in an architect's concept of a building. For this project, the architect's vision included exposed massive concrete piers along the 630-ft. total length of the main structure plus the two ancillary areas. The exposed piers visually break the long horizontal line of the building and reduce its scale. Architect on the project was Sherman, Carter, Barnhart, Lexington, KY. The useful structural property possessed by these piers is stiffness. To take advantage of each pier's inherent stiffness, the steel trusses were designed with fixed ends. Fixing the ends of the trusses reduced the design moments by a factor of 8/12 and reduced deflections of the truss by a factor of 5. Thus, the system—including the trusses and concrete piers—was designed as a single bay with a fixed base.

Taking advantage of the pier stiffness allowed for a reduction in truss depth from approximately 15 ft. to 10 ft. Reducing the depth of the trusses accomplished two things. First, the amount of conditioned space was reduced by 10% with a comparable reduction in the mechanical systems size and cost. Second, the amount of exterior wall and insulation was reduced by approximately 15%. Thus, reduction of truss depth resulted in significant savings to the project not only initially, but over the entire life of the facility due to continued savings from reduced utility costs and maintenance on the exterior facade.

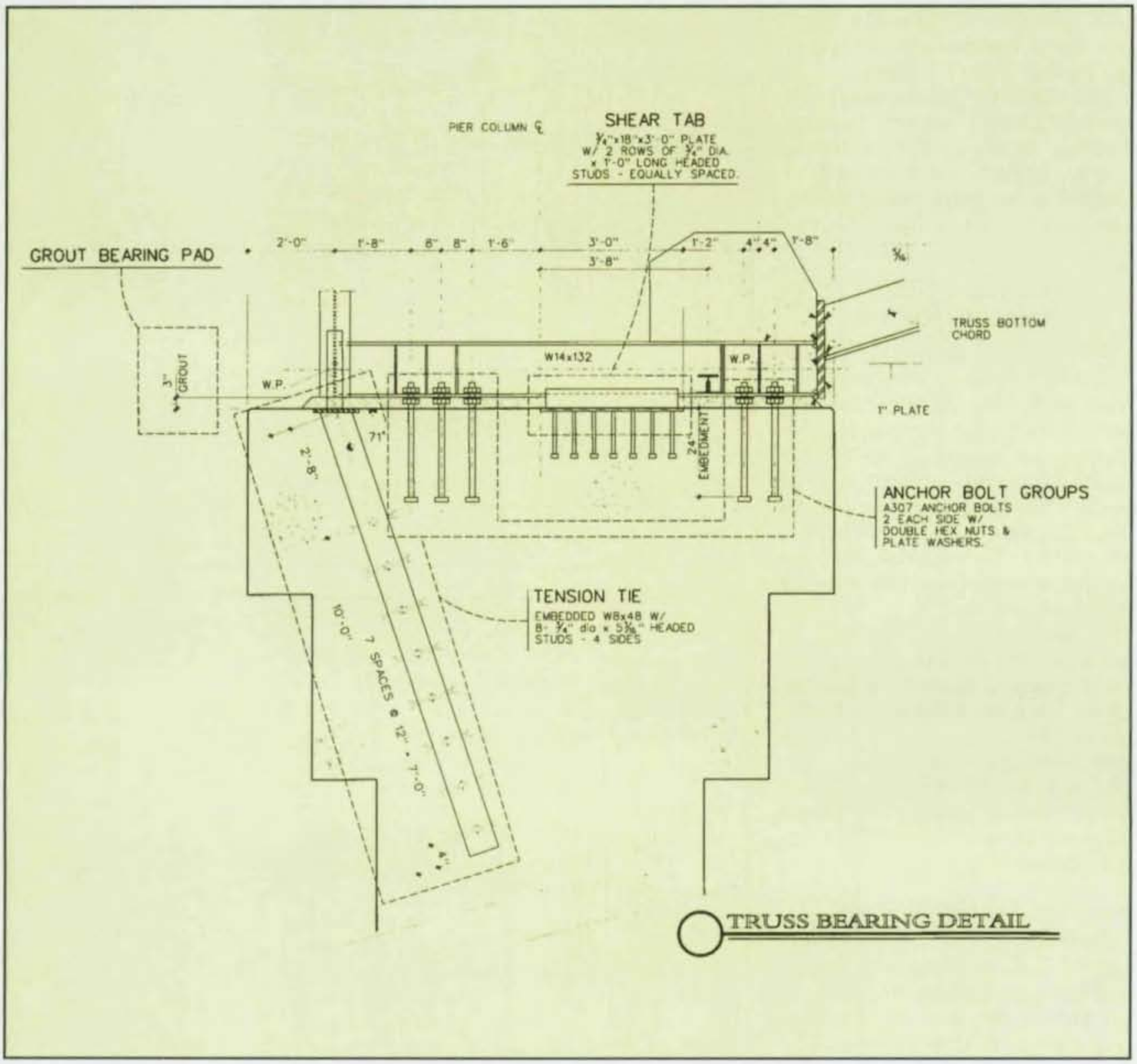
Maximum calculated vertical deflection of the trusses under



SECTION AT SHEAR TAB



SECTION AT TENSION TIE



full gravity loads is approximately 2.5-in. This represents a deflection span ratio of 1/1,200, which is impressive for a standard truss weighing only 250 plf.

While fixed connections certainly provided significant savings in the structural steel design, detailing proved to be a challenge. The connection had to transfer large forces in a relatively small area and account for dimensional tolerances. Forces attracted to the concrete pier include bending, axial, shear, thrust forces and restraint forces due to temperature effects.

The idea of erecting a series of 250-ft.-long trusses on concrete piers with complicated cast-in-place connections was not a pleasant one. Experience has shown that often a majority of connection items embedded in concrete end up in the wrong place. Also, the maximum 24-in. pier width left no room for a redesign in the field. And finally, if problems were to occur, there was no time for arguments between the steel erector and the concrete subcontractor who set the embeds. Thus, the connection design had to account for a reasonable tolerance for steel erection and concrete placement. General contractor on the project was C.E. Pennington, Lexington, KY.

The connection includes four major elements:

1. Two sets of $1^1/_2$-in.-diameter anchor bolts for initial placement
2. A tension tie consisting of an embedded WF section and two weld plates
3. A shear tab consisting of 14 $^3/_4$-in.-diameter headed shear studs, embed plate and two shear tab plates
4. A grout pad to transfer compression loads

The two anchor bolt groups supported the truss during erection and allowed for adjustment before final welding. Four bolts on the inside of the connection resist compression and six bolts on the outside resist tension. After final adjustments, the cranes could cut loose from the truss and welding could be completed later. This design helped to speed erection of the trusses.

In the field, the connection worked very well. The steel erector was able to erect the main trusses at a rate of two per week utilizing a set of three cranes. More importantly, no field changes to the connection were required. The major problem in execution of the detail came during placement of the concrete piers. Congestion resulting from the large amount of reinforcement, the WF tension tie, and the anchor bolts made proper consolidation of the concrete in the pier difficult. However, some adjustments in the concrete mix solved the concrete placement problem.

To complete the system design, the large thrust force that results from the five to 12 slope of the truss had to be resolved. Since this thrust is applied at the top of the piers more than 24-ft. above the foundation, the thrust creates significant bending moments and shear in the concrete pier. The shear forces are transferred to the underlying rock through passive pressure in the face of the rock excavation and friction. The bending forces are transferred through a force couple consisting of compression on the rock and tension in a grouping of 12 #11 reinforcing bars grouted into the rock.

This project was completed successfully with the use of rolled steel sections because of the flexibility they afford in design and construction. The combination of the steel trusses with the exposed concrete piers to create a continuous frame provided an economical and efficient structure that fulfilled the vision of the architect and the dreams of the University.

The flexibility of the steel structure allowed for the architectural form envisioned by the architect and approved by the owner. Further, the custom design procedure provided unique supports for the mechanized netting, lighting systems, and mechanical units required for optimum operation of the facility. Where pre-engineered systems tend to make the user revise the facility program to match the needs of the structure, traditionally designed and fabricated steel structures resulted in a building that matches the owner's requirements both aesthetically and functionally.

Kurt D. Swensson, Ph.D., P.E., and Douglas W. Robinson, P.E., are associates with Stanley D. Lindsey and Associates, Ltd., in Atlanta.

BUILDING ON AN ATHLETIC TRADITION

A new athletic facility/alumni complex for the University of Houston is designed to help recruit top-notch student athletes

The multi-use athletic facility/alumni center features five distinct spaces, including an 84,000-sq.-ft. practice field and areas for weight training, lockers, offices, and other sport activities.

By Douglas G. Ashcraft, P.E., S.E., and Shatha Lingle, P.E.

THE UNIVERSITY OF HOUSTON, WHOSE MAIN CAMPUS STANDS VIRTUALLY IN THE SHADOW of the skyline of the country's fourth largest city, enjoys a rich tradition of intercollegiate athletic excellence.

Over the last three decades, the Division I-A school has spawned a rare group of athletes, headlined by: Carl Lewis, who went on to become one of the world's greatest track athlete; Andre Ware, winner of the 1989 Heisman Trophy; and Hakeem Olajuwon, main man of the memorable Phi Slamma Jama UH teams of the 1980s and reigning NBA Most Valuable Player. Building on the achievements of these and earlier athletic heroes, the University recruited strongly and enjoyed a reputation as a team to be reckoned with in the Southwest Conference in every major sport through the 1980s.

However, as recruiting pressures for top talent increased from both in- and out-of-state competitors, the University found itself suffering from a lack of modern athletic facilities. With their most recent on-campus training and sports venue constructed in 1969, the school's facilities were not on par with what athletes could find at competing NCAA Division 1-A institutions. Indeed, some recruits from high schools in Texas and elsewhere would find that com-

ing to the University of Houston would mean a step down in facilities. It was clear that to continue to attract top student-athletes in the modern landscape of college athletics would require a large investment in athletic infrastructure. Fueled with the vision and funding of a highly successful UH alumni contribution campaign, the University has taken a large step to correct this deficiency with the construction of a state-of-the-art new Athletic/Alumni Complex. When complete in late 1995, this $25 million complex will provide a new indoor varsity athletic training facilities featuring a covered football field, weight training room, a sports Hall of Fame and alumni services center, and adjacent varsity baseball and tennis facilities.

Alumnus John Moores, founder of BMC Software, envisioned that his alma mater would be a thriving force in the community with a combination of academic and athletic excellence. Moores, who serves on the university's Board of Directors, backed up his vision by giving $51 million to the university, the largest private donation ever for a public institution. Knowing that successful intercollegiate and intramural programs are an integral part of a thriving university, nearly half that gift was earmarked for a combined athletic training facility and alumni association center. Ground was broken for the facility in the winter of 1994.

MULTI-FUNCTION FACILITY

The architectural team, which included lead design firm HOK/Sport of Kansas City, and associated architects Kendall-Heaton Associates, Pleas Doyle Associates, and Molina Associates, all of Houston, was challenged to design a multi-function facility within a very tight budget. The development manager, Hines Interests Ltd. Partnership, was charged to oversee the design and construction within the time and budget constraints.

The single 220,000-sq.-ft. building will house five distinct functions. A 16,000-sq.-ft. area for weight training, lockers, showers, and sports medicine for several men's and women's sports will fill much of the first floor. Coach's and athletic offices will be located on the second floor along with significant space devoted solely to student-athlete programs. In addition, portions of both the first and second floors will house offices and meeting areas for the University of Houston Alumni Association. The fourth functional space in the building is an athletic Hall of Fame.

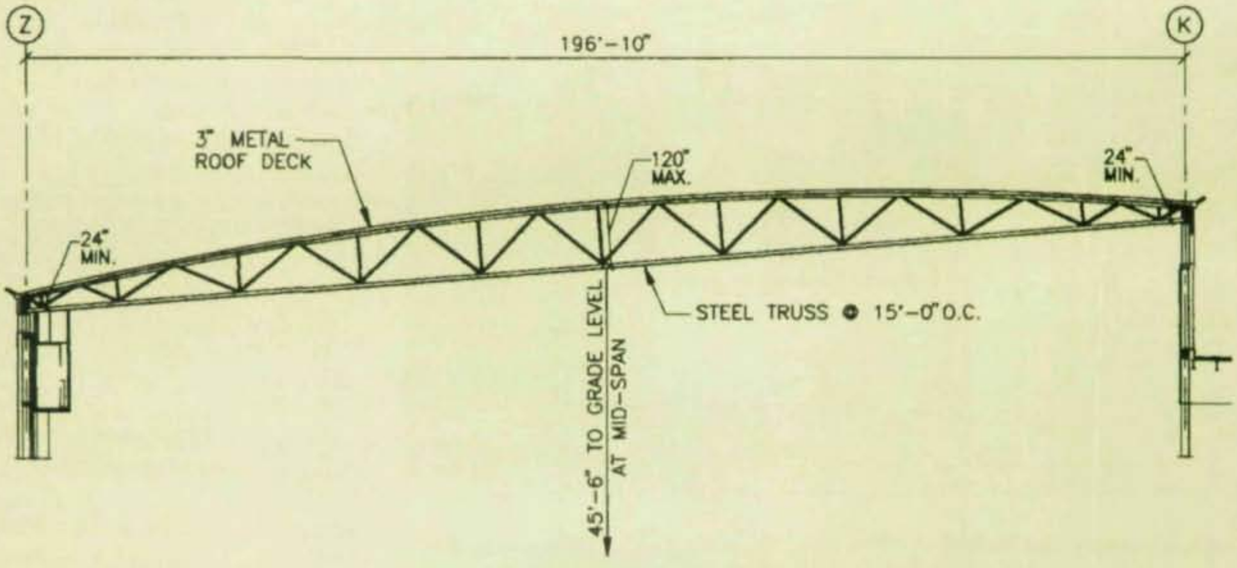
The fifth component space—and visually the most dominant feature of the building—is an 84,000-sq.-ft. indoor practice facility. Measuring 420-ft. by 200-ft., this fieldhouse will provide facilities and space for track, tennis, basketball and volleyball training its typical configuration. When the "Magic Carpet" is unrolled on a cushion of air, the fieldhouse rapidly and gracefully transforms into a full-length football field with adequate height to practice all phases of the game except punting.

Vertically-retractable hung curtains give the facility additional flexibility by subdividing the area, allowing simultaneous practice of multiple sports.

Structural Systems

A number of limitations challenged structural engineer Walter P. Moore and Associates, Houston, in the design of the multipurpose indoor practice facility. The distinctive exterior architectural facade features a gracefully curved roof surface, dictating a complex curved superstructure. Interior columns were obviously undesirable within the 200-ft. by 420-ft. practice space. Further, football practice activities dictated a minimum clear midspan height of more than 45 ft. beneath the roof. At the same time, clearances for mechanical ductwork at different locations within the facility limited the depth of the roof framing elements. Lateral bracing was limited to the perimeter, posing yet another structural challenge.

Walter P. Moore's engineers found that a braced structural steel frame featuring long-span steel bow-string joists best met the structure's design requirements. The sloping joists, manufactured by AISC associate-member Vulcraft, typically span 197-ft. between side wall frames. The joists, which are spaced at 15-ft. centers, range in depth from a maximum of 120-in. at midspan to just 24-in. at bearing points. Typical top chord sizes are (2)L5x5x5/8; typical bottom chord sizes are (2)L5x5x1/2. A 3-in.-deep, 16 gage acoustical metal deck was chosen to span between the joists and serve as a diaphragm to efficiently transfer lateral loads to the perimeter braced frames. The bottom chords of the joists are braced with five rows of diagonal bridging. Where required to accommodate mechanical ductwork within the structural depth, horizontal bridging was used. Rolled steel W12 beams span between the bottom chord of the joists to support dividing cur-



TYPICAL BOW-STRING TRUSS PROFILE

Spanning accross the 200-ft. by 420-ft. practice space are large bow-string trusses. These joists, which are spaced 197-ft. on center, range in depth from 120-in. at the midspan to just 24-in. at bearing points.

tains and a batting cage, allowing these elements to be hoisted up out of the way when not in use. Perimeter W27 girders support the bowstring joists and anchor the deck diaphragm. The perimeter beams in the end walls parallel to the joists consist of W16 and W18 shapes that were shop curved to follow the graceful architectural profile. A combination of X- and K-shaped angle bracing at two bays on each side of the perimeter provides simple, efficient resistance to the 90 mph hurricane winds expected in Houston. Wide flange steel columns support the roof and rest on bellbottom footings. The first floor consists of slab on grade built over select fill.

The remainder of the Athletic/Alumni complex is a two-story construction that contains other in-building functions. Typical bays are 28-ft. by 30-ft. After considering a conventional concrete pan joist system to frame the second floor, a composite steel system was chosen for its economy, speed of erection, light overall weight and aesthetic appeal. The composite floor consists of a 2-in.-deep, 19-gage metal deck topped with

Shown above is the still-under-construction exterior of the new athletic facility / alumni center at the University of Houston.

3.25-in. of lightweight concrete. A 1-in.-deep galvanized metal form deck with insulating concrete created an economical roof deck over the steel joists.

When completed in the fall of 1995, the University of Houston Athletic/Alumni Complex will showcase the past accomplishments of the University's premier teams and performers and provide a truly modern facility to help recruit and train the University's next generation of athletes.

Douglas G. Ashcraft, P.E., S.E., is a vice president and Shatha N. Lingle, P.E., is a senior associate of Walter P. Moore and Associates, Houston, and served, respectively, as structural project manager and project engineer. Walter P. Moore and Associates provided all structural engineering services for the University of Houston Athletic / Alumni Complex.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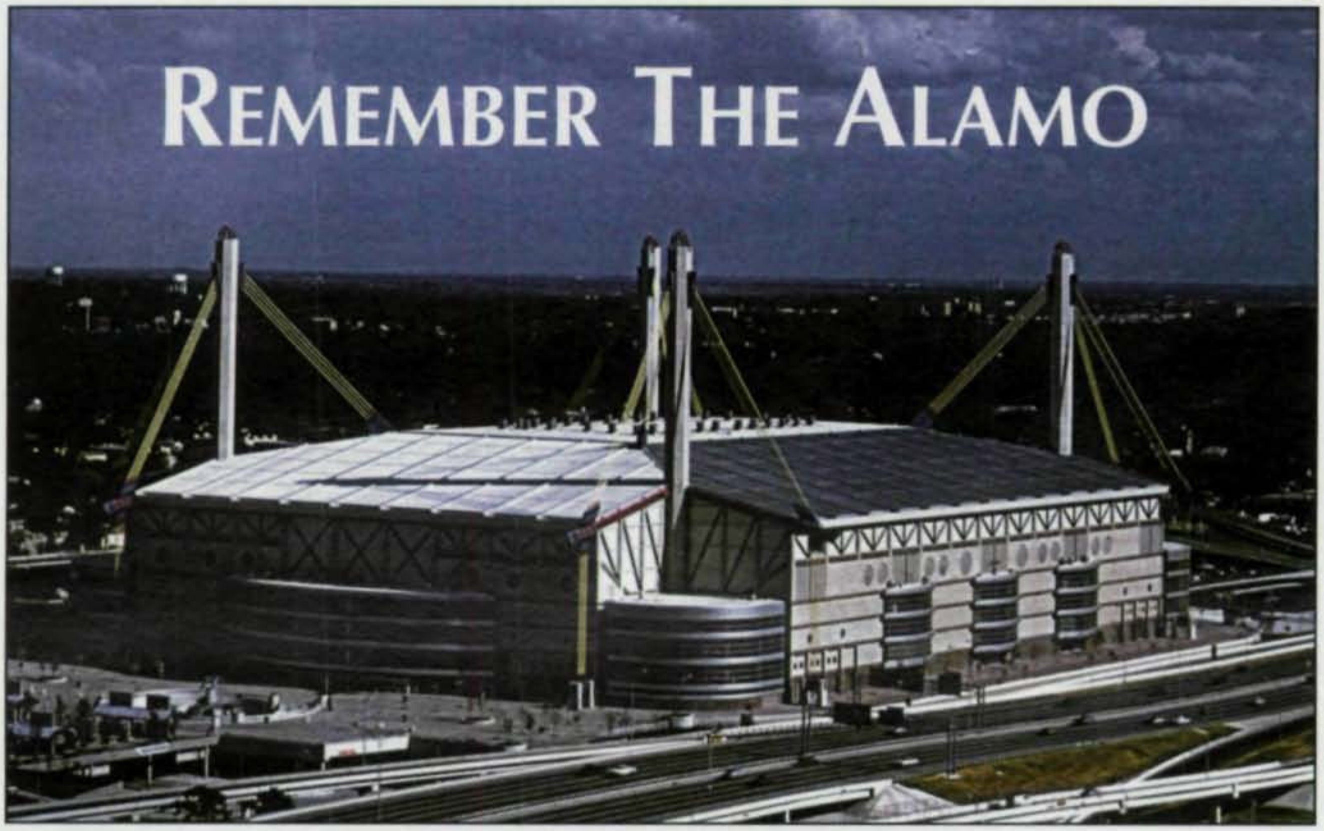

The Alamodome was designed to accommodate a wide variety of activities, from conventions to professional football

By H. Douglas Steadman, P.E. & John Wall, P.E.

ALREADY A DISTINCTIVE TOURIST DESTINATION, MANY COMMUNITY LEADERS in the mid-1980s believed that a domed stadium would greatly benefit the economy of San Antonio. After a city-wide referendum approved tax financing, a design team, including Marmon Mok (prime local architectural and MEP firm), W.E. Simpson Company, Inc. (San Antonio-based civil and structural engineering firm), and HOK Sports Facilities Group of Kansas City, was quickly put in place.

The design of the stadium was dictated by three major factors: a relatively narrow site; the need to create a large domed stadium with unobstructed sight lines; and a budget set at \$105 million. A fourth criteria, the need to be able to "blackout" the facility during daytime events to expand its multipurpose use, also was critical to the design criteria. To start with, the design team examined existing domed stadiums, which pretty much fell into three categories. The first generation of domed stadiums, including the Astrodome in Houston, Superdome in New Orleans, and Kingdome in Seattle, were true domes, where heavy structures of steel or concrete were combined with a circular shape to support long span roofs. Unfortunately, these stadiums proved very expensive to construct and the circular shape wouldn't fit the site. The second generation, including the Silverdome in Pontiac (MI), Metrodome in Minneapolis, Carrierdome in Syracuse, HoosierDome in Indianapolis, and B.C. Place in Vancouver, utilized light, air-supported fabric structures. Though less expensive to construct, the fans required to pressurize the structures were very energy intensive. Also, there were problems moving people and trucked equipment through air locks into the facilities and with deflations in some facilities. The latest generation, including the Skydome in Toronto and Suncoastdome in St. Petersburg, have relied heavily on "high-tech" solutions. The Skydome, for example, used very heavy steel arched trusses and a movable roof, which proved to be both complicated and inordinantly expensive.

Early in the design process, air supported roof structures were eliminated from consideration—despite the fact that the original budget was based on this type of design. More than a dozen structural schemes were considered, though these were quickly pared down to five:

- Space Frame
- Suspension System with Sticks
- Two-way Cable Truss
- Tied Arch
- Folded Plate Space Frame

The tied arch and folded plate space frame were next eliminated through a ranking system that considered the following four items (with associated weighted importance factors: cost (importance factor of 1.0), erection sequencing (0.7) architectural aesthetics (0.9), and function (0.5). The three factors that weighed heavily in the selection of the Cable Suspended System were: the increased opportunity for local participation; the simpler, proven technology; and the fact that it would be difficult to blackout the interior of the fabric roof proposed with the two-way cable system roof.

(Left) The outside tieback truss was held in place by the crane until cables were installed.

(Above) Field fabrication was required for the main support seat and strut truss. The seat is 27-ft. tall and the strut is made of two W36 members and plates.

Structural Design

The structural design needed to address site restrictions, budgetary constraints, maintenance considerations, even configurations, weight requirements for variable sound and light systems, and aesthetic quality. The design team included: W.E.

THE ATLANTA AI
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Arched chord joists top the Arrivals Hall, International Concourse, Hartsfield Atlanta International Airport

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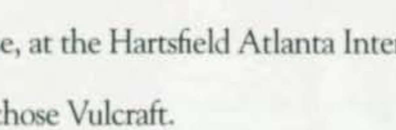

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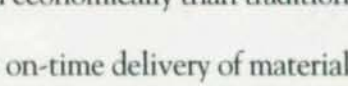

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Please circle # 37

Simpson Company, Inc., of San Antonio, Martin/Martin of Denver, and Structural Engineering Associates of San Antonio.

The footprint of the stadium covers approximately 9.4 acres and is 561 ft. by 729 ft. in plan. The stadium is composed of cast-in-place concrete skip-pan joists spanning 42 ft. between bents that support concourse floors and cast-in-place raker beams that support the precast concrete seating deck. The concrete frame is separated into eight segments by expansion joints occurring two to a side. There are extended through the frame and precast seating sections, but terminate at the roof eave beams so that the roof is not broken by joints. Across the three center bays of each stadium facade, steel X-bracing members transfer lateral forces from the roof and stadium to the foundation.

The roof frame is supported on the perimeter eave beams with slide bearings occurring every 21 ft. These allow the roof to expand and contract in directions parallel and perpendicular to the supporting beam. However, directly over each of the three bay exterior X-bracing, relative motion parallel to the eave beams is denied by slide blocks to allow transfer of the lateral forces to the ground.

The utilization of the stay cable system resulted in reducing the overall span of the roof system. Four roof suspension points are supported by stay cable assemblies extending to four 306-ft. tall concrete masts. The suspension points are elevated by a compression strut extending from the "hip" of the mast. The masts, located near the four corners of the stadium, are then tied back with twin rear supporting cables that extend to twin steel compression outriggers, which also extend from the mast hips. From the outriggers, cables extend vertically down to a tie-down structure anchored by prestressed, drilled and underreamed piers.

By providing four interior suspension points, the roof spans were significantly reduced—from 561 ft. to 210 ft. and from 729 ft. to 378 ft. Manufactured steel joist trusses from AISC-associate member Vulcraft, 10-ft. deep-, span from supporting perimeter steel framing to four interior bowstring trusses, which in turn span between suspension points. Attached to the joist is 3-in. acoustical metal decking, which is utilized as the main diaphragm element of the roof.

The masts are 15-ft.-wide octagonal, hollow concrete sections extending 306-ft. above the field level floor. Each stay cable

In the photo above, the west main truss is already in place and the east main truss is ready for lifting.

assembly is made up of four individual steel pipes ranging in size from 5 to 6 in. in diameter. Within each pipe, various numbers of 0.6-in. prestressing strands are inserted and anchored with proprietary cable anchors. Cables were sized from tensile forces determined by the STRUDL model. Small, future adjustments can be achieved by rejacking and adjusting the prestress cable-head shims. Also, if future conditions require it, complete replacement, one at a time, is possible since the roof deadload can be adequately carried by only three of the four cables.

BOWSTRING TRUSSES

The flat roof is supported by four bowstring trusses, whose boxed post ends rest on the suspension seats and form a rectangular frame. Closing in this frame are long-span joists. During construction, and until all permanent roof dead loads are in place, the ends of the trusses at the post are allowed to rotate upon a 30-in.-by-30-in.-by 3-in. bearing pad. After dead loads are in place and the suspension points brought to their correct elevations, the post is welded to 4-in. continuity plates tied into the suspension connection assembly.

The two main roof bowstring trusses are each 378-ft. long and 52-ft. deep. Each weigh 200 tons and cost approximately $1 million. The top chords were fabricated from W36x720 and W36x650 Grade 50 jumbo sections and were rolled by AISC-associate member TradeARBED in Belgium and shipped to San Antonio in 60-ft. lengths. Fabrication of the truss took place in a vertical position on scaffolds that provided the needed camber. The bottom chord is composed of three 4-in. diameter wire ropes, the verticals are 12-in. and 16-in. diameter pipes and the diagonals are $1^1/_4$-in. diameter wire ropes. More camber than required was provided in the top chord to make the attachment of the bottom chord wire ropes to end fixtures easier.

The connection from the bottom chord to vertical pipe consists of twin steel plate saddles, milled with grooves to receive the strand and then clamped upon the strands with ASTM A490 bolts, torqued to 70% of specified minimum tensile strength. This clamping force provides the friction necessary to maintain the vertical pipes from slipping along the bridge strand.

As one can imagine, the contractor was very cautious in lifting such important and expensive trusses. General contractor was LYDA, Inc., San Antonio, and erector was the Dallas office of John F. Beasley Construction Co.Cutler-Galloway Services were retained as consulting engineers for the erection details. Preparation to lift the first truss took two months but the actual lift took only 28 minutes using two Manitowac cranes in ringer configuration 30 ft. from each end and one Manitowac 4100 crane conventionally configured at the center of the truss. The end cranes did the actual lifting while the center crane maintained 27 kips of force, that, through a series of diagonal cables, impaired 1 kip of lateral force at the bottom chord to offset the reaction of a horizontal movement caused by the 90 plus degree temperature. The truss itself was laterally braced with an auxiliary bow string truss each side attached to the top chord. The bottom chord was diagonally braced by tension cables running from the top bowstring outriggers.

The longer truss carries the majority of the roof load. Its dead load deflection is remedied by cambering $17^1/_4$ in. Total design load deflection amounts to 36 in. at mid-span with the top chord taking approximately 2,700 kips compression and 3,200 ft.-kips of moment. The bottom chord strand takes 2,270 kips of tension.

Major challenges faced during the construction included: record rainfalls that created staging difficulties and created stability problems for the heavy lift cranes; multiple prime contracts on site that resulted in the need for tight coordination of project access, work sequence and logistics; the instability of the steel framing until completion of erection, which resulted in the need to provide numerous guys and braces to stabilize framing until it was tied in with other members; and the temperature changes, which made it difficult to monitor and stabilize some of the long, massive members.

Roof Framing System

The roof system is composed of 10-ft.-deep joist trusses spanning 174 ft. and 210 ft. These rest upon the interior bowstring trusses with traditional top chord seats and are bottom chord supported at the exterior eave beams. Catwalks extend throughout the roof upon the bottom chords to allow access to suspended equipment and to support stadium lights. The joist trusses were designed by proprietary manufacturers based on numerous load combinations spelled out in the design documents. The roof structure is designed to support a 60 kip Jumbotron, a 90 kip scoreboard, and 162 kips for sound and lighting hanging loads in various locations within the central rectangle formed by the four major trusses. Outside of that area, the included longspan joists are designed to support an additional 4,000 lbs. of load at each panel point to provide upmost flexibility for the hanging loads required by a myriad of stadium functions.

Construction cost on the project was $107 million, only 2.15% above the original budget—which was based on an air supported roof structure.

H. Douglas Steadman, P.E., is a consultant and John Wall, P.E., is senior vice president with W.E. Simpson Company, Inc., consulting engineers/architects.

Construction Track | Engineering Track—Management | Engineering Track—Technical | Fabricator Track

1995 NSCC Schedule

MONDAY MAY 15, 1995

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

WEDNESDAY MAY 17, 1995

8:00 - 9:45 a.m. — Educator Program

10:00 a.m.- Noon — SPECIAL SESSION
Seismic Design After Northridge
Speaker: **TBA**

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

1:15 - 2:00 p.m. — GENERAL SESSION
Visual Weld Inspection
Speaker: **Duane Miller**, *Lincoln Electric Co., Cleveland, OH*

2:00 - 3:00 p.m. — GENERAL SESSION
Team Building/Partnering
Speakers: **Jim Scotti**, *Brown and Root*
Peter Van Nort, *H.B. Zachry*

3:00 - 6:00 p.m. — **Exhibits Open**

5:45 - 6:30 p.m. — Exhibitor Workshops

6:30 - 8:30 p.m. — AISC Welcome Reception

THURSDAY MAY 18, 1995

7:00 - 8:00 a.m. — Speaker Breakfast

7:30 - 8:30 a.m. — **Exhibits Open**

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

8:30 - 10:00 a.m. — Technical Sessions
1. Team Building/Partnering
5. Paintings/Coatings/ Avoiding Field Failures/ Shop Economics
6. Legislation - OSHA
14. Steel Erection Awareness
18. Industrial Buildings
21. Developments in Structural Tubes

10:00 a.m. - 3:00 p.m. — **Exhibits open**

10:00 - 10:45 a.m. — Coffee Break in Exhibit Hall

10:45 a.m. - 12:15 p.m. — Technical Sessions
2. Constructability
4. Quality Management
7. Legislation - EPA
9. Hollow Structural Section Fabrication
12. Protecting Your Firm from Lawsuits
16. Reducing Structural Steel Costs

12:00 - 1:30 p.m. — Lunch can be purchased in Exhibit Hall

1:30 - 3:00 p.m. — Technical Sessions
1R. Team Building/Partnering
10. Developments in Equipment and/or Methods
11. Flame Straightening Technology
13. Effective Project Specifications
17. Steel Construction in Mexico
20. Steel Moment Frames After Northridge

3:10 - 4:40 p.m. — Technical Sessions
3. Bar Coding
8. Reliability Based Maintenance/ Preventative Maintenance
15. Inspection of Welded and Bolted Joints

Construction Track | Engineering Track—Management | Engineering Track—Technical | Fabricator Track

1995 NSCC Schedule

18R. Industrial Buildings
19. What the Structural Engineer Should Know About Fracture Mechanics, Metallurgy and Weldability

5:00 - 5:45 p.m. — Exhibitor Workshops

6:00 - 10:00 p.m. — Optional Conference Dinner

FRIDAY MAY 19, 1995

7:30 - 8:30 a.m. — **Exhibits Open**

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

8:30 - 9:30 a.m. — GENERAL SESSION
T R. Higgins Lecture
Winner to be announced.

9:00 a.m. - 1:00 p.m. — **Exhibits Open**

9:15 - 10:15 a.m. — Coffee Break in Exhibit Hall

10:15 - 11:45 a.m. — Technical Sessions
- **3R.** Bar Coding
- **7R.** Legislation - EPA
- **9R.** Hollow Structural Section Fabrication
- **13R.** Effective Project Specifications
- **15R.** Inspection of Welded and Bolted Joints
- **16R.** Reducing Structural Steel Costs

11:30 a.m.- 1:00 p.m. — Lunch can be purchased in Exhibit Hall

1:00 - 2:30 p.m. — Technical Sessions
- 2R. Constructability
- 6R. Legislation - OSHA
- 10R. Developments in Equipment and/or Methods
- 12R. Protecting Your Firm from Lawsuits
- 19R. What the Structural Engineer Should Know About Fracture Mechanics, Metalurgy and Weldability

2:40 - 4:10 p.m. — Technical Sessions
- **5R.** Paintings/Coatings/ Avoiding Field Failures/ Shop Economics
- **11R.** Flame Straightening Technology
- **14R.** Steel Erection Awareness
- **20R.** Steel Moment Frames After Northridge
- **21R.** Developments in Structural Tubes

Special Bracing Short Course

Sponsored by AISC & SSRC

FRIDAY MAY 19, 1995

5:30 - 9:00 p.m. — Session I

SATURDAY MAY 20, 1995

8:30 - 11:30 a.m. — Session II

Faculty: **Joseph A. Yura,** *University of Texas-Austin*
Todd Helwig, *University of Houston*

Course Outline:
1. Column & Frame Bracing
2. Lean-On Systems
3. Torsional Bracing
4. Beam Buckling
5. Beam Bracing

Cost: $175 for AISC/SSRC Members
$200 for non-members
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Steel Stands For The Future

AISC's National Steel Construction Conference is the paramount industry event for everyone involved in the fabricated structural steel industry—structural engineers, fabricators, contractors & construction managers, erectors, detailers, educators, researchers and architects.The conference offers a wide range of practical seminars and also includes an extensive product exhibit hall—including numerous software and equipment demonstrations. Exhibitors include manufacturers of: fabricating equipment; bolts; fasteners; paints and coatings; and computer software & hardware.

In addition to staying current with today's state-of-the-art design and construction practice, the NSCC offers industry professionals an ideal setting for networking. Whether it's the opportunity to informally meet with other professionals, or the chance to make important future business contacts, the conference provides a perfect locale. And, of course, CEU credits are given for seminar attendance.

This year's conference is being held in San Antonio, America's tenth largest city. As always, a complete guest program is offered, including city tours and museum visits. Popular local attractions include: The Alamo; Arneson River Theatre; Brackenridge Park and Japanese Tea Gardens; Cowboy Museum; Market Square; McNay Art Museum; Riverwalk; Mexico Cultural Institute; Southwest Craft Center; Saltzberg Circus Collection; Institute of Texan Cultures; Fiesta Texas; San Antonio Botanical Gardens; Plaza Theatre of Wax; Spanish Governor's Palace; Splashtown Water Park; Vietnam War Memorial; Witte Museum of Natural History; San Antonio Zoo; and Sea World of Texas. For more information on any of these attractions, contact the San Antonio Convention & Visitors Bureau, P.O. Box 2277, San Antonio, TX 78298; phone: 210/270-8700 or 800/447-3372; fax 210/270-8782.

New This Year

For the first time, the NSCC is offering four separate education tracks for attendees: **structural engineering**; **engineering management**; **steel fabrication**; and **construction management**. While various seminars have been assigned to each track, attendees can attend seminars from different tracks at no additional charge.

Seminars include such topics as:

- How to reduce structural steel costs
- New concepts in industrial building design
- Designing connections for structural tubing
- How to protect your firm from lawsuits
- Correct inspection of welded & bolted joints
- Avoiding field painting failures
- Improved fabrication equipment & methods
- New flame straightening technology
- Capitalizing on team building & partnering
- Using bar coding for material management

Bracing Short Course

In addition to conference events, a bracing course will be held Friday evening and Saturday morning. AISC and the Structural Stability Research Council are sponsoring ***The Bracing of Steel Structures***. The course includes five sections: Column & Frame Bracing; Lean-On Systems; Torsional Bracing; Beam Buckling; and Beam Bracing. Faculty for the course includes Joseph A. Yura of the University of Texas-Austin, and Todd Helwig of the University of Houston. Cost for SSRC or AISC members is $175; non-member price is $200. For more information contact: Robert Lorenz, AISC Director of Education at 312/670-5406 or fax him a note at 312/670-5403.

NSCC Co-Sponsors

Co-sponsors this year include:

- American Galvanizers Association
- American Iron and Steel Institute
- American Society of Civil Engineers
- American Welding Institute
- American Welding Society
- Canadian Institute of Steel Construction
- Construction Industry Institute
- Council of American Structural Engineers
- Edison Welding Institute
- 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
- Steel Tube Institute of North America
- Structural Engineers Association of Texas
- Texas Structural Steel Institute

Simultaneous Spanish Language Translation

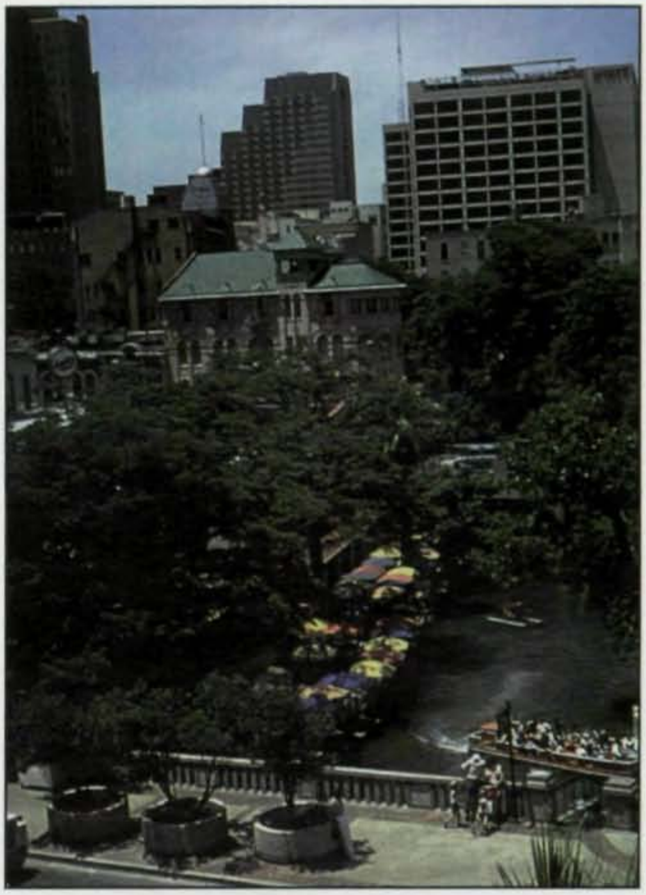
All general sessions (including the special session on *Beyond Northridge* and the *T.R. Higgins* lecture), plus selected technical sessions, will be simultaneously translated to Spanish as a courtesy to our Spanish-speaking attendees.

Local Travel

To get from the airport to either convention hotel, either take a taxi (approximately 8 miles and about $12 each way) or take the Star Shuttle which runs continuously and costs $6 each way (call 210/341-6000 for more information). All convention activities will be held at the San Antonio Convention Center located at Market and South Alamo Streets.

Climate/Clothing

San Antonio's average May temperature is 86 degrees F in the daytime, dropping to a comfortable 66 degrees F in the evening. The dress code is medium, casual and comfortable during the day, and casual or sportswear for the evening.



Hotel Reservations

Blocks of rooms have been temporarily reserved for NSCC attendees at two hotels close to the convention center. To receive the special convention rate, identify yourself as an attendee of the AISC National Steel Construction Conference when making your reservations. ***Reservations must be made by April 16; after that date, reservations on space available basis only.***

Convention Headquarters Hotel:

Hilton Palacio del Rio
200 South Alamo
Tel: 210/222-1400
Fax: 210/270-0793
Rates: $119 single
$139 double

Convention Hotel:

Marriott Riverwalk
711 E. Riverwalk
Tel: 210/224-4555
800/648-4462
Fax: 210/554-6248
Rates: $139 single
$159 double

1995 NSCC Registration Form

Conference Attendee

(Registration for attending seminars & trade show)

Please circle as appropriate:

Full Registration (includes AISC reception, exhibit hall pass and all seminars--except bracing short course; lunch extra):

AISC Member Fee *(AISC Active, Associate and Professional Members)*	**$270**
Non-Member Fee	**$320**
Educator Fee *(Employed full-time at accredited architectural or engineering school)*	**$ 90**
Student Fee	**$ 50**
First Exhibitor Fee	**No Charge**
Each Additional Exhibitor ***(max. three per 10x10 booth)***	**$ 85**

Partial Registration (half-day or full-day)

Half-day — Circle one:

Thursday morning	**$ 80**
Thursday afternoon	**$ 80**
Friday morning	**$ 80**
Friday afternoon	**$ 80**

Full-day — Circle one:

Wednesday	**$ 90**
Thursday	**$150**
Friday	**$150**

Exhibit Hall Pass *(Included free with registrations)*	**$ 5**

TOTAL ATTENDEE REGISTRATION FEES: ____________

Guest Tours & Special Events

(Registration for people interested in attending the bracing short course and optional & guest events)

Bracing Short Course *(AISC/SSRC member)*	**$175**
Bracing Short Course (non-member)	**$200**
Special Guest Registration (please circle) *(Includes AISC welcome reception, exhibit hall pass and one guest program event on both Thursday & Friday — please indicate below which events are requested)*	**$100**

Event	*# Tickets*		*Total Price*
SE1: AISC Welcome Reception *(Wednesday at 6:30 p.m. — included free with both Full Registration and Special Guest Registration)*	____	@ $25	____
SE2: Conference Dinner *(Thursday at 6 p.m.)*	____	@ $39	____
GT1:Half-Day City Tour *(Thursday a.m.)*	____	@ $24	____
GT2:All-Day City Tour *(Thursday)*	____	@ $51	____
GT3:Remember the Alamo *(Friday a.m.)*	____	@ $23	____
GT4:Europe Texas Style *(Friday a.m.)*	____	@ $24	____
GT5 Nuevo Laredo, Mexico *(Friday)*	____	@ $44	____

TOTAL GUEST & SPECIAL EVENT FEES: ____________

PLEASE REGISTER (type or print)

Name ______________________ Nickname (for badge) ______________

Company ______________________ Title ______________

AISC Member (circle one): yes no

Mailing Address ______________________ City/State/Zip ______________

Bus. Phone ____________ Fax ____________ Home Phone ____________

Guest Name ______________________ Nickname (for badge) ______________

Conference Fees Payable

Registration Fee: $ ________

Guest Fee: $ ________

Add'l. Guest & Special Events: $ ________

TOTAL: $ ________

Mail completed form and fees to:
American Institute of Steel Construction
1995 NSCC
P.O. BOX 806286
CHICAGO, IL 60680-4124
ph: 312/670-2400 fax: 312/670-5403

Circle One:

Check enclosed (US funds) payable to AISC

Please charge my credit card (VISA or Mastercard only)

\# ______________________

circle one: VISA MASTERCARD

Signature: ______________

Expiration Date: ______________

Name on Card: ______________

Serviceability Concerns In Multi-Use Structures

Vibration played a critical role in the design of the Meydenbauer Center in Bellevue, WA

Photo by James F. Housel

By Dan Symonds, P.E.

WHILE VIBRATION IS ALWAYS A CONCERN IN LONG-SPAN STRUCTURES, the addition of a ballroom in Meydenbauer Center made serviceability a crucial consideration. The convention facility, located in Bellevue, WA, includes a full production theater, 35,000-sq.-ft. column-free exhibition space, banquet facility, meeting rooms and the aforementioned ballroom—all built over three-levels of below grade parking.

The parking levels are post-tensioned flat slabs on a 30-ft. grid system, while the exhibition hall floor is a reinforced concrete beam slab system designed to carry HS-20 loads. The meeting level—including the ballroom and kitchen—is supported on 18-ft. deep steel trusses, 30-ft. on center and spanning 170 ft. Separating the exhibition hall from the meeting rooms is an atrium with escalators and walkway links to the theater and grand entrance, which includes a 70-ft. glass wall entry.

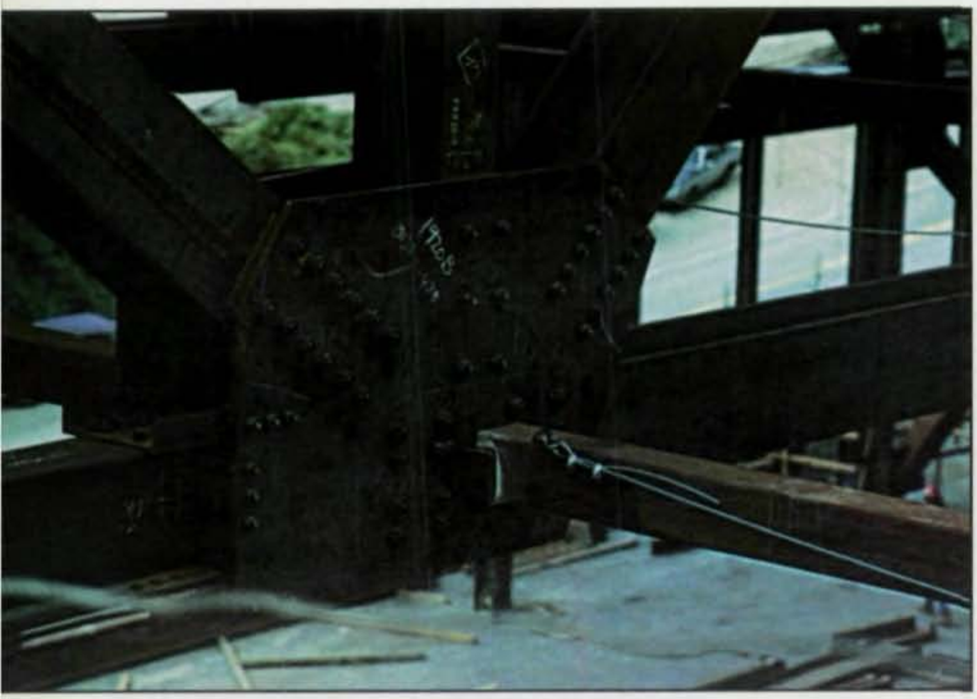
Floor Vibration

Above the parking level are steel columns, ranging in size from W14x120 to W14x500. The exhibition hall is spanned by five trusses at 30-ft. on center. These 18-ft.-deep steel trusses support occupied floors at both their top and bottom chords. The areas supported combine a wide variety of activities, ranging from dancing on the floor supported by the top chord to office space

on the floor supported by the bottom chord. The top chord consists of W14x665 shapes, while the bottom chord consists of W14x455 shapes. Vertical and diagonal members also are W14 shapes. All of the connections are bolted using A490 high-strength fasteners.

The varied uses created an unusual serviceability problem. Rhythmic dancing of a large group of people on a big truss is analogous to a single person walking on a floor beam. The masses and stiffness are simply larger. Still, various convention type facilities across the country have received complaints due to human activity induced vibrations in long-span structures. Preliminary investigation of the initial design for Meydenbauer Center revealed a potential problem. Analysis revealed that 150 people dancing would produce an acceleration of 0.7% of gravity. While this would be barely noticeable to the dancers and not objectionable to a nearby audi-

Shown on top is the main atrium between the theater and the exhibit hall. Pictured above is the braced frame. Note the large amount of steel girders on the exhibit hall floor slab. This is an example where construction loadings may be the highest a floor will receive. Photo, top left, by James F. Housel; photos, top right and above, by Steven G. Hall



Shown is a closeup view of the first bottom chord connection out from truss to column top chord connection.Photo by Steven G. Hall

ence, it would be perceptible and distracting to office workers on the floor level supported on the bottom chord of the trusses.

The designers at HNTB briefly considered the possibility of having coordinating schedules so that such "incompatible" uses did not occur. Obviously, though, this alternative was much too constraining. Instead, the decision was made to modify the structure.

However, changing the stiffness or mass would not be practical since any changes would have to be large because the accelerations only change with the square root of these properties. Instead, a modification was designed that linked the trusses together at mid-span, similar to bridging used on open web joists.

The X bracing bridging was constructed out of 10-in. steel pipes. These pipes coupled the trusses together dynamically, increasing the number of trusses effectively resisting the vibration. This reduced the acceleration from 0.7% to 0.2% for moderate sized groups. This acceleration is at the lower limit of human acceptability and, therefore, was deemed acceptable. The bridging added approximately 10 tons of steel to the job.

The roof is also supported on trusses, though these smaller trusses have WT6x95 top chords and WT6x85 bottom chords with double angles used for the diagonals and tubes for the vertical members. The roof truss is 14-ft. deep at its midspan and 9-ft. deep at each end.

Lateral Force Resisting System

The exhibition hall structure consists of the eastern portion of the complex and is comprised of the parking structure, exhibition level, meeting rooms and the exhibition hall roof. This portion

Shown is a closeup view of a braced frame. The brace members were typically slotted, held in place with erection bolts, and field welded to gusset plates. Photo by Steven G. Hall

is generally separated from the theater portion by an isolation joint that separates the two structures such that they will act independently when responding to seismic excitation.

This isolation is not complete at the lower levels in that the theater entry level is connected to both structures. In addition, the columns and braced frames of the theater structure at that location are supported by a shear wall that forms the western boundary of the exhibition hall structure. Because these linkages are near the base of the structure, as well as the fact that the structure's mass associated with them is small, their effects on the seismic response is minor and has been included in the design. Typically, the floor systems are a concrete slab supported by a 3-in. composite metal deck, for a total depth of $5^1/_2$ in. The slab system is supported by composite steel beams and girders. Columns are wide flange or tubular shapes. The roof system is comprised of steel beams supporting an open metal deck (no concrete fill), which acts as a diaphragm. The exhibition floor structure is reinforced concrete beams and slab.

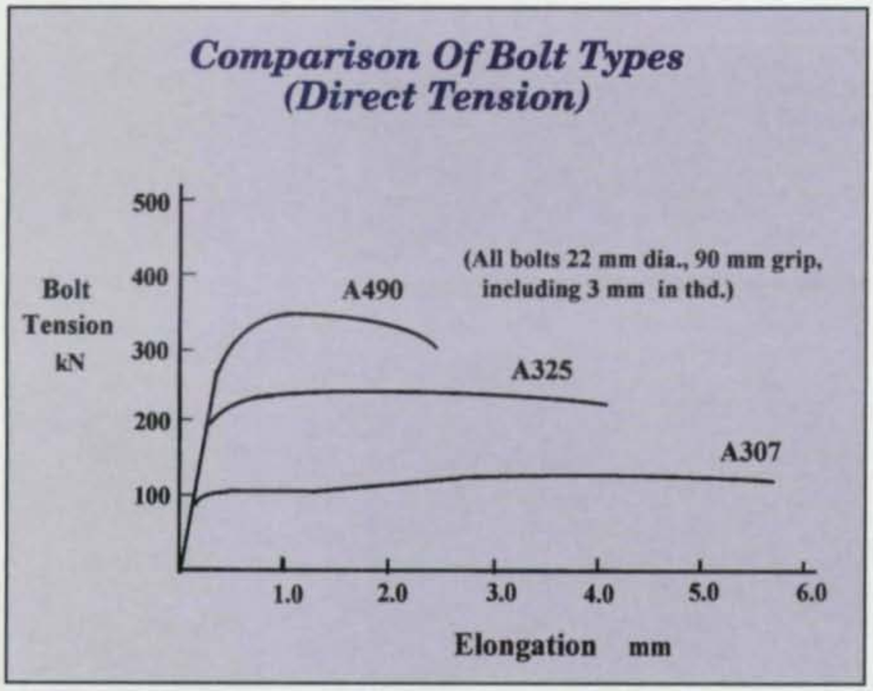
The exhibition hall is essentially a large box and the lateral support system consists of X-bracing, primarily composed of square tubes, though some round tubes were used. Fabricator on the project was AISC-member Canron West and steel erector was The Erector, Seattle.

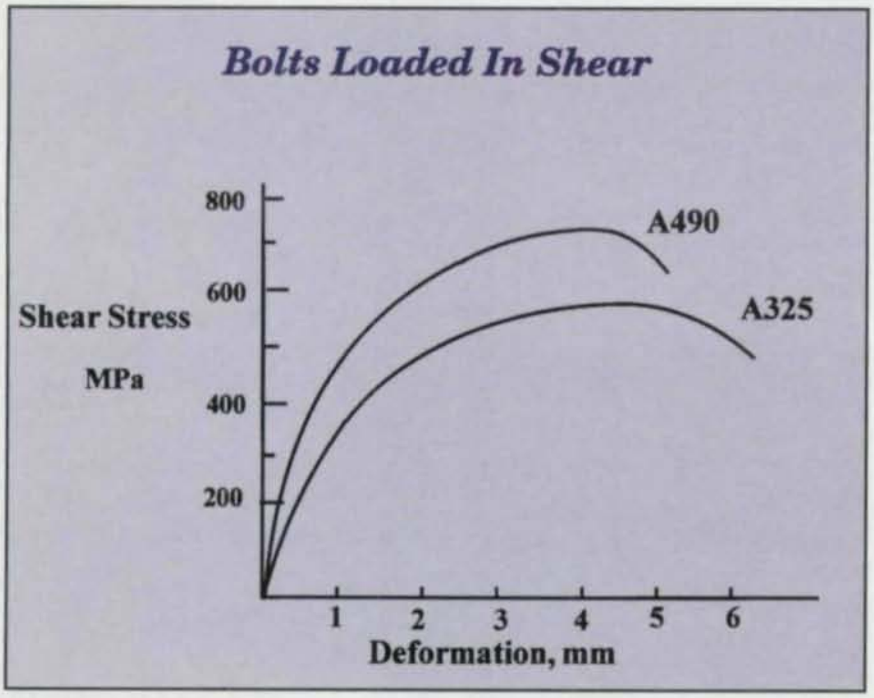
Dan Symonds, P.E., is a project engineer with HNTB's Bellevue office.

Please circle # 57

HIGH-STRENGTH BOLTS

A primer on the installation, inspection and behavior of high-strength bolts for engineers who missed last year's NSCC bolting lecture





By Geoffrey L. Kulak

WHILE WELDED CONNECTIONS OFTEN REQUIRE THE SERVICES OF A SPECIALIST, PROJECT ENGINEERS ARE EXPECTED TO BE CONVERSANT WITH ALL ASPECTS OF DESIGN, INSTALLATION AND INSPECTION OF BOLTED CONNECTIONS—from ASTM A307 (common or "ordinary" bolts) to ASTM A325 and ASTM A490 (high-strength bolts).

ASTM A307 bolts are used on lightly loaded structures and can be installed with a spud wrench. While the induced level of pretension will be indeterminate, it can be expected to be low. The bolt material has an ultimate strength of 60 ksi (450 MPa). In contrast, ASTM A325 and ASTM A490 bolts are much stronger than A307 bolts, with ultimate tensile strengths of 120 ksi (830 MPa) and 150 ksi (1040 MPa), respectively.

SLIP CRITICAL CONNECTIONS

Slip-critical connections are specified when:

- Load is repetitive and changes from tension to compression (fatigue by fretting could occur).
- Change in geometry and structure would affect its performance.
- High-strength bolts and welds must share the load (new work only).
- Slotted holes are used in the direction of loading.

In ASD, engineers use service loads, not factored loads, while in the latest LRFD Specification, engineers can use service loads or factored loads. However, the most common application of slip-critical connections is for bridges. In this case, designers should probably use the fatigue truck load or an overload truck.

In a slip-critical connection, if the connected parts slip, the amount of slip can theoretically be as much as two hole clearances. However, both laboratory test data and field experience show that if joint slip takes place, it is usually in the order of one-half of the hole clearance.

From the point of view of inspection of high-strength bolts in a slip-critical connection, it is important to recognize that the preload in the bolt is essential to proper functioning of the joint and that the bolts are not acting in shear.

When designing slip-critical connections, remember:

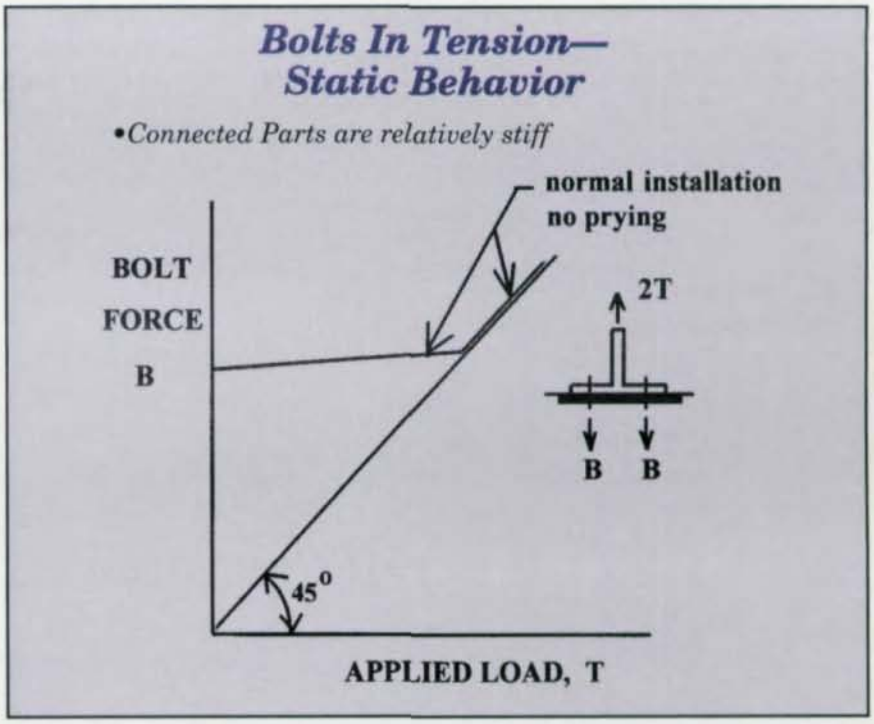
- AISC gives permissible shear stresses for bolts in a slip-critical connection (even though the bolts are actually not acting in shear).
- There is about a 5% probability of slip (clean mill scale case) when these rules are used and installation is by turn-of-nut.
- There is about a 10% probability of slip for the same case but when installation is by calibrated wrench.

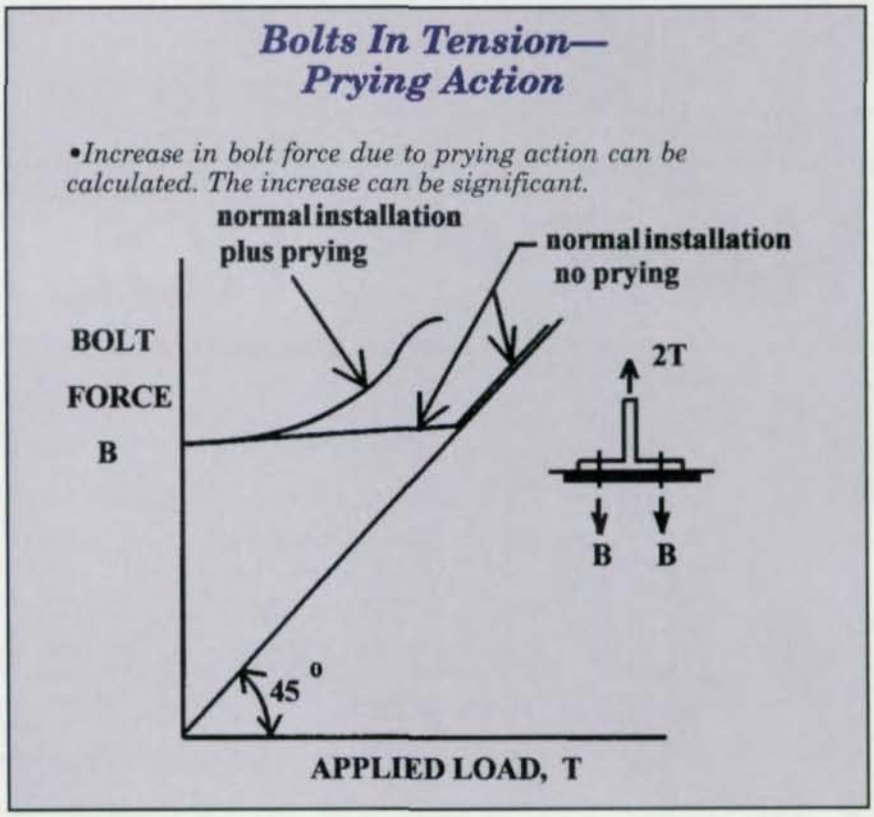
Bearing-Type Connections

In bearing-type connections, the bolts are in shear and their capacity must be examined. The connected material is in bearing adjacent to the bolts and its strength must be examined.

Bolt shear strength is about 0.62 times bolt tensile strength and AISC values reflect this relationship. When designing bearing-type connections, engineers should consider shear plane through threads, if necessary, and the effect of joint length, if necessary.

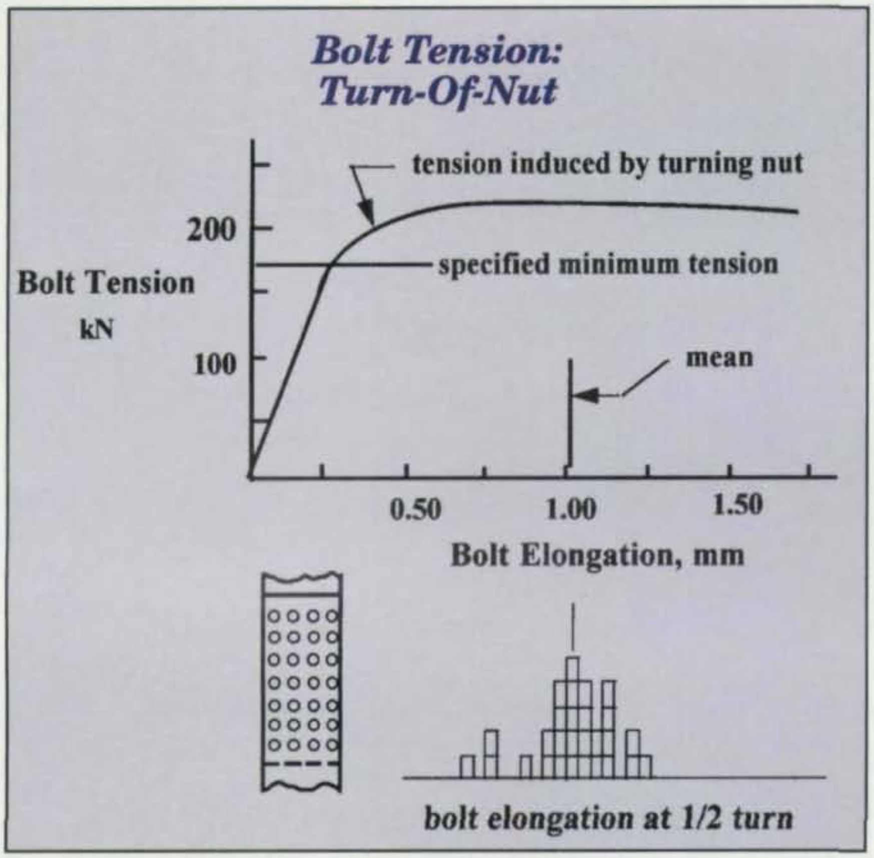
When designing bearing-type connections it is important to recognize that the shear strength of the bolt is not dependent on the pretension of the bolt.





Bolts In Tension

With bolts in tension, the connected parts are compressed by the preload supplied by the bolts and there is initial tension and equal and opposite precompression in the connected parts. When an external load is applied, the bolt elongates, but contact pressure between the connected material decreases. In both theory and testing, the bolt load increase is less than about 10%, as long as parts do not separate. After separation, bolts must carry all of the external



load.

Prying action in bolts is greater than the nominal value of F/n. Also, local bending takes place at the shank/bolt head junction. With bolts in tension, the increase in bolt force due to prying, which can be significant, can be calculated.

The capacity of a bolt in tension is a product of the ultimate tensile strength of the bolt and the tensile stress area of the bolt. AISC Specifications directly reflect this calculated capacity.

INSTALLATION OF HIGH-STRENGTH BOLTS

Regardless of the type of installation method used, preload is required for:

- Bolts in slip-critical connection.
- Column splices and certain girder splices in high-rise buildings.
- Bolts in tension connections.

Preload is not required when

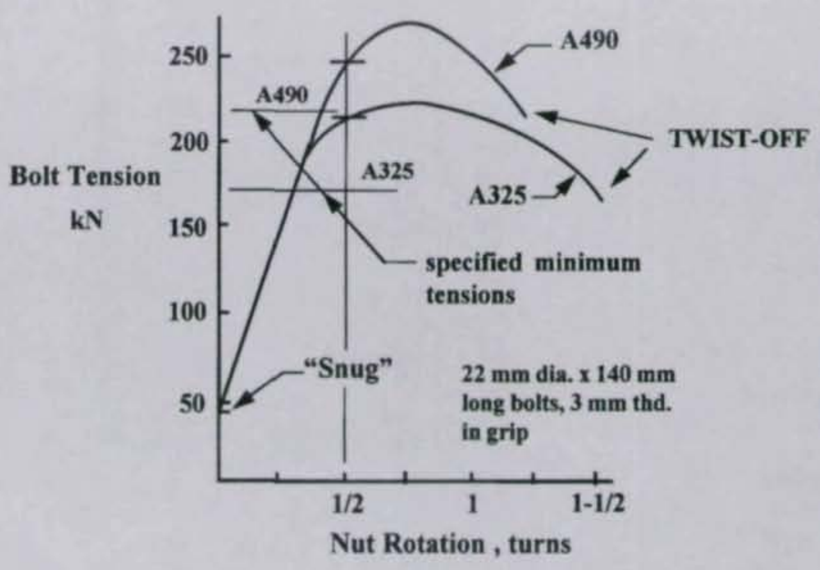
snug-tight bolts are sufficient, which is typical in the majority of applications.

With pretensioned bolts, preload must be at least 70% of the bolt minimum specified tensile strength. Permitted installation methods for pretensioned bolts are: turn-of-nut; calibrated wrench; alternative design bolts; or direct tension indicator tightening.

Turn-of-nut is one of the simplest and most common installation methods. A few bolts are installed and used to bring the part together such that the material is in close contact. The remaining bolts are installed to a snug-tight condition (full effort using a spud wrench or first impact of an air wrench). And finally, each nut is turned the prescribed part (e.g., one-half turn), working outward from the most rigid part of the joint.

Another common, though slightly more expensive, installa-

Turn-Of-Nut Method: Nut Rotation



tion method is the use of a calibrated wrench. The installer begins by calibrating the wrench using at least three bolts of a given grade, diameter and length, which are installed in a device that reads bolt force (Skidmore-Wilhem). The wrench is then adjusted to stall at 5% over the specified minimum preload. In both the test and on the job a washer must be used.

It is important to recognize that installation using a calibrated wrench is a torque control method. This means that if any of the conditions that can affect the pretension change (condition of the threads or the washers, changes in bolt diameter or length, or bolt lot), then a new calibration must be done.

In addition to turn-of-nut and calibrated wrench bolt installation, there are a number of proprietary methods on the market. These include: Huck bolts; tension-control bolts (twist-off bolts); and load-indicating washers. All of these methods are directed toward establishing the specified minimum preload in the bolt. Of course, prior to specifying one of these methods, it should first be determined whether or not pretensioned bolts are required for the project. Generally speaking, though, any of these methods will work satisfactorily if properly calibrated.

Field Inspection

Field inspection is necessary to ensure that installation requirements of the Specification are being met as the bolts are being installed. After installation, it becomes difficult, uncertain and incredibly time-consuming to determine whether correct bolt pretension has been achieved. However, during inspection, it is important to distinguish between the requirements necessary when bolts must be pretensioned and when bolt pretension is not required.

During inspection of snug-tight joints, the inspector should note that all plies must be in firm contact with no compressible material within grip. Inspection should also establish that all connected plies are in "full contact"(note that pretension is not an issue). However, notwithstanding the "full contact" statement, some incomplete contact can be tolerated since the strength of the joint is independent of bolt pretension and contact area.

In joints where pretension is required, a representative sample of bolts must first be installed in a bolt calibrator (regardless of installation method) to establish the validity of the procedure to be used. The requirements for inspection of a particular method follow from the qualification procedure. (For example, with turn-of-the-nut, does one-half turn actually produce the required pretension? Does the installation of the bolts proceed from the stiffest part of the joint outward?)

Points to consider about inspection:

- Pretension values greater than those specified are not cause for rejection.
- Rotation tests are useful for short-grip bolts or coated fasteners.
- Use of a torque wrench to install bolts is not permitted by AISC rules (the results are highly variable).
- Use of a torque wrench to check the pretension attained in a bolt is unreliable.
- Torque methods of installation (e.g., calibrated wrench and twist-off bolts) are sensitive to the quality of the lubrication, thread fit, dirt and grit on the job site, air pressure, etc. If these change, a new calibration is required.

Geoffrey L. Kulak is Professor of Civil Engineering at the University of Alberta in Edmonton, Canada and is a member of the Research Council on Structural Connections. This article is based on a paper delivered at the 1994 National Steel Construction Conference.

High Shear Nail

Hilti has recently introduced the High Shear Nail (HSN) for the attachment of metal roof and floor deck to bar joist and light structural steel beams. Designed to be equivalent in allowable shear capacity to a $^5/_8$-in. puddle weld, the nail is compatible with Hilti's DX 36M Powder Actuated Fastening System. Contact: Kenneth Walsh, Manager of Marketing Communications, Hilti Inc., 5400 South 122nd East Avenue, Tulsa, OK 74146 (800) 727-3427 ext. 6726 or **circle #96.**

Lock Pin & Collar

Huck International has introduced a high-strength alternative to ordinary nuts and bolts: The Huck Lock Pin and Collar fastening system. It exceeds ASTM A325 and A490 requirements and meets AASHTO, ICBO, AISC, and ASME requirements. The system is U.S. made, and the fasteners are completely traceable and factory mill certified. Contact: Mark Brenner, Huck Fastening Systems, 6 Thomas, Irvine, CA 92718 (714) 855-9000 or **circle #95.**

Direct Tension Indicators

Direct tension indicators (DTI) produced by J&M Turner are now identified with a new trademark symbol as well as a lot identification number for complete product traceability. In addition, a full range of metric DTIs are now available that meet the provisions of ASTM A325M and A490M. Contact: J&M Turner, 1310 Industrial Blvd., Southamton, PA 18966 (800) 525-7193; fax (215) 953-1125 or **circle #69.**

Structural Bolts

St. Louis Screw & Bolt specializes in manufacturing various types of structural bolts used in the construction of bridges, tall buildings, power plants and large manufacturing plants. The plant produces Heavy Hex Head Structural Bolts that are principally designed for use in turn-of-the-nut method of installation. Sizes range from $^1/_2$-in. through $1^1/_2$-in. diameter in all lengths, plain Type I and III, hot dipped galvanized and mechanical. The company also produces special head configurations such as countersunk, buttom head and square. For more information, contact: Janet Kruzs, St. Louis Screw & Bolt, 6900 N. Broadway, St. Louis, MO 63147 (800) 237-7059; fax (314) 389-7510 or **circle #36.**

Roll Threading

Fabsco Corp. has expanded its roll threading capacity to 3-in. diameter so there's no need to upset large diameter rods to achieve full size thread engagement. Time and money can be saved by roll threading sag rods and tie rods. Also stocked are $^5/_8$-in., $^3/_4$-in. and 1-in. diameter roll thread anchor bolts as well as a large range of plastic anchor bolts sleeves up to 3-in. diameter. The company also carries hot dip galvanized bridge bearing studs up to $1^1/_4$-in. diameter. For more information, contact: Fabsco Corp., 1745 West 124th St., Calumet Park, IL 60643 (708) 371-7500; Fax (708) 371-7524 or **circle #97.**

Anchor Bolts & Tie Rods

Haydon Bolts manufacturers anchor bolts and tie rods up to 4-in. in diameter. The products are offered in ASTM A36, A449, A325, and A588. A complete line of standard and special sizes of ASTM A325 and A490 are available in hexagon heads as well as button head shear bolts. For more information, contact: Haydon Bolts, Inc., Adams Avenue & Unity Street, Philadelphia, PA 19124-3196 (215) 537-8700; Fax (215) 537-5569 **or circle #98.**

Twist-Off Bolt

Nucor Fastener has developed a twist-off bolt with a major difference from competitive products: a hex head. This head makes the bolt easy to install, easy to remove, and easy to check its installation with a torque wrench. They are fully tested and certified, including full compliance with FHWA, DOT and AASHTO specifications for bridge construction. For more information, contact: Nucor Fastener, P.O. Box 6100, St. Joe, IN 46785 (800) 955-6826; (219) 337-5394 or **circle #38.**

Tension Control

Rapid Tension Bolts from NSS Industries are a type of structural bolt-nut-washer assembly where the proper tension is indicated when the spline has been separated from the bolt. Installation is achieved by first assembling the bolt, washer & nut into the connection hole and tightening by hand until snug. Next, the 12-point socket of the installation tool is engaged. Rapid tension is achieved by simply pressing the trigger of the installation tool; while the outer socket rotates clockwise, the inner socket rotates counterclockwise, ensuring a snug fit. Contact: NSS Industries, 9075 General Dr., Plymouth, MI 48170 (800) 221-5126 **or circle #99.**

Tension Control Indicator

The TS Fastening System is a high-strength tension control indicating system and is designed for the installation of high-strength A325 and A490 bolts.Contact: Bristol Machine Company, 19844 Quiroz Court, Walnut, CA 91789 (800) 798-9321 **or circle #100.**

Rapid Delivery

Mid-South Bolt & Screw Co. provides a wide range of products including anchor bolts, tie rods, studs, crane rail bolts, U-bolts, J-bolts, swedge bolts, eye bolts, structural hex bolts, tension control bolts, load indicator washers, concrete anchors, weld studs, threaded studs, plastic sleeves, clevises, turnbuckles and all types of nuts, screws, bolts and washers. Contact: Mid South Bolt & Screw Co., Inc., 499 Cave Road, Nashville, TN 37210 (800) 251-3520 **or circle #74.**

Proper Tension Installation

The LeJeune Tension Control Bolting System provides a calibrated bolt, nut and washer assembly that installs at the proper tension for that particular size and grade of bolt. A-325 Type I and III, as well as A-490, Grades are available in 3/4-in., $^{7}/_{8}$-in. and $1^{1}/_{8}$-in. diameters. Contact: LeJeune Bolt Co., 8330 West 220th St., Lakeville, MN 55044 (800) 872-2658 **or circle #**

Threaded Fasteners

Portland Bolt is an 82-year-old, domestic manufacturer of anchor rods, tie rods, high-strength head bolts up to $2^{1}/_{2}$-in., welded anchor bolt assemblies with common plates, and threaded rod. Portland Bolt produces product from $^{1}/_{2}$-in. to 4-in. diameter, in any grade, and up to 40-ft. long. All galvanizing can be done in-house. The company also offers specification compliance thorugh a the latest testing equipment. Contact: Portland Bolt & Manufacturing Co., Inc., 3441 N.W. Guam, Portland, OR 97210 (800) 547-6758, fax (503) 227-4634 **or circle #**

Bolt Storage

Intromark's new Size Master storage system makes inventory and storage of bolts easier and simpler. The color-coded storage and display system keeps nuts and bolts organized for easy selection. For more information, contact: Intromark, Inc., 217 Ninth St., Pittsburgh, PA 15222 (800) 851-6030; fax (412) 288-1354 **or circle #**

Portable Wire Feeder

Lincoln Eletric's new LN-25 Portable Wire Feeder is designed for maximum versatility and portability in a variety of field applications. It can be used on most constant voltage or constant current DC power sources and is primarily for "across-the-arc" operation without a control cable. Simply connect the electrode cable, clip it to work and it's ready to weld. Includes 2-in. O.D. spindle with Readi-Reel adapter for up to 30 lb. spools. The wire drive assembly features a quick release mechanism for easy changing of the electorde. The LN-25 is available in three models (with voltmeter; with voltmeter and gas solenoid; and with voltmeter, internal contractor and gas solenoid). For more information, contact: The Lincoln Electric Company, 22801 St. Clair Ave., Cleveland, OH 44117-1199 (800) 408-1188 ext. WELD **or circle #57.**

Inverters

PowCon's inverter welding *power sources allows up to* 75% reductions in the size and weight of the transformer and up to 45% energy savings as compared to SMAW or GTAW power supplies. When combined with an engineered plastic housing, the result is a power source that weighs only one-third that of a conventional transformer and takes up significantly less floor space, yet provides the same power output. PowCon, which also sells plasma cutting tools, has entered into a marketing agreement with Kemppi to sell their product line of inverters in the U.S. For more information, contact: Wade Chase, PowCon Inc., 8123 Miralani Dr., San Diego, CA 92126 (800) 833-9353 or **circle #103.**

Stud Welding

TRW Nelson Stud Welding Division offers a wide range of stud welded shear connectors, concrete anchors and threaded fasteners for bridge, building and industrial structures connection requirements. Complete stud welding systems and power generators are available for sales or lease. For more information, contact: TRW, 7900 West Ridge Road, P.O. Box 4019, Elyria, OH 44036-2019 (216) 329-0400 **or circle #104.**

Portable Flux Recovery

The Mighty-Might PS-2 and PS-2-A/P from Weld Engineering are inexpensive, protable and rugged flux recovery systems. These lightweight systems feature powerful electric or air vacuum systems to meet medium duty flux recovery needs. The PS-2 features a self-cleaning dust filter, while there are no internal filters in the PS-2-A/P, which allows hours of uninterrupted operation. Dust and fines are collected in a dust bag for easy cleaning. For more information, contact: Weld Engineering Co., 34 Fruit St., Shrewsbury, MA 01545 (508) 842-2224; fax (508) 842-3893 **or circle #105.**

Welding Power Supplies

Miller Electric offers a range of welding products from 600 amp engine-driven welding generators to hand-portable inverter-based welders to plasma cutters. In addition, a complete selection of factory and field options enables users to customize the power source to their application. For more information, contact: Miller Electric Mfg. Co., 1635 W. Spencer St., P.O. Box 1079, Appleton, WI 54912-1079 (414) 734-9821 **or circle #106.**

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