MODERN STEEL CONSTRUCTION

February 1994

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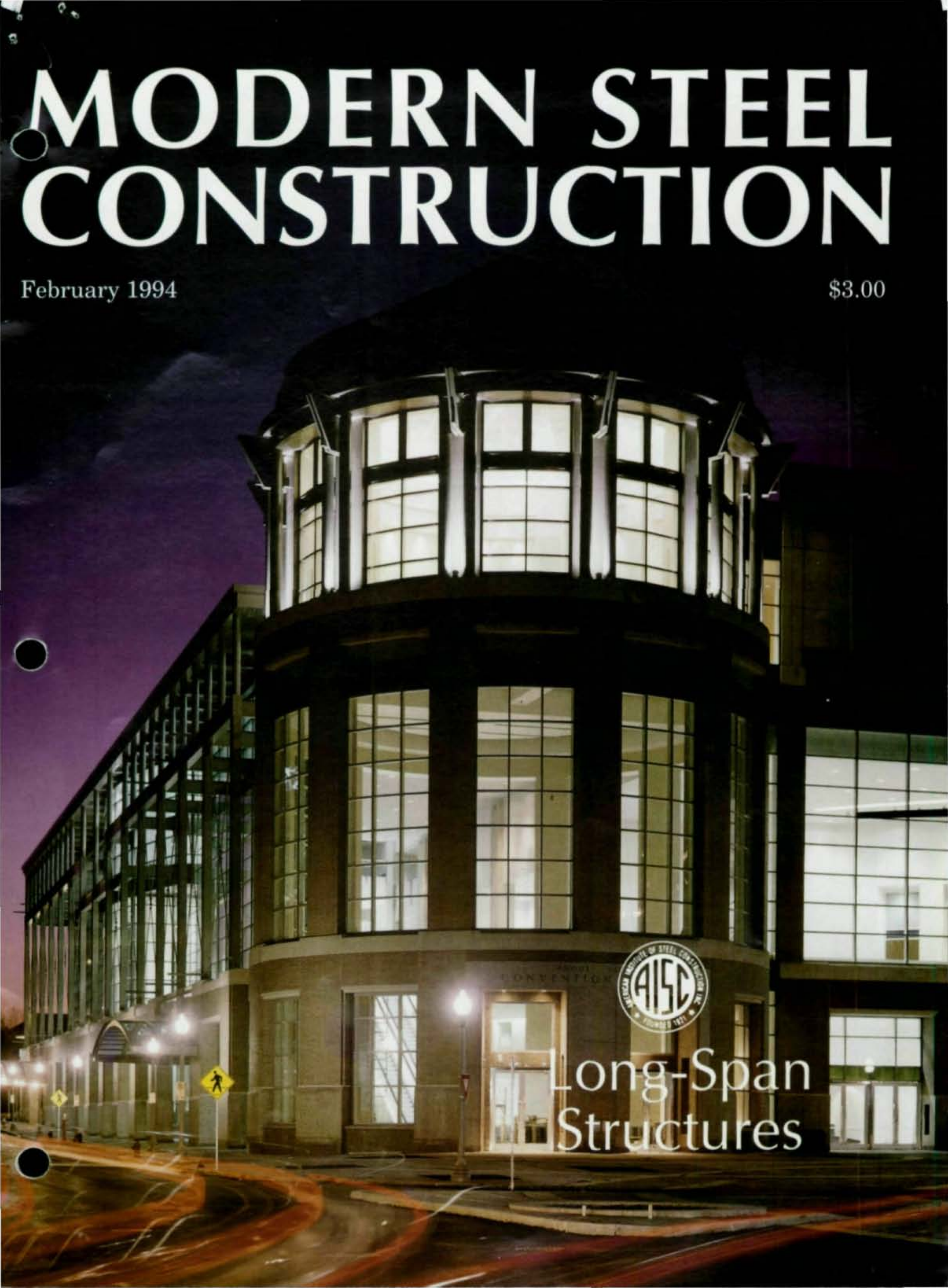

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JOIST DESIGN DATA SHEET

No. 2

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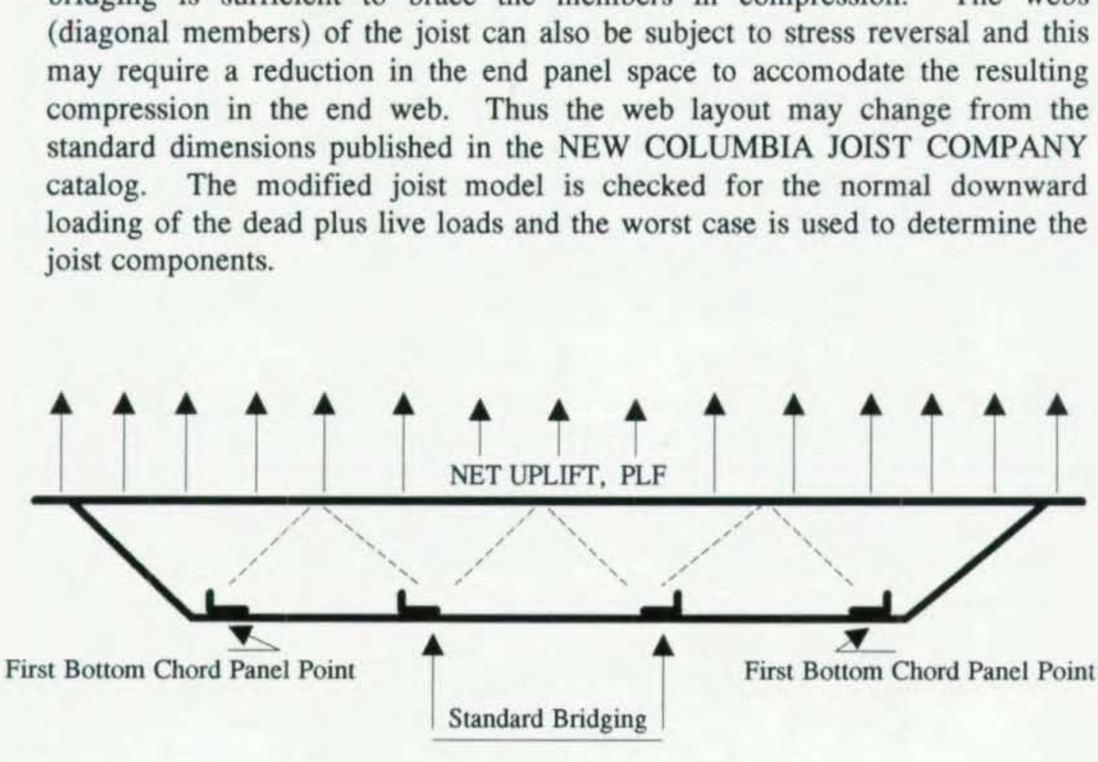
Wind uplift forces must be determined by the design professional and shown on the contract drawings as NET UPLIFT. (The net uplift force on the roof joist is the gross uplift minus the dead load including the joist weight.) This temporary reversal of loading creates compression forces in the bottom chord which, as a result, may require lateral bracing. The Steel Joist Institute (SJI) recognizes this by specifying a single line of bridging near the first bottom chord panel point to brace the bottom chord. The remainder of the bottom chord must be checked by the joist company (NCJ) to see if the SJI standard bridging is sufficient to brace the members in compression. The webs (diagonal members) of the joist can also be subject to stress reversal and this may require a reduction in the end panel space to accomodate the resulting compression in the end web. Thus the web layout may change from the standard dimensions published in the NEW COLUMBIA JOIST COMPANY catalog. The modified joist model is checked for the normal downward loading of the dead plus live loads and the worst case is used to determine the joist components.



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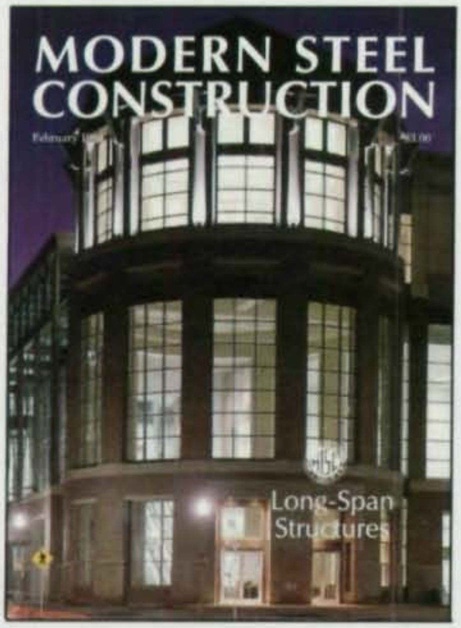

To meet site constraints, the Rhode Island Convention Center was designed with a vertical orientation. The story behind this unique structure begins on page 28.

FEATURES

DEPARTMENTS

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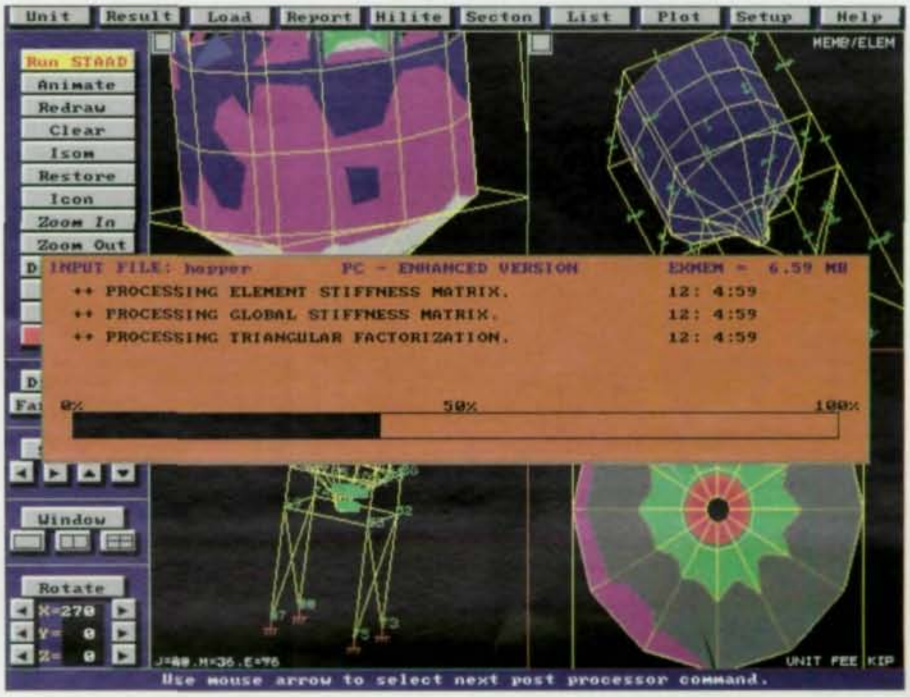

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Putting The Professional Back In P.E.

What do Iowa, Alabama, West Virginia and New Jersey all have in common? They're pioneers in requiring continuing education as a prerequisite for the professional licensing of engineers.

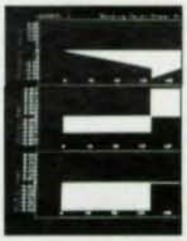
Hurray for them, and all the other state licensing boards which are considering a similar requirement (for more information, turn to page 12). Engineering is not a static profession and practicing engineers need to maintain their knowledge of the latest developments in their industry. With the privilege of approving design drawings comes the responsibility for maintaining their expertise.

The task of obtaining 15 hours of continuing education (the typical requirement) should be simple for most engineers, given the wide range of activities that meet the requirement: local or in- house seminars; attending conventions; speaking; writing papers; attending professional meetings; or, in some cases, taking a videotaped course. Engineers in Iowa and Alabama—the only two states with experience in requiring continuing education—report that obtaining the needed number of hours has not been a problem. For example, attending the National Steel Construction Conference in Pittsburgh this May can provide nearly all the required hours. Need more hours? AISC Seminars are being held this year throughout the country.

The requirement is not without potential pratfalls, however. Some engineers are hesitant to give the state the power to determine what courses are allowable. Others worry about adding another layer of government bureaucracy. Stories about problems in dealing with government agencies are certainly legion. For example, a close friend of mine recently had a run-in with the Illinois Department of Registration and Education—the agency that licenses engineers and, in her case, occupational therapists. Renewing her license should have been the simplest thing in the world. There's nothing to verify. All she needs to do is send in a $40 check before the end of the year. Well, that's not strictly true. She needs to send it in far enough in advance for the Illinois bureaucracy to handle it. Unfortunately, Illinois doesn't stagger its license renewal, so it takes at least eight weeks before they can even verify that they have received a check, let alone begin to process the renewal application. If this type of bureaucratic backlogging is commonplace, then engineers will have something to worry about. Hopefully, it's not—again, engineers and Iowa and Alabama do not report similar problems.

Continuing education is important. If engineering is to maintain its status, then individual practitioners must endeavor to fulfill the requirements of professionalism—and that includes staying up-to-date with the latest information in the engineering field. SM

How to get from here



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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 312/670-2400 ext. 433.

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

When asked to design a temporary bracing system for steel beams and columns during the erection phase of construction, what loads are used and what factors of safety are employed for the bracing and its connections?

The primary source for the loading requirements for temporary bracing is the AISC Code of Standard Practice Section 7.9.1 which states that the temporary supports are to be designed for "...loads comparable in intensity to those for which the structure was designed, resulting from wind, seismic forces and erection operations, but not loads resulting from the performance of work by or the acts of others, nor such unpredictable loads as those due to tornado, explosion or collision." It should be noted that steel frames are open structures and may have wind loads greater than predicted by the projected area of the completed building. Also partially completed cladding may add to loads in the supports for non-self-supporting structures. The topic of erection bracing and the loads applied to it is discussed in detail in a paper by Fisher and West published in the Proceedings of the 1993 SSRC Conference held in Milwaukee.

(Information on obtaining a copy of "Is Your Structure Suitably Braced" can be obtained from the Structural Stability Research Council by calling 215/758-3522.)
Michael A. West, P.E.
Computerized Structural Design
Milwaukee, WI

When is it appropriate to use clips instead of hook bolts to secure rails for top running crane runways? What are service versus cost considerations?

In general, hook bolts can be used for attaching rails for light duty, slow moving cranes and clips are used for heavy duty, faster cranes. the dividing point is arbitrary. However, hook bolts should never be used for heavy duty cranes as they are not strong enough to resist the lateral forces likely to be present. Clip plates, on the other hand, can be used for the full range of crane sizes but, for smaller cranes, it may be impossible to place the required gauges in the crane beam and cap channel. This is the situation where hook bolts can be used. There is a third option for attaching rails to crane beams—proprietary adjustable clip devices which are welded to the tops of the crane beams. Crane rails should never be welded directly to the crane beams, regardless of crane size.

When considering the economics of hook bolts vs. clips there is nothing to consider. One must do what must be done. If one owns a Ferrari, one should not worry about the cost of high octane gasoline. A crane runway is a dynamically loaded structure placed inside a statically loaded structure and must be treated as such. Hook bolts are usually a purchased item, the rails must be punched or drilled to receive them, and they must be monitored frequently to keep them tight and to assure proper alignment of the rails. Rail clips are easily fabricated by most fabricators, the crane beams must be punched or drilled, alignment is more positive, lock nuts or fully torqued high strength bolts are required.

For the sake of this discussion light duty could be interpreted as CMAA classes A2 and B; heavy duty as CMAA classes A1, C, D, E, and F. Each crane capacity has its relative speeds, for instance, 150 ft per minute would be slow for a 50 ton crane, medium for a 100 ton crane, and fast for a 150 ton crane. For further information see, Tips for Avoiding Crane Runway Problems, AISC Engineering Journal, Vol. 19, No. 4 (1982), pages 181-205.
David T. Ricker, P.E.
Payson, AZ

When welding to AWS D1.1 requirements what is a "seal" weld and what are the applicable inspection criteria for the same?

Steel Interchange

A seal weld is designed primarily to provide a specific degree of tightness against leakage, or melt-through, until the final weld meets the requirements for throat size in assembly gaps of 5/16-in. or less. The seal weld is not included in the integrity inspection of the weld itself.
David L Simpson
U.S. Army Corps of Engineers
Muscat, Oman

AWS A3.0 defines a seal weld as "any weld designed primarily to provide a specific degree of tightness against leakage." This does not define the type of weld but rather its intended function.

AWS D1.1 does not address the specifics of a seal weld, however for a weld to conform to the requirements of AWS D1.1 it must meet the prequalified criteria of Section 2 (unless qualified through testing). Consequently any type of prequalified weld which is applicable to the given joint may be used as a seal weld. The inspection criteria for the weld would be the same as if it were used in a structural application.

This may seem excessive to some readers but consider one recent example:

A construction elevator derailed because a seal welded cap plate on the tube support breaks away allowing the support to fill with water. The weep holes were clogged and during the winter months the water froze deforming the support.
Neal White, P.E., CWI
Special Testing Laboratories
Hartford, CT

New Questions

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

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

Questions and responses will be printed in future editions of Steel Interchange.

In designing composite steel girders in accordance with the LRFD Method, it has been well established that significant reductions in beam sizes can be achieved. However, in my experience, I have found, in some cases, the more economical girder sizes derived may be unsafe during unshored wet concrete construction. This can occur when the metal deck runs parallel to the girder and, in my judgement, does not afford significant lateral restraint to the top flange of the girder in compression. For this condition, the unbraced length is the spacing between the beams supported by the girder. Significant reduction in the non-composite moment capacity can occur due to lateral torsional bucking which may not be adequate for the unshored wet concrete construction.

No criteria for this serviceability problem or guidance appears to be given in the LRFD specification. I would like to know whether there has been any testing or research to demonstrate that metal deck, parallel to the girder does indeed provide adequate restraint or that checking the beam size for the temporary construction condition, should be carried out as outlined above.
Peter J. Maranian, S.E.
Brandow & Johnson Associates
Los Angeles, CA

When designing using the ASD Manual, what is the allowable weak axis bending stress on channel? The manual does not seem to specify this.
Adam Samuel
Riley Stoker Corp.
Worcester, MA

When welding a steel that has dual certification (A36 and A572 Gr50) is there a low hydrogen electrode requirement?

A36 is a group I base metal and ASTM A572 is a group II base metal. Is Aws D1.1 Table 4.1, note 1 applicable to this condition?
Neal White, P.E., CWI
Special Testing Laboratories
Hartford CT

A box at the top of each Steel Interchange contains a sentence that has caused a lengthy discussion in the office. The sentence is as follows: "It is recognized that the design of structures is within the scope and expertise of a competent licensed engineer, architect or other licensed professional for the application of principals to a particular structure."

Fellow office members did not understand the assertation made by the sentence. We think it means architects and other design professionals can practice the art of sizing structural members. If we have the correct meaning, then this presents an open conflict between two design professions. We could only guess at the meaning of "other licensed professional." Please tell me we do not understand the sentence. We fear that the meaning is an example of further eroding of the position professional engineers active in the design of structures enjoy. Persons not trained nor licensed as professional engineers competent in structures, use competitive methods which the average buyer may not understand. Perhaps a better comment would be explaining the many benefits of hiring the trained professional with the legal authority to assume responsibility.
Robert E. Ferguson, P.E.
Engineering Enterprises, Inc.
Bloomington, MN

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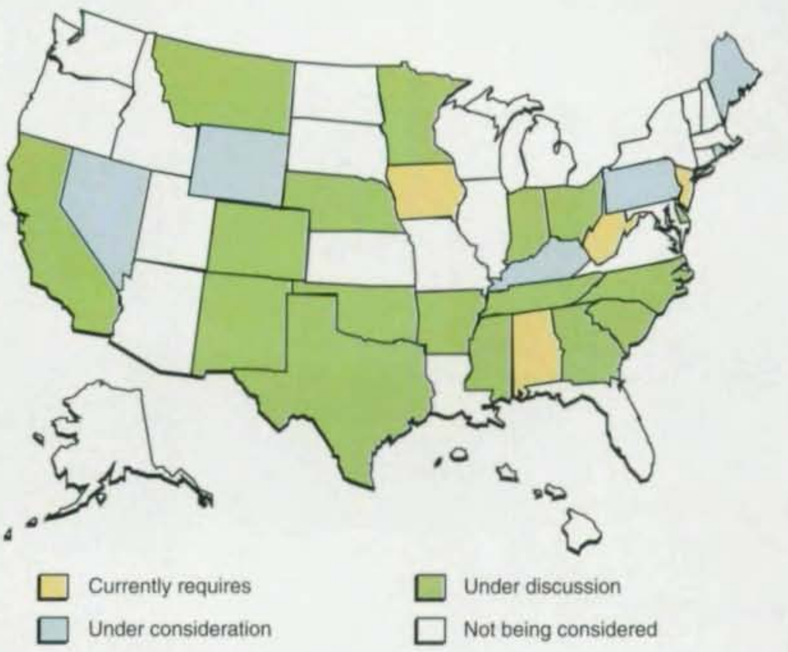
Continuing Education Requirements For Professional Engineers



Graphic by Darren Szrom

Nearly half the state registration boards are now considering requiring engineers to participate in continuing education as a requirement for license renewal. The move began in Iowa in 1978 and it was more than a decade before a second state, Alabama in 1991, followed suit. Since then, two more states—West Virginia (effective in 1994) and New Jersey (effective in 1996)—have added requirements. And six more states are expected to soon join them.

Most of the proposals are quite similar and resemble those that have already been enacted. Alabama is typical and requires 15 Professional Development Hours (PDH) per year, with a maximum of 15 additional hours allowed to be carried over to the following year. In other words, an engineer must complete 30 PDH during every two year period. According to Alabama's rules, a wide range of activities qualify for PDH credit, including college courses, seminars, tutorials, short courses, correspondence courses, videotaped courses, in-house programs, and attending programs at technical or professional meetings. In addition, teaching and writing articles or papers qualifies for PDH credit. Generally, one hour of professional development education equals 1 PDH and one Continuing Education Unit (CEU) equals 10 PDH.

"As professional engineers working for a professional company, we've taken the requirement very seriously," states Alan Speegle, P.E., Quality Assurance Coordinator for the Structural Department at Rust Engineering Co., a 40+ engineer company in Birmingham. "Continuing education is very important to the profession. Engineering is changing—not in the end results, but in how we arrive at those results," Speegle explains. Rust has formalized the professional development process and has a central person responsible for keeping track of all employees continuing education activities. They also hold a number of in-house seminars each year, as well as encouraging staff engineers to attend outside programs. "There are enough programs offered that it is not burdensome to us, though I can see how it might be at a smaller company."

But Bud Romei, P.E., president of the four-engineer firm of Structural Technics in Irondale, AL, says he hasn't found the requirement to be a problem. "I think its a very important requirement. It enhances the entire profession. Continuing education is important to being what we claim to be—a professional engineer. It requires you to keep up with the latest developments. And 15 PDH is such a small number of hours that getting it is almost automatic. However, it does require people who are not really active to rethink their need for a license."

However Romei, who is registered in 13 states, did express one concern. "It would make it much easier if all states had a common form," he says. Fortunately, the states now considering adding continuing education requirements are cognizant of that desire. For example, Russell F. Geisser, P.E., a member of the registration board in Rhode Island, says that when his state adopts a continuing education requirement, it will probably accept other state's requirements. "If an engineer met Alabama's requirement, that would be fine. We're not trying to be nitpicking or onerous." Likewise, Alabama's rules state: "Continuing educational requirements may be met without completing the entire renewal form if a registrant resides in another state which is listed by the Alabama Board as having continuing educational requirements acceptable to the Alabama Board and the registrant certifies in the appropriate section that all continuing educational and registration requirements for that state have been met." Geisser expects his board's recommendations to be firmed up shortly and public hearings on the subject to be held later this year.

Another objection that seems to pop-up in discussions about continuing education concerns the need for older or very experienced engineers to meet the requirement. While Alabama and West Virginia only address this issue as it applies to retired engineers, Geisser says he recommends that an exemption be given based on a combination of age and experience.

While most engineers favor taking continuing education courses, many oppose mandatory requirements, according to Ben Nelson, P.E., a senior project engineer with Martin/Martin, Inc., Wheat Ridge, CO, and committee chairman for continuing education with the Structural Engineers Association of Colorado (SEAC). In a survey of 90 structural engineers from 75 firms, 60% favored voluntary standards while only 40% were in favor of state requirements. "A good number of engineers [75% according to the survey] already meet a requirement for 15 PDH, but a lot of engineers have a problem with establishing another layer of bureaucracy," Nelson says. Another question, he says, revolves around who decides what qualifies. However, many state boards are handling this question by recommending that the level of requirement be suggested by the engineering associations, rather than the state boards.

The survey also revealed disagreement among engineers over reporting provisions. Seven out of 10 engineers favored leaving reporting up to the individual engineer while the remainder favored more stringent reporting procedures. Geisser also reported that some engineers in his state had problems with requiring engineers to keep their own records. But Iowa, the only state with substantial experience in the matter, reports that this is not a problem. "We have voluntary reporting with spot audits," says Patricia Peters, executive secretary to the Iowa Engineers and Land Surveyors Examining Board. Approximately 5% of the renewing engineers are audited each year, and of those, more than 90% have satisfactory documentation. Iowa has about 5,400 registered engineers and renews half each year.

SEAC is expected to issue a position statement early this year. Previously, the Colorado board recommended to the legislature that a continuing education requirement be established for land surveyors, but the legislature denied that due to a lack of administrative funds. A requirement for engineers is still under discussion in that state, however.

"I think that continuing education to maintain your license is necessary," says Richard Weingardt, P.E., president of *Richard Weingardt Consultants*, Inc., a large Denver-based consulting engineering firm and president-elect of ACEC. "Many of us complain that engineers get no respect, but if we want the same respect as other professions we should be willing to meet the same requirements. According to Nelson, teachers in Colorado are required to take 30 PDH over a two-year period, CPAs have a 15 PDH requirement per year, and lawyers and nurses have a PDH requirement of approximately 20 hours per year, depending on their specialty.

The major professional engineering societies have, to date, resisted any proposals for mandatory continuing education requirements. For example, the National Society of Professional Engineers states that they can find "no compelling justification for any jurisdiction to adopt a continuing professional competency requirement...." However, they do recommend that any state that does adopt a continuing education requirement utilize the model requirement developed by the National Council of Examiners for Engineering and Surveying, which is quite similar to Alabama's. NSPE's major objection, according to their position statement, is that approximately 70% of the practicing engineers in the U.S. are not registered and requiring continuing education for the 30% who are would be an unfair burden. However, as others point out, with the privileges of registration come responsibilities.

While ASCE has not addressed the topic of state requirements for continuing education, a committee last April recommended against a continuing education requirement for ASCE membership. ASCE membership is clearly not unanimous on that point though, as is illustrated by a December 1993 Forum article by James W. Poirot, P.E., ASCE president and former chairman of CH2M Hill *in Denver. "I continue to believe* there would be great value to individuals, ASCE and the civil engineering profession in establishing a phased continuing-education program as a condition of membership renewal," he wrote. Poirot points out that the situation today is analogous to the situation at the turn of the century when states began licensing professional engineers. At that time, ASCE opposed licensing, a position they held until 1930. "Let's hope ASCE doesn't take 30 years to recognize that the public, project owners and our non-engineering team members expect engineers to provide evidence that we are continuously studying the state of engineering practice and are current and competent in our field of practice," Poirot writes. "The image of civil engineering is enhanced by requiring CPD because 'respect' comes from taking responsibility. On the other hand, our respect will dwindle if we do not follow the other professions, such as the American Institute of Architects, as they commit to CPD as a condition of membership."

Steel Calendar

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

The new 1993 LRFD Specification is the first major revision to the original 1986 LRFD Specification. The lecture will include a discussion and explanation of the major changes, including such items as the stability of unbraced frames, web crippling equations, slip-critical joints at factored loads, alternative fillet weld design strength and Chapter K clarifications.

The session on Software for Structural Steel will demonstrate methods for using the latest steel design software to create the more efficient designs. Also, a practical transition to LRFD will be explored. Integration in designing various elements and connections in steel also will be featured.

National building codes have undergone a major overhaul on their rules for seismic design of buildings as recommended by the Building Seismic Safety Council and federal agencies. This lecture will cover the "why" and the "how to" of these changes, and their impact on steel design.

And finally, the lecture on semi-rigid composite connections will explain the use of this very economical system.

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

1994 Seminars

WEST	
San Diego	3/17
Irvine	4/21
Sacramento	6/15
San Francisco	6/16
Los Angeles	6/23
Seattle	9/27
Salt Lake City	9/29
Phoenix	10/20
Portland, OR	11/15
Las Vegas	11/17
MIDWEST	
Chicago	3/8
St. Louis	3/24
Des Moines	5/5
Milwaukee	5/31
Minneapolis	6/2
Detroit	10/11
Indianapolis	10/13
NORTHEAST	
Meriden	2/1
Boston	2/2
Portland, ME	2/3
New York	4/14
Albany	9/13
Rochester	9/14
SOUTHWEST	
New Orleans	2/8
Alburquerque	2/10
Denver	2/17
Kansas City	3/10
San Antonio	3/15
Dallas	4/19
Houston	6/21
Oklahoma City	9/8
SOUTH	
Greenville	3/1
Charlotte	3/2
Raleigh	3/3
Birmingham	5/3
Atlanta	9/20
Richmond	9/22
Memphis	10/18
Miami	11/1
Orlando	11/3
MID-ATLANTIC	
Baltimore	4/5
Washington, DC	4/7
Pittsburgh	4/12
Edison, NJ	10/4
Philadelphia	10/6
Cleveland	10/25
Columbus	10/26
Cincinnati	10/27

Steel Calendar

Long-span roof design, innovations in low-rise construction, and the effective use of high-strength steel in building design are just some of the seminar topics expected to attract attention at this year's National Steel Construction Conference in Pittsburgh on May 18-20.

A total of 16 technical sessions will be offered, including a live version of the popular *Steel Interchange* section of this magazine. The session will be limited to questions about connections.

Continuing Education Units (CEUs) will be offered for all sessions.

For more information about the conference, which will run concurrently with an Exposition featuring more than 100 exhibitors, contact: AISC at (312) 670-5421 or fax a request to AISC at (312) 670-5403.

Roberto Leon, the 1993 T.R. Higgins Award winner, will be giving a lecture on Semi-Rigid Composite Construction on February 18 from 7:45 a.m. to 9:30 a.m. at the Midland Hotel in Chicago. Cost for the breakfast seminar is $5. For more information, contact: Bill Liddy, AISC Marketing at (708) 527-0770.

The International Conference on Fatigue will be held May 9-10 in Toronto. The sessions will include information on the fatigue design of weldments, the use of bolted connections, and fatigue design for hollow structural sections. For more information, contact: American Welding Society, 550 N.W. LeJeune Road, P.O. Box 351040, Miami, FL 33135 (800) 443-9353; fax (305) 443-7559.

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Erection And Constructability Issues For Long-Span Roofs

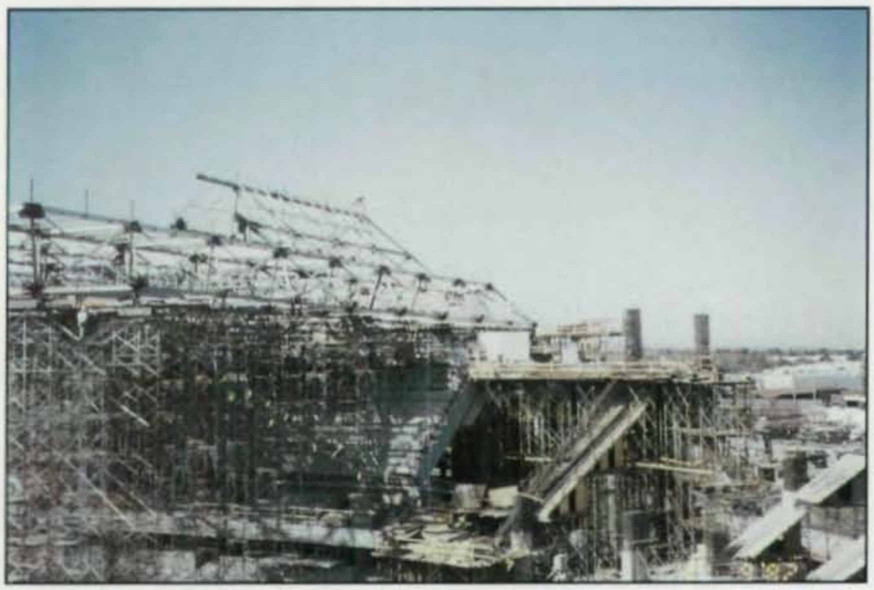


Figure 1 (top): The Fargodome. The 200-ft. x 320-ft. section in the center was lifted from the ground to its final position in one day.

Figure 2 (above): San Jose Arena. During the erection of the roof space frame structural steel was shored to the arena floor at every third panel point.

A look at lessons learned on a series of long-span construction projects

By Jack Petersen, P.E.

The complexities involved in designing and constructing long-span roof structures create unique problems and challenges. As a result, different approaches to these projects are needed. Specifically, engineers need to anticipate and accommodate erection problems.

Erection Procedure

The erection scheme used to construct long-span roofs on buildings will often determine if a project will be successful. The demands of roof construction may dictate access to the site and influence the project schedule. It may be impossible for other trades to continue working below while erection is in progress. A creative scheme, which addresses the special problems of a long-span structure, may result in a competitive edge to the bidding contractor.

When the contractor for a long-span roof is selected via the bid process, the EOR must define a reasonable, buildable erection scheme on the drawings as a common basis for bidding. In this case, the EOR must make assumptions regarding the erection procedure with limited knowledge of construction issues. These issues may include construction sequence, site constraints, coordination of temporary supports, and site access. When the Design-Build method is used to retain the project team, the EOR may have considerable input from the contractor during design on these issues.

The erection scheme shown on the construction documents must clearly communicate the basis for the EOR's design while maintaining flexibility for the contractor to change or improve it. It must define what factors the EOR has considered in his design. In addition to design information required on typical steel structures, the contract documents must define locations and extent of assumed temporary supports, lateral bracing at intermediate stages, splices and cambers. For two-way systems, the distribution of dead load in the structure results from the erection sequence. For such structures, final member sizes are dependant on the sequence of erection, therefore, the sequence must be defined to allow the contractor to evaluate steel quantities.

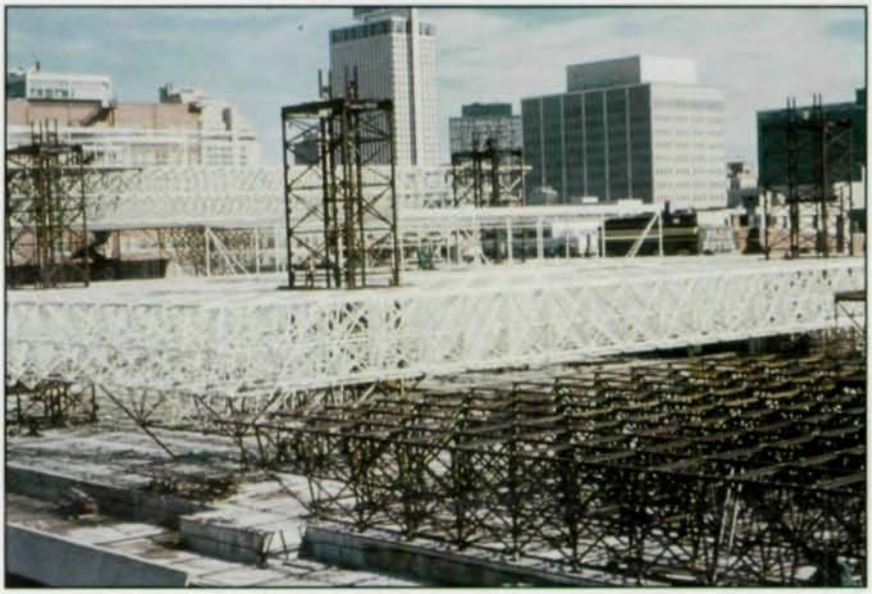


Figure 3 (top): AlamoDome. The primary trusses were designed to be constructed on falsework and lifted by a single crane at each end.

Figure 4 (above): Currigan Hall. The entire roof space frame was constructed on the convention floor and jacked into place in sections.

Assigning Responsibility

The EOR must also clearly define what he has and has not designed and assign appropriate responsibility to the Contractors' Engineer. In typical long-span roofs, these topics might include:

• Design of shoring towers, work platforms, and their foun-

Figure 5: McNichols Sports Arena. Originally designed as a two-way system with cable bottom chords, the system was revised to accommodate redistribution of dead loads from the one-way erection scheme shown above. Cables were added in the opposite direction after erection of all trusses.

dations.

•Design of bracing required at stages prior to completing the system.

•Design of system to resist wind and seismic forces in its incomplete form.

•Position adjustments for temperature and length adjustments due to shrinkage and tolerances.

•Procedures for and times of surveys within the erection sequence to verify erection tolerances and structure performance.

To achieve the most economical solution, the EOR should permit alternate erection schemes, prepared by the contractor's engineer. By allowing other schemes, the EOR encourages economical solutions developed by the persons most knowledgeable about fabrication and erection. The alternate schemes must meet all of the structural design criteria of the EOR's scheme. Also, deducts or adds for erection scheme alternates must be clearly defined in the bidding documents. If alternate schemes are permitted, the specifications must define the responsibilities of the contractor's engineer in preparing the scheme as well as the role of the EOR in review. Alternate schemes may not be acceptable if the configuration does not meet architectural criteria or delays the construction schedule. Some topics which may be addressed in the erection submittal include:

•Stress reversals in members due to alternate position or different number of temporary supports.

•Alternate member size, configuration or splice points.

•Alternate connections to accommodate hoisting, jacking, or erection clearances.

•Changes in member sizes and connections due to alternates in two-way systems.

•Predicted cambers and bearing positions based on alternate schemes.

In some cases, the EOR may not be permitted to participate as the contractor's engineer due to the owner's liability. In most cases, re-engineering for the contractor's scheme is not within the scope of the EOR's service.

Requiring an erection submittal in the specifications is an excellent method of communicating the assumptions of design between the EOR and the Contractor's Engineer. The responsibilities of the contractor's engineer can be clearly defined, as well as the extent of review by the EOR.

Stable Erection

An excellent example of the procedure described above with an alternate erection scheme is the FargoDome, a covered stadium and multi-purpose arena in Fargo, North Dakota. On this project, the primary long span trusses were designed to be stable under selfweight for erection. The erection scheme shown on the drawings indicated these trusses could be erected first and then joists added.

The Contractor was awarded the work on the basis of the bid drawings and then submitted an erection scheme in which the entire center section of the roof was jacked off the perimeter roof. MARTIN/MARTIN reviewed the erection submittal for its effect on final design and provided loads for the design of alternate connections.

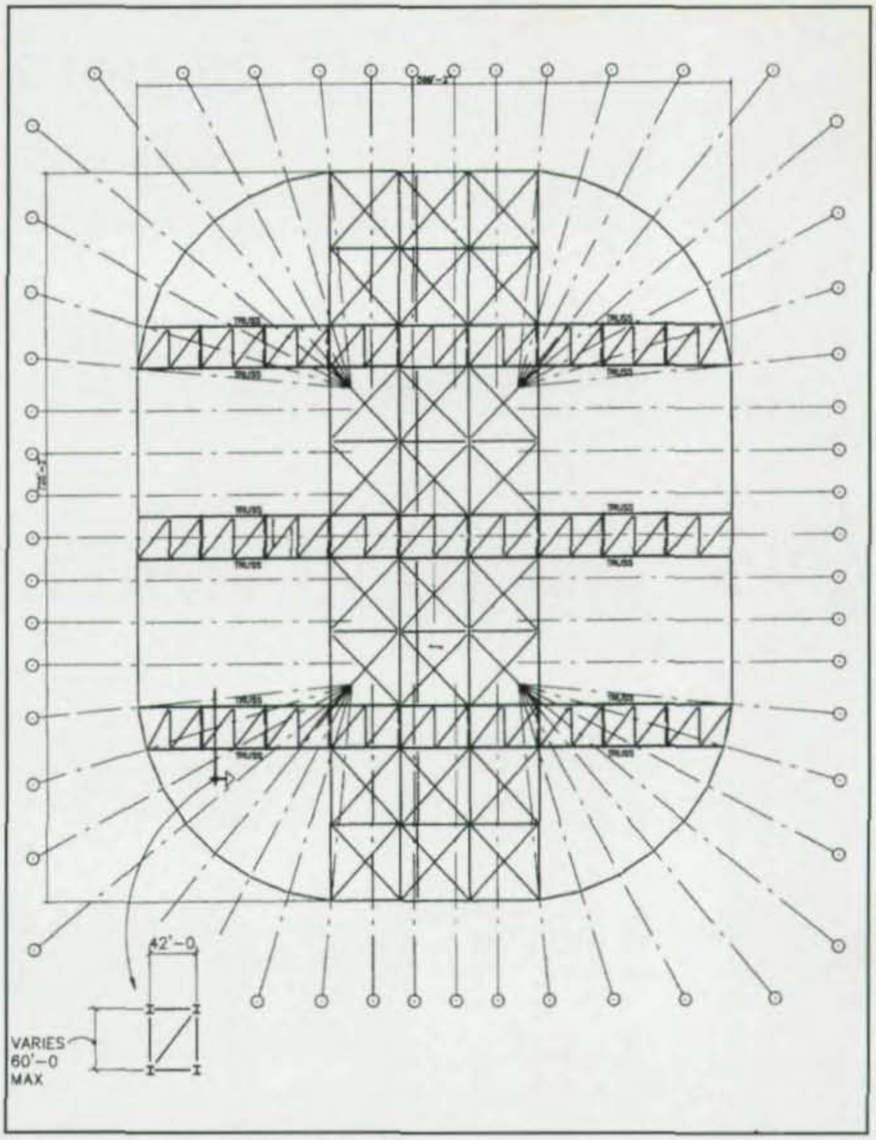
By building the center section on the ground and jacking it up in one piece, most work was completed near ground level without disrupting adjacent construction activities. The alternate scheme required only slight modifications to connection details at bearings. A photo of the roof during erection is shown in figure 1.

At the other end of the spectrum, the Contractor may elect to follow the erection scheme shown on the drawings. The recently completed San Jose Arena, a large (425'-0 x 425'-0) space frame (see figure 2) is such an example. Due to an almost infinite number of possible erection schemes, MARTIN/MARTIN presented the roof structure as completely shored on the contract documents. During construction, the frame was supported at every third panel and jacked down as a complete structure.

Structural System Design

Long-span structures have a number of unique problems related to erection which influence the selection and economics of the structural system. For these buildings, system selection based on weight may not yield the most economical or constructable design. Architectural constraints may eliminate some systems from consideration. If the EOR is defining an erection procedure on the bid documents, special considerations of constructability must be addressed.

The erection of spans in excess of 300-ft. typically require some in-place assembly of structural steel. Often times, this work is performed on platforms of limited size, far above the ground surface. Lifting and positioning of each piece of steel requires a large crane due to reach and accessibility considerations. For safety reasons, construction by other trades below is often interrupted while erection proceeds. Delivery of material and/or access for cranes may dictate the sequence of substructure construction. These factors significantly affect project schedule and may influence overall project economy.



To alleviate the problems due to in-place assembly, the structure may be designed to maximize shop fabrication and pre-assembly. The details of the structure should specify shop welds or bolts for the largest sections which may be shipped or lifted on the job. An example of designing to lift capacity is the AlamoDome in San Antonio, Texas (figure 3). In this project the primary trusses, spanning 400 ft., were designed based on the hoisting capacity of one crane lifting at each end. If a contractor is on board during the design phase, the engineer should utilize his knowledge of site constraints, lifting capacity, and construction sequence in developing details. Connections which can be constructed in the shop may offer better thermal control, improved welding position, and improved access for welding inspection by the testing agency over those done in the field. Any problems arising from distortion and weld shrinkage can be identified and corrected prior to erection.

Pre-assembly of large sections on the ground or on low level falsework can greatly improve system economy. Individual pieces on such sections may be lifted by small cranes or lifts reducing demand on large equipment. Construction of access platforms may be greatly simplified near ground level and exposure of workers reduced. Ideally, pre-assembled sections should be designed to rest on bearing seats at erection. Design should allow them to be tied back and stabilized with adjustable connections using erection bolts. Final connections, after alignment and positioning, may be welded or bolted consistent with detailing on the project.

Pre-Assembling An Entire Roof

Extending the pre-assembly concept to incorporate the entire roof is possible. An example of such a structure is Currigan Hall in Denver, Colorado (figure 4). On this project, the roof space frame was constructed on the convention hall floor with prefabricated elements and "uni-

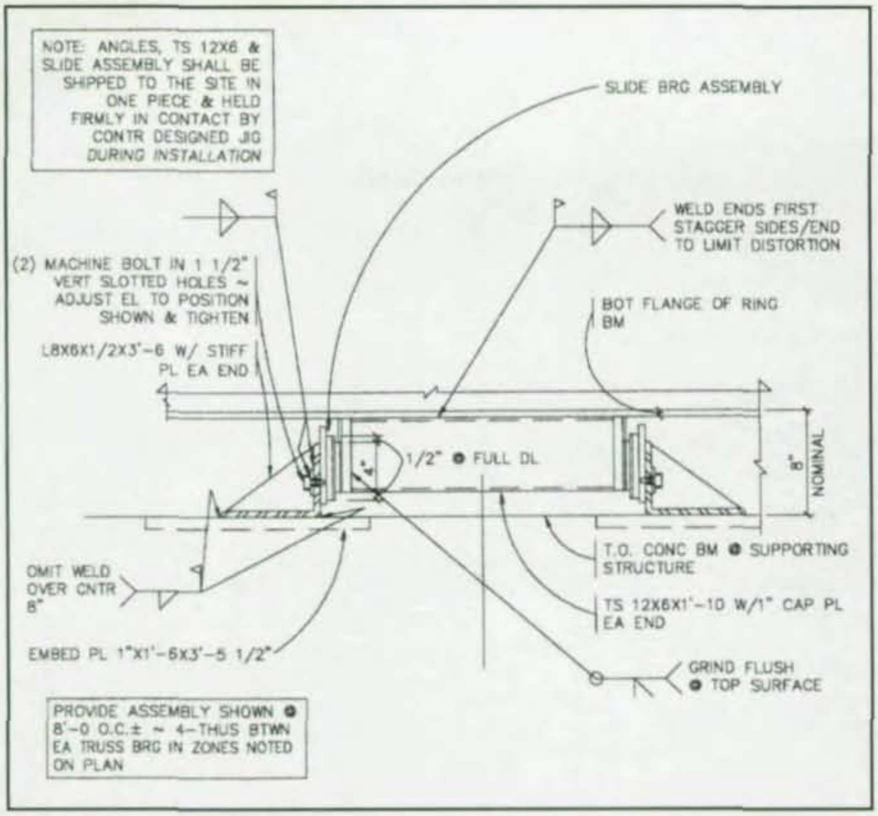


Figure 6 (left): Two-way scheme for St. Louis Stadium. The east-west box trusses were designed to span 600 ft. and remain stable during erection. The north-south trusses would be erected only after the completion of the rest of the structure and only carry live load and provide redundancy.

Figure 7 (above): Slide bearing for transfer of lateral forces from roof perimeter to substructure of St. Louis Stadium. Design permits pieces to be field adjusted in each direction and provides adequate access for welding.

versal" joints common for all connections. The roof was then jacked into place in sections on the building columns.

Another condition unique to long-span design is occupancy. Many long-span structure's (arenas, stadiums, concert halls, etc.) primary use is for functions with large crowds. A redundant structural system is desirable for such facilities to localize failure in the event of an overload. Although more difficult to erect, two-way systems are efficient and redundant. Redundant systems can sometimes be designed to be installed after the primary system is complete. Deferring installation of some components of these systems may simplify construction until the end of erection. An example of this idea is McNichols Sports Arena in Denver, Colorado (figure 5). The structure was designed as a two-way truss and was shown on the construction documents on six temporary shoring towers. After bidding, the EOR was retained to re-design the structure as a series of one way span trusses which were built on the ground and then lifted into place. After erecting all of the one way trusses, cables were added in the opposite direction to share live-load forces and provide redundancy.

When developing splice locations, connection details, cambers and other information, the design and fabrication must be coordinated with the assumed erection procedure. Consideration should be given to each piece which will be self-supporting during the erection sequence to determine if it can be easily stabilized. Due to the height and size of these structures, tieing off or cabling to an adjacent structure may be difficult if not impossible. Reducing the amount of temporary bracing/strong-backing that is required will lower costs and speed erection.

Systems designed to achieve stability in sections prior to completion of the structure are very desirable. They are typically redundant in nature and may reduce interruption of work by other trades. Despite being slightly heavier in unit weight of steel, well conceived systems such as these may permit reuse of shoring towers and rigging equipment, thus reducing erection costs. An example of such a system shown in figure 6, was one system considered for the new St. Louis Stadium.

Design of Structural Details for Constructability

Despite the wide variety of structural systems and configurations used in long-span roof structures, there typically are a number of common considerations in their details. These details must be addressed early in design to create economical, buildable solutions.

Properly detailed and constructed bearings are essential in assuring that the actual structure behaves as assumed in design. For long-span roofs, they tend to be quite large based on the scale of framing elements, and the loads. Bearings are typically installed at the interface of the roof structure and the substructure which frequently leads to construction difficulties. Often times, this point represents the accumulation of fabrication and erection tolerance for both the roof and substructure. Space for bearings may be limited by architectural constraints. Finally, bearings require careful

Figure 8: MGM Special Events Center roof structure. WT Shapes notched at each joist were used to brace trusses and carry lateral forces. In this manner, bracing is installed over two or more spans with a single lift of the crane.

handling and protection during installation due to use of elastomers and other materials.

Design of bearings to prevent construction difficulties requires the Engineer to visualize how the bearing will be erected, positioned, and anchored. Of utmost importance is providing enough room for the bearing and for installation. Accumulating tolerances should be considered with some allowance for an out of tolerance condition. Expansion bearings which are adjustable will permit the erector to align elements to the correct location after erection. Jigs or temporary connections between the elements of expansion bearings will reduce the possibility of damage to sliding surfaces. Anchorage of the bearing to the substructure must have some adjustment. "Fix" details or repairs for bearings may be difficult since they are easily damaged by spatter due to flame cutting or overheating during welding. An example of an adjustable bearing design with bolted connections is shown in figure 7. This bearing was designed to transfer wind and seismic forces from the roof structure to the substructure on the new St. Louis Stadium. Proper positioning of bearings to compensate for camber and thermal conditions of erection as well as temporary lock-ins may require good communication between the EOR and the Contractor.

Bracing Considerations

Another common detail to long-span structures constructed of structural steel (or built-up) shapes is the need for bracing. In typical structures metal deck is often relied on to brace roof framing. In long-span structures, however, deck may not have adequate capacity to brace the very large chord sections, hence structural steel bracing is required. Bracing presents a problem because it is ideally placed near the elevation of the metal deck, hence it is interrupted every five to ten feet by purlins which support the deck. Bracing is usually an element which is erected in place, so individual pieces installed in between purlins are time consuming during erection. Multiple span pieces or tension only (i.e., cable) systems are often good solutions to this problem. An example of a multiple span bracing system utilizing WT shapes notched at each purlin is shown in figure 8. In this roof, the MGM Special Events Center in Las Vegas, the flange of the WT has adequate area for strength as it passes over the joist, and the stem between joists makes the shape stable in compression about a horizonal axis.

In areas subject to snow and ice build-up, some type of retainage system is commonly required at the perimeter of long-span roofs. This "gutter" or "fence" closes the edge of the roof structure and prevents discharge of ice which might injure pedestrians. The perimeter details are an excellent example of where the design for construction convenience can save erection time and costs. Figure 4 shows perimeter details on two stadium roofs. On a similar project, the St. Louis Stadium, the design drawings specify a perimeter truss erected in one piece and tied off, thus greatly reducing the number of pieces handled and connected.

The above are just a few examples of how the design and drawings can improve the constructability of long-span roofs. Typically such details require fewer repairs and fixes resulting in less time spent by both the EOR and the contractor. A well designed long-span structure will consider the unique construction problems faced by the contractor. Attention to stability, tolerances, and constructable details can reduce costs and cut the erection schedule. Design-Build projects offer unique opportunities in the design of long-span roofs and allow the contractor to set the parameters to which details of the structure will be designed.

Jack Petersen, P.E., is a senior project engineer with Martin/Martin Consulting Engineers in Wheat Ridge, CO.

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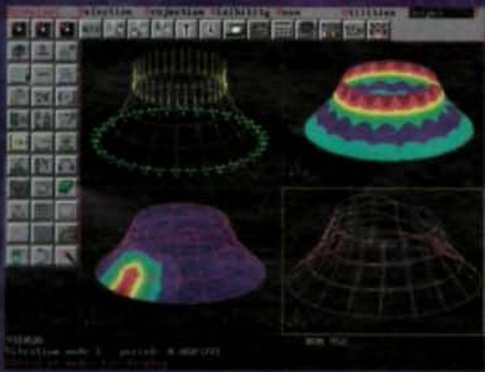

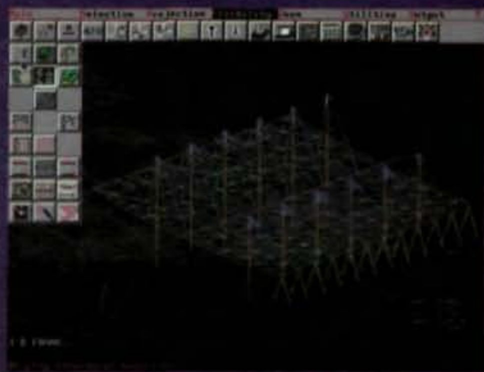

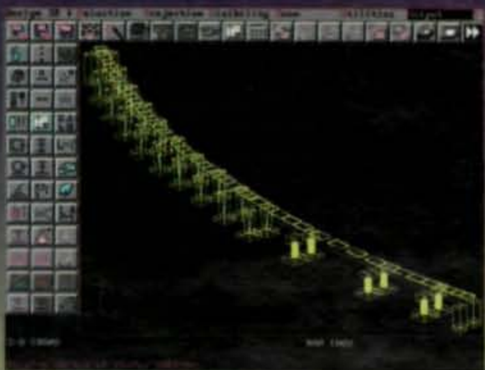

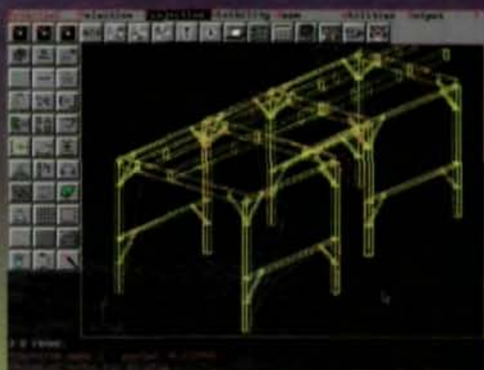

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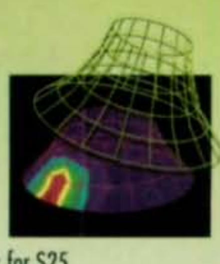

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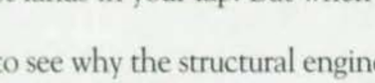

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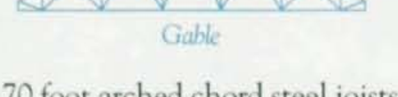

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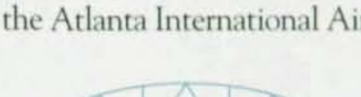

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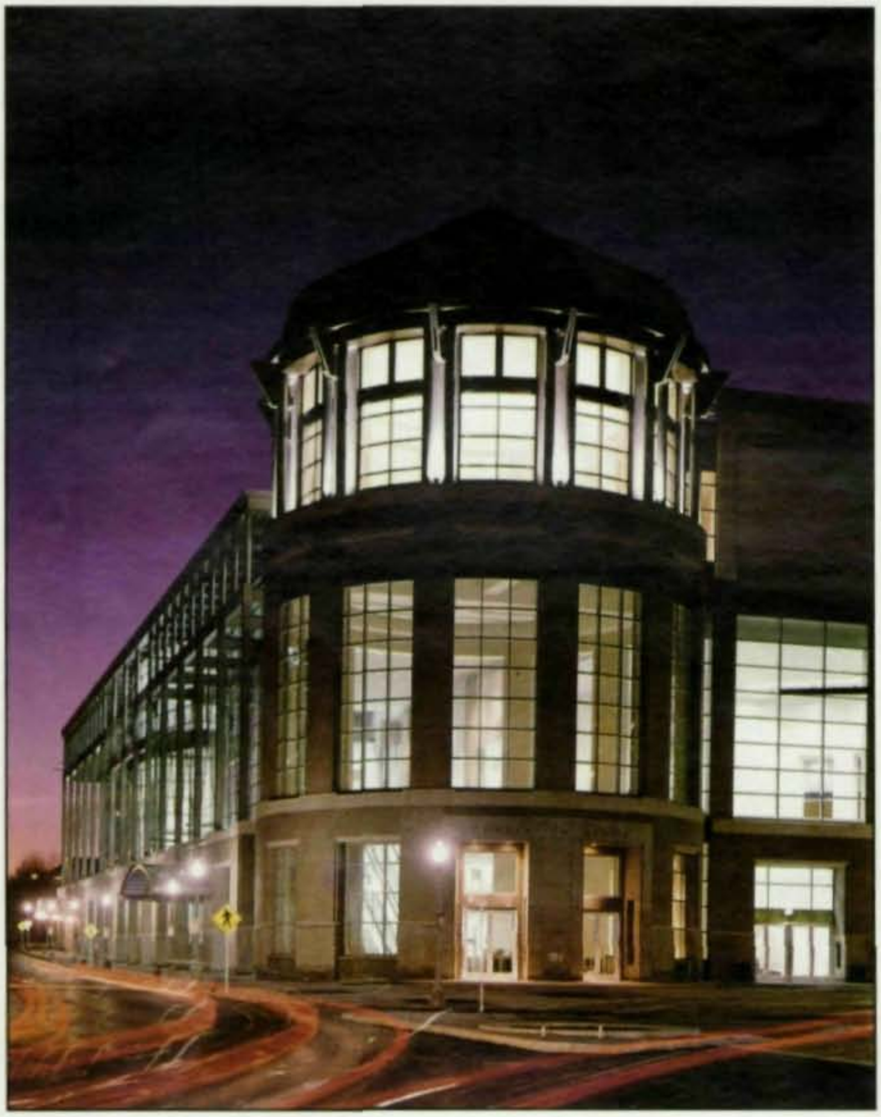
Meeting Vertical And Horizontal Constraints

A new generation of vertically stacked convention centers fit into urban cores without destroying the city grid



Site considerations played a key role in the design of the Providence Convention Center.

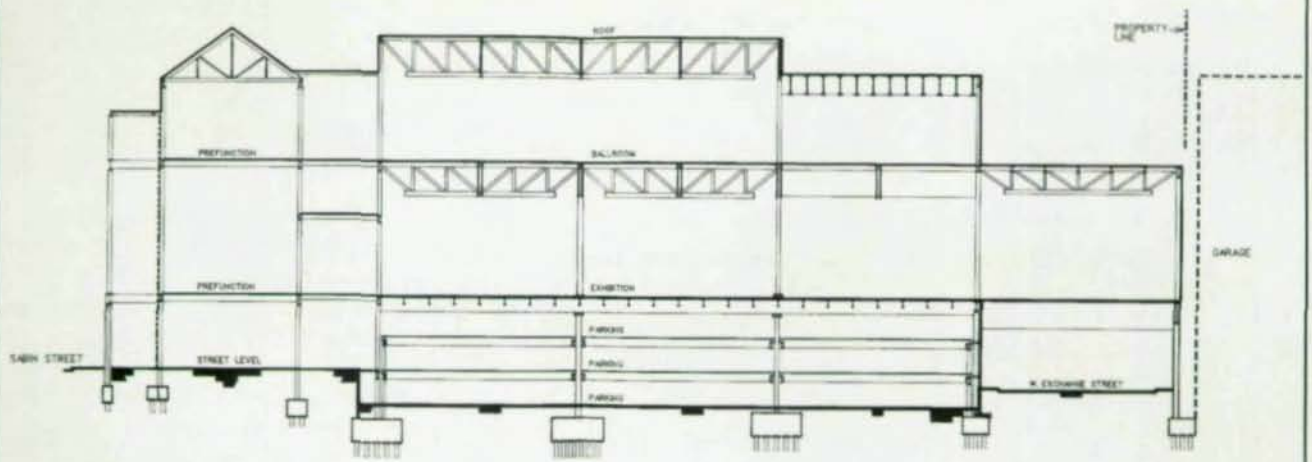
By John C. Alfano, Greg M. Detmer, AIA, Brian T. Eaton, P.E., Donald I. Grinberg, AIA, and John T. Holcomb

Unlike its sprawling predecessors, the Rhode Island Convention Center is a vertically stacked convention center whose design was dictated by a tight site and a restricted urban location.

The five-level structure had to meet depth restrictions dictated by the water table, height restrictions imposed by the City of Providence, and horizontal space restrictions resulting from the tight site in the heart of downtown Providence at the intersection of two heavily travelled streets and immediately adjacent to the existing Civic Center. In addition, the architects had to meet Capital Center District design criteria. And finally, the design needed to take into account functional and cost considerations that made it desirable to minimize floor-to-floor heights without compromising the required vertical clearances in the key leasable spaces.

From bottom to top the 365,000-sq.-ft. structure includes: three levels (one above, one at, and one below grade) of parking for 720 cars; a 100,000-sq.-ft., 30-ft.-high exhibition hall on the next level; and above that a 20,000-sq.-ft., 25-ft.-high ballroom, 18 meeting rooms, and a full-service kitchen, which can produce 5,000 meals per day.

The building was the result of a developer-builder-architect competition won by developer MetroPartners, Providence, architect HNTB, Boston, and



CROSS-SECTION THRU PROVIDENCE CONVENTION CENTER

construction manager Gilbane Building Company, Providence. HNTB's structural consultant on the project was Boston-based Zaldastani Associates.

Vertical Requirements

Because the 100,000-sq.-ft. exhibition hall could not fit on the ground level without impacting city streets and adjacent hotel and parking garage access, the designers placed it 25 ft. above ground and extended it over one of the streets. This vertically stacked design required that the structure support large loads using long spans above grade. To accomodate the design, a construction plan was developed based on erecting the center in five 120-ft. vertical sections from the ground right up to the roof. Working from west to east, the steel erectors finished one section at a time to allow continuous crane access.

Beneath the exhibition hall is a three-level garage. After examaning a variety of options, the design team chose a hybrid form of construction with steel building columns and precast 60-ft. double tee planks. The combination of precast pieces and steel columns made for a very economical design, especially in light of the unusual site and tight construction schedule. Also, the sep-

arate steel perimeter beams and columns for the garage structure, together with a center expansion joint, meant the garage could expand and contract independently under the effects of ambient temperature changes.

Since the center was built in vertical segments, the coordination of fabrication, shipment and erection of the various structural components was critical to the construction schedule. Therefore, to promote efficiency and control, the production and erection of the precast tees was made part of the fabricator's contract. Since each tee had to go in sequentially and since the next level of structural steel for the building segment followed directly behind, one missed tee delivery would have had a serious impact on the erection process. The precast tees are supported by paired W21x83 steel beams. All of the steel for this project was A572 Grade 50.

To minimize the elevation of the exhibition hall above, the garage was designed with a tight floor-to-floor height. In addition, there was a restriction to the thickness of the floor sandwich of the exhibition hall immediately above the garage. The structural engineer devised a system of plate girders on each end of the grid lines, limited in depth to three ft., which were bolted and welded together to create multiple continuous spans. The girders sit on top of supporting columns, some of which extend up through the building. The columns that terminate at this level have a cap plate designed like a bridge rocker bearing. These elements allow the shallow plate girders to experience stress and deflection under large design loads without imparting bending moments to the columns, particularly those above, which are unbraced for 40 ft.

Contained in the floor sandwich between the upper level of the garage and the exhibit hall are the exibition hall concrete slab, the plate girders that support it, and the utility floor boxes, wires, pipes and ducts that service the exhibition floor, and finally the suspended ceiling of the garage. In addition, the entire plenum space is heated. Very detailed and well-coordinated drawings were required to meet all these requirements in a shallow space with a total plenum depth of 5 ft.

The structural engineer used STAAD-III from Research Engineers and ECOM's design package for design and analysis.

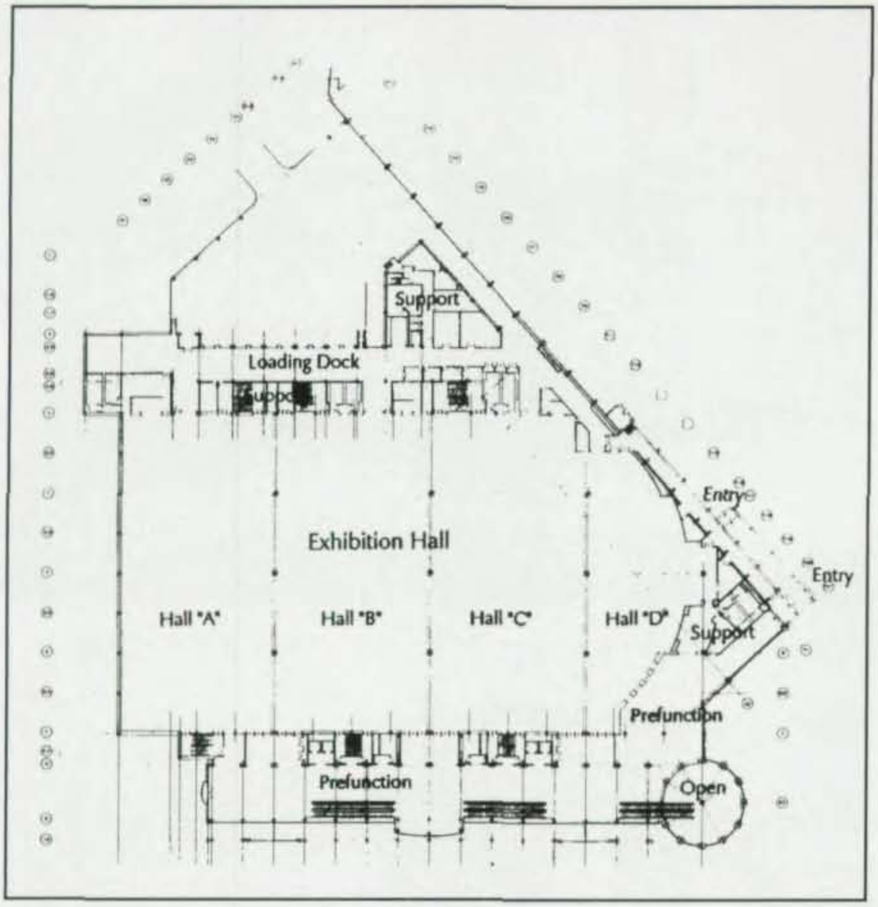
Elevated Exhibition Hall

The exhibition hall floor is designed to carry a live load of 350 psf, permitting it to carry heavy equipment, vehicles and displays—including boats. The plate girders and beams support a 9-in. concrete slab with a compressive strength of 5,000 psi. The slab was poured in 30-ft.-wide strips and was leveled using a vibratory highway screed running on temporary rails supported off the plate girders. Its surface was made more durable by applying a shake-on metallic aggregate hardener. The hardener was applied using a mechanical spreader that spanned across the pour. These techniques easily permitted the contractor to achieve a finished surface with an ACI flatness number of F(F) = 25.

In constructing the exhibition hall, the team used air rights over West Exchange Street, placing columns on the far side of the street on new foundations shared with the adjacent parking garage. Basically, the contractors built a building floor plate elevated 25 ft. in the air with an entry lobby, garage, service road and city street underneath. Building columns are built on a regular grid. However, site conditions require some transfers, which are accomplished with plate girders as deep as nine ft. Some of the building columns extending up to the ballroom and roof are as large as W14 x 730. The hall itself is supported on large 10-ft.-deep trusses with top and bottom chords ranging from W14x109 to W14x283.

As the exhibition hall level is elevated above street level, so is its lifeline, the truck dock. Once again, the team examined a number of construction options before ultimately determining that steel construction was the best choice for the truck dock. However to allay concerns associated with long term performance and durability, the steel beams and columns are coated with zinc rich primer; the lower portion of the columns is encased in concrete, and all the top and

Large members were required to support the large clear spans on the center's upper levels.



bottom concrete reinforcing bars are epoxy coated. Metal deck between the beams is used only as a slab form. This nine-in. concrete slab has a compressive strength of 5,000 psi and contains a microsilica admixture for greater protection against moisture penetration. A first-class traffic-bearing membrane also was applied to the top surface of the slab. All steel beams are fireproofed, and the space below the slab and above the suspended ceiling is heated.

Special Considerations For Ballroom Design

The 20,000-sq.-ft. column-free ballroom, located above the exhibition hall, required special considerations. The floor, which is 70 ft. above street level, is designed to support a static live load of 150 psf, equivalent to the design load for libraries and warehouses. The team used 60-ft. and 120-ft. trusses to support the ballroom and to provide the clear spaces required for the exhibition hall below. Because the exhibition hall height requirement was 30 clear ft. and it was desirable to minimize the vertical elevation of the ballroom level, the depth of the trusses could be only 10 ft.

Stiff 60-ft. span trusses with a natural frequency profile that is out of the range of rhythmic dances were used under the main ballroom, where a variety of functional uses and loading occur, and 120-ft. trusses were used to support the kitchen and meeting rooms that make up the balance of the floor.

The roof structure over the ballroom is a series of trusses, partially exposed to give an architectural treatment. Some have W14x233 top chords and W14x193 bottom chords, while others have 18-in. tees as a top chord with W18 bottom chords.The balance of the convention center roof consists of 120-ft.-long, custom-made steel joists. Since the ballroom itself can be subdivided into five smaller function spaces through the use of moveable partitions, the roof truss system was designed to support a suspended track system for these partitions. Roof and floor deflections under potential live loads were coordinated with track and partition details to ensure proper operation.

Exposed Steel

Architecturally, a primary vocabulary of the building is exposed steel: the trusses in the upper prefunction space; elements of the entry rotunda; and the front facade of the building with 30-in.-deep beams and columns and two-in.-diameter rods as architectural cross bracing.

The architects considered various options for the exterior treatment of the building, including the use of granite and masonry to wrap the steel. However, it was decided to expose the structure of the upper portions of the building, thus achieving a desired layering and transparency between the interior pre-function areas and the surrounding city. The exposed steel contrasts with the more traditional granite and brick used at the arcade's street level.

Exposing the structural steel within the ballroom itself carried the theme and transformed the ballroom into a unique space. A versatile and flexible indirect lighting system softly modulates the ceiling and partially exposed trusses, thereby creating an elegant and dramatic effect. The theme of exposed steel is even carried through the custom fabricated stainless steel railings.

Complex Steel Erection

Steel erection on this tight site was a complex task. Prior to the start of design, the team met with fabricators and erectors to discuss how they would erect this building and how they would deliver the required trusses to the site. It was at this point that the scheme of constructing the building in vertical segments was created. This method of construction had the advantage of providing a fast learning curve for the contractors, as building erection at each level was repeated with each section. However, according to Lee Burneson, Gilbane superintendent/manger: "The most difficult part of the construction occured as the crew met the skewed angle created at the east end of the site.

"The last sections of the building to be erected were bounded by the skewed property line of the site. This line of the structure had complex steel connections for trusses and vertical bracing that had to tie into the mass of the already plumbed and erected structure.

"At the same time, there was less and less site available for laydown. However, we still had the benefit of our experience in erecting so much of the building from bottom to top already."

Because of the heavy trusses 75-ft. to 90-ft. in the air, the boom of the 300 ton crane was restricted in reach. To assist in truss erection, AISC-member Berlin Steel brought in a tower crane to work in tandem with the heavy crane.

Steel assembly was enhanced by the determination that the long-span trusses could be delivered to the site in one piece, greatly increasing quality control and reducing on-site staging requirements. At any one time, the steel contractor controlled 25-30% of the site, including areas for lay down.

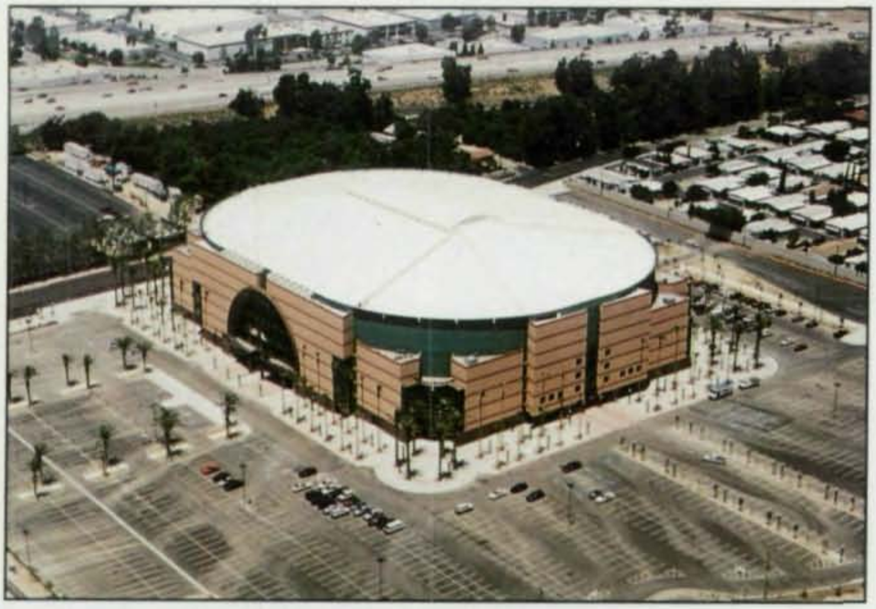
The variety in types, lengths and weights of steel framing elements on the project was carefully planned by the erector, Berlin Steel. Assembly was further challenged by the spherical bearings at the bottom of many columns, which required temporary bracing until the major elements of the ballroom superstructure were in place.

Structural steel erection was the backbone of the construction schedule. Once the steel was set in place, aligned, plumbed, bolted and welded, connections were inspected by a testing agency. Penetration welds were ultrasonically tested, and every plate girder connection, truss connection and bracing connection was tested.

John Alfano is a project executive with Gilbane Building Company, Providence. Donald I. Grinberg and Greg M. Detmer are principal architects with HNTB, Boston. Brian T. Eaton, P.E., is an associate with Zaldastani Associates Inc., Boston. John T. Holcomb is vice president of AISC-member The Berlin Steel Company, Berlin, CT.

Spanning Hockey's Newest Pond

The roof of Anaheim's new arena clear spans 329-ft.-by-444-ft. to provide uninterrupted views



By Thomas Z. Scarangello, P.E.

Nothing's worse then paying top-dollar for a sports ticket and then finding yourself behind a pillar. To avoid any obstructions in the new stadium for the NHL's Mighty Ducks, the roof clear span stretches 329-ft.-by-444-ft. over a 19,200-seat arena known as "The Pond At Anaheim".

Clear-spanning such a large area required a complex structure. After careful consideration, a steel two-way tied arch system was selected. Advantages of the system in this application included:

1. Tied arches minimize outward thrust, and therefore eliminate the need for a perimeter ring to resist tension (as would be needed with untied arches) or compression (as would be required with air-supported and tensile structures). The roughly rectangular footprint, ideal for arena seating layouts, was not suitable for a round or oval tension or compression ring around the roof edge.

2. Two-way spanning action accommodated the architectural intent of the project. To minimize the visual bulk of the project and the enclosed volume, the architect designed the roof with shallow edge depths and corners that were trimmed back to curves and then depressed as distinct "pie slices". Such special corner and edge conditions were easily accommodated by the two-way system selected. In contrast, one-way span systems are best suited for rectangular roof plates of uniform elevation.

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

4. The trussed arch profile selected has low arch profiles and sagged tension ties, minimizing roof height, visual bulk and enclosed volume without sacrificing overall structural depth or obstructing sight lines.

5. Large, efficient structural depths were created by combining shallow, easily shipped upper trusses with simple tension ties and queen posts.

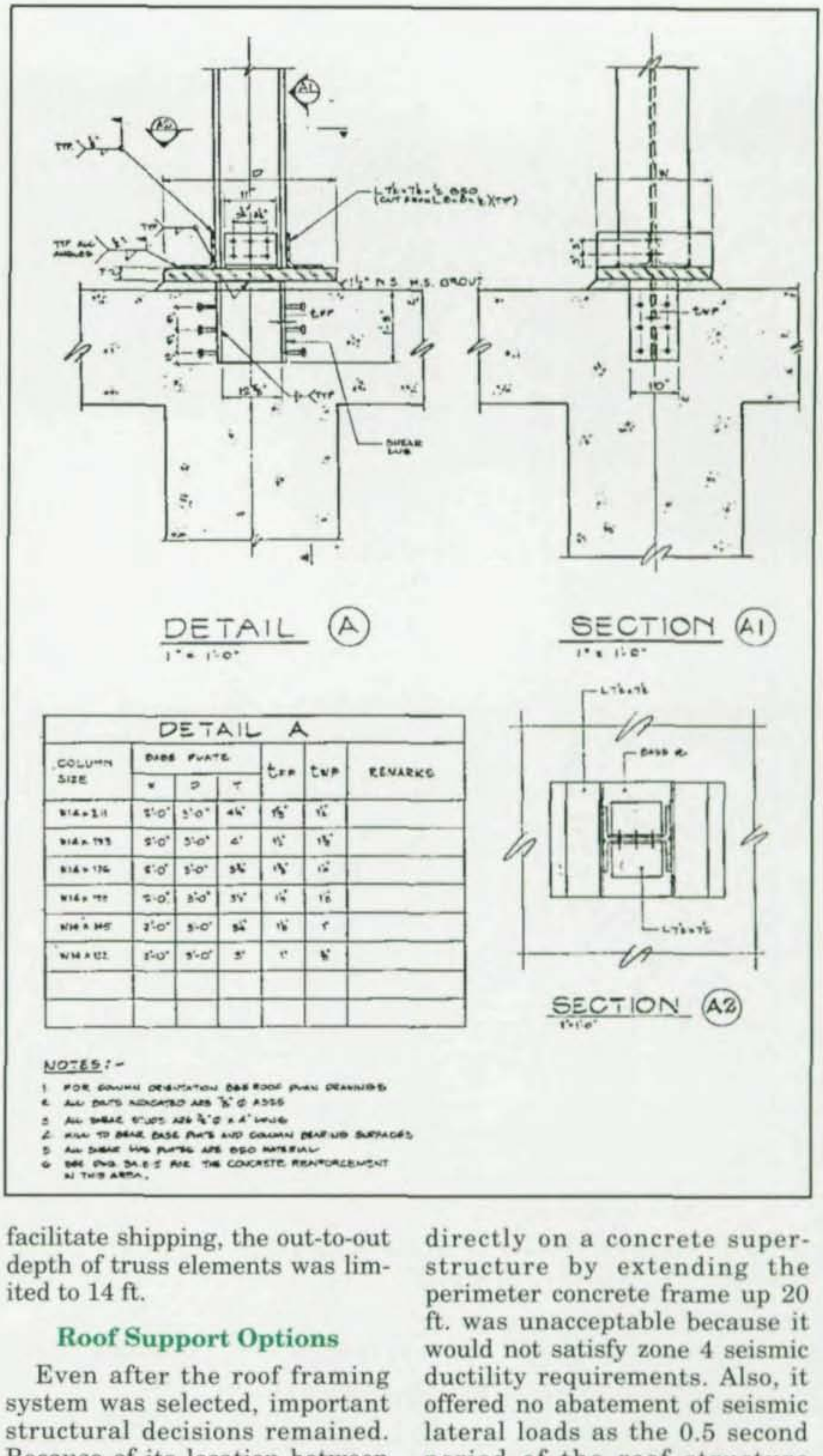
6. Erection was straightforward and efficient by erecting primary trusses first and then dropping in secondary framing.

The roof structure uses six intersecting steel broken-back tied arch trusses to form the main roof framing elements. The trusses are 12.5-ft. deep between chord centerlines and slope downward 28 ft. towards each end. The tension ties connect to truss bottom chord ends, and pitch downward an additional 10 ft. to central posts, creating an effective overall depth of approximately 50 ft. The typical trusses have bottom chords ranging from W14x30 to W14x43 and top chords ranging from W14x43 to W14x68.

Infill trusses, also 12.5-ft. deep, cover the central roof area by spanning between the main tied arch elements and the roof perimeter.

The "pie slice" sections at each corner of the roof are depressed 5 ft., but the interior view is unaffected as the radial trusses here are only 7.5' deep. Wide flange purlins on 15-ft. centers span between trusses and support the roof deck. The deck, supplemented by in-plane bracing elements, forms the roof diaphragm system.

All trusses have wide flange top and bottom chords, ties and support columns. The tension tie members range from W14x61 to W14x176. Truss verticals, horizontal elements and bracing elements are double angles. To



facilitate shipping, the out-to-out depth of truss elements was limited to 14 ft.

Roof Support Options

Even after the roof framing system was selected, important structural decisions remained. Because of its location between the stiff roof plane and the stiff seating structure, the support system for the long-span truss reactions plays an important role in proper seismic performance. Several options were evaluated for the roof support system.

Supporting the roof trusses directly on a concrete superstructure by extending the perimeter concrete frame up 20 ft. was unacceptable because it would not satisfy zone 4 seismic ductility requirements. Also, it offered no abatement of seismic lateral loads as the 0.5 second period of the roof structure essentially matched the concrete superstructure. And finally, it would have been cost-prohibitive: it carried a high cost of a "flying form" construction (about $700 per cu. yd. of concrete) and required rocker and slide bearings with an estimated cost of

$200,000 to accommodate thermal movements between the roof and the superstructure.

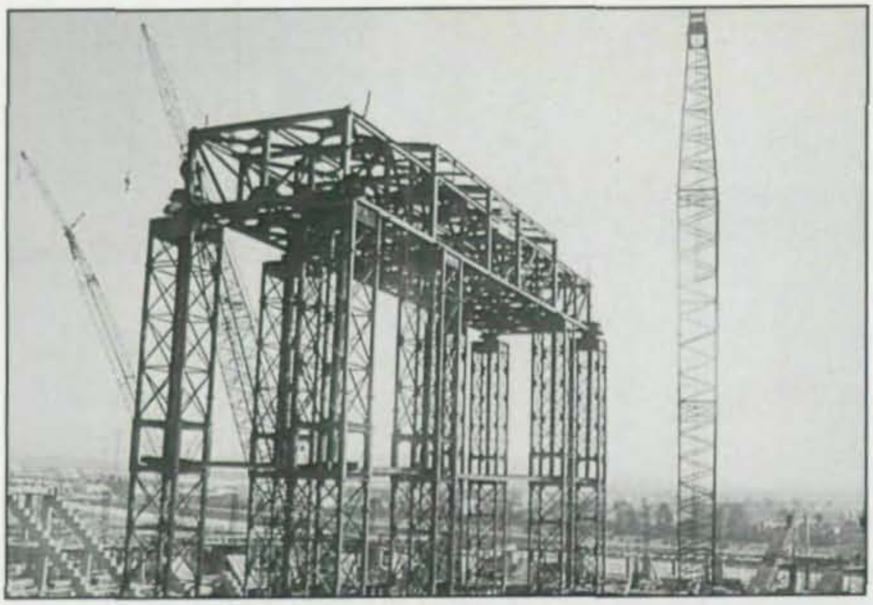
Base isolation also was evaluated and rejected. This system still required an extended concrete frame, but by increasing the roof structure's fundamental period to approximatley two seconds, it reduced seismic forces at the superstructure by approximately 60%. However, with the design requirements for base isolation mandated at that time by code and building officials, the actual benefit would have been smaller. The cost of the base isolation bearings and associated details would also have exceeded the savings from eliminating rocker and slide bearings.

The system finally developed incorporates steel to provide a longer roof period and beneficial ductility without requiring expensive framing or special bearings. Flexible steel perimeter columns rise from the top of the upper concrete seating level, connecting to roof truss and mechanical mezzanine members to form truss girder frames. Other perimeter members connecting to the columns form a circumferential frame. Both frame systems qualify as Special Moment Resisting Space Frames (SMRSF) by code. Together they provide the ductility and inelastic energy dissipation capacity required to resist earthquake forces and deformations in seismic zone 4. This dual steel frame system increases the roof structure's fundamental period to 1.3 seconds, reducing superstructure seismic forces by 50 percent. Its inherent flexibility also accommodates thermal movement without rockers or slide bearings. The columns range from W14x132 to W14x193.

Another criitical element is the steel-to-concrete interface. A deceptively simple-looking baseplate and clip angle connection provides critical performance and efficient erection in an economical manner. It acts as a fixed connection for low-level seismic events, but provides ductile, stable energy-absorbing inelastic behavior during severe events. In addition to good performance, the incorporation of this "simple" connection provided erection tolerance and ease of assembly to help keep the project on schedule and under budget.

Roof Erection

The success of any long-span structure hinges on its erection method. The Anaheim Arena roof erection scheme used eight falsework towers, one at each tied arch intersection, to support the 29-ft.-by-144-ft. center box



and the 150-ft.-long trusses that radiate from it.

The key ingredient of the erection procedure was the location of the central box truss and its shoring towers, which enabled the entire roof structure to be erected without negatively impacting the construction of the 600,000-sq.-ft. concrete seating bowl. The major components and the steps in the erection procedure are as follows:

1. Erection of eight shoring towers within the arena floor footprint, with the compression struts and connection hubs placed within the towers at the onset of steel construction.

2. Shop fabrication of the roof truss sections with a maximum depth of 14-ft. out-to-out to facilitate shipping.

3. Field assembly of the center box on the ground and its erection as a single 130-ton unit utilizing two cranes.

4. Erection and field welding of the perimeter support columns and special ductile moment frame just ahead of the follow-up roof trusses.

5. Dual east-to-west erection of the tension ties between the end of the tie-trusses and the compression strut connection hubs.

6. Erection and bolted connection of the tension ties between the end of the tie-trusses and the compression strut connection hubs.

7. Follow-up placement of the 3-in. roof deck.

8. Lowering of the shores three inches to the liftoff position after the completion of all of the items above.

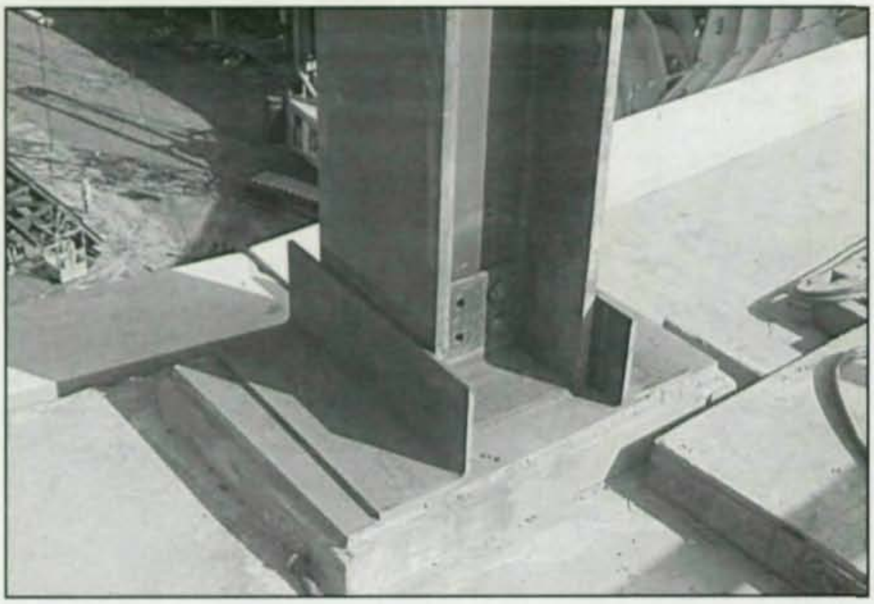
9. Bolting of the slip connection between the truss bottom chord, the mechanical mezzanine and the perimeter columns, which was incorporated to accept the dead load movement at these locations and therefore minimize any potential dead load bending moments in the columns. This made it possible to incorporate the truss-girder frames in the roof's seismic resistance system, which enhanced the system's overall performance.

With all of the roof elements in place, the shores were lowered the anticipated three inches to the lift-off position on June 17, 1992, only three months after the start of erection. This schedule permitted early completion of the project.

Partnering

A key to the success of the project was the tremendous cooperation among all of the parties involved. From day one, a partnering approach from the top down was instituted. As a result, the owner, architect, general contractor and owner's general consultant all worked together and kept the $67 million arena project barreling along towards it completion on June 18, 1993.

Each month the principals from the owner (the City of Anaheim), the architect (HOK Sport), the general contractor (Huber Hunt Nichols), and the owner's representative (Turner Construction Co.), met to grade the job, resolve disputes before they became problems, and ease the channels of communication among all disciplines. The partners maintained continuous contact throughout the process and even organized outings to baseball games. Everyone has



worked on good projects where there was a sense of everyone working together. However, on some projects—and especially on some public works projects—there is often an adversarial relationship between the designers and constructors. Partnering offers a forum for problem solving before the lawyers get involved.

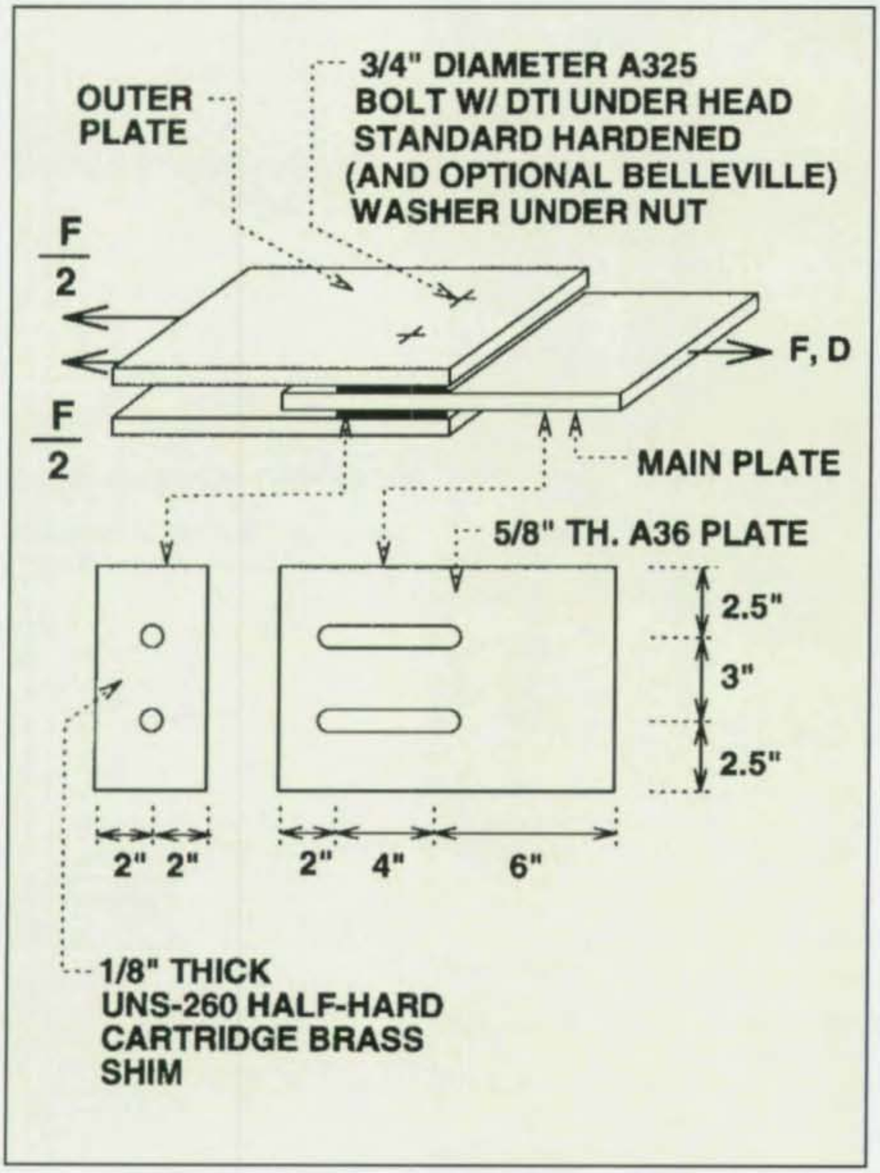
This spirit of cooperation also existed among the steel fabricator, steel erector and structural engineer Thornton-Tomasetti Engineers. Upon award of the steel contract, the fabricator, engineer and other members of the building team began intense discussions on the most efficient and safest erection sequence compatible with the entire construction process for the other portions of the building. The result was a 50-step, 30-page erection procedure prepared by the fabricator and reviewed and approved by Thornton-Tomasetti.

As a result of the teamwork and design, the project was completed ahead of schedule and within budget.

Thomas Z. Scarangello, P.E., is a senior associate with Thornton-Tomasetti Engineers in New York.

Cost-Effective Energy Dissipating Connections

Slotted Bolted Connections show promise of becoming an inexpensive, easily designed method of energy dissipation in seismic areas



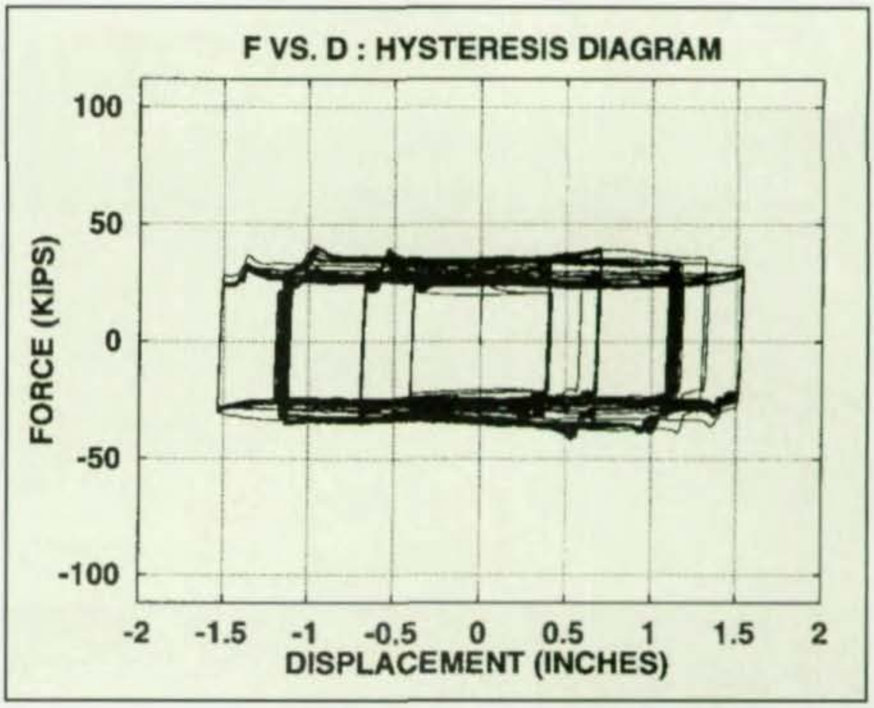
(This article has been adapted from a paper written by Carl E. Grigorian, Tzong- Shuoh Yang and Egor Popov for the University of California (Berkeley) Earthquake Engineering Research Center, which was later appended in Steel Tips, *a publication of the Steel Committee of California.)*

While various types of energy dissipating devices—such as friction damped braced frames and three-stage friction grip elements—have been tested and studied, they have been rejected by the engineering community due to their high cost arising from the difficulty of manufacturer and installation. Development continues on energy dissipating devices, however, and one alternative, the Slotted Bolted Connection, seems highly promising.

The Slotted Bolted Connection (SBC) studied by the authors is a bolted connection where the elongated holes or slots in the main connecting plate (in which the bolts are seated) are parallel to the line of loading. In addition, Belleville and DTI washers were used under the nut and heads of the bolts respectively. Later reserach has shown that the use of the Bellevilles only marginally improves the performance of the SBCs. Two types of SBCs have been studied: one with brass insert plates and one without. Only the SBCs with brass insert plates were found to be reliable as energy dissipation devices.

The performance and function of the SBC is very straightforward. When the bolts are tightened, the main plate is "sandwiched" directly between the brass insert plates and the outer steel plates. The holes in the brass insert plates and in the steel outer plates are of standard size. When the tensile or compressive force applied to the connection exceeds the frictional forces developed between the frictional surfaces, the main plate slips relative to the brass insert plates and the outer steel plates, depending on the type of SBC used. This process is repeated with slip in the opposite direction upon reversal of the direction of force application. Energy is dissipated by means of friction between the sliding surfaces. Application of cyclic loads of a magnitude equal to the slip force results in approximately rectangular hysteresis loops, which demonstrates a consistently repeatable energy dissipation.

More than 50 SBC specimens have been tested using an MTS loading frame and, last year, a sample structure incorporating 12 SBCs was tested on a shake table at UCBs Earthquake Simulator Laboratory.



Pictured above is a figure of force and displacement histories and hysteresis loops for the specimen shown on the opposite page. The displacement hisotry was imposed coaxially on this specimen using the MTS machine.

Experimental Results

Two specimens, both using A36 steel, were presented in the EERC research paper. The steel surfaces were cleaned to clean mill scale condition. The brass plates were of the widely available half hard cartridge brass variety (UNS-260). The test specimens were prepared by a local structural steel fabricator so as to simulate industry conditions. Holes and slots in the steel plates were punched and the edges deburred.

The two specimens described in the paper are each two-bolt specimens using 1/2-in.-diameter, 3-1/2-in. long A325 bolts. The Belleville washers used were 8-EH-112 Solon compression washers, one under each nut. Direct Tension Indicator (DTI) washers were used to ensure a bolt tension in the range of 12 to 14 kips.

The specimens were placed in an MTS loading frame with a ram capable of applying forces of 300 kips statically and 250 kips dynamically, with a maximum displacement stroke of six inches. Both displacement and force control was possible through a controller unit, and a function generator enabled the servoram to produce preprogrammed load or displacement histories. All testing was done under displacement control.

The study revealed a major shortcoming in SBCs with friction between steel surfaces. During the test, there was an almost immediate increase in the slip force followed by a quick drop to a magnitude several times less than the peak slip force. Although this behavior has not been observed in all tests of SBCs with friction between like steel surfaces, it has been present, to various extents, in the majority of cases. In tests with specimens where the mill scale surfaces were polished by wire brushing and those in which the surfaces were roughened and the mill scale removed by sand blasting, this behavior not only did not disappear but was actually intensified. The occurrence of this behavior in SBCs where friction occurs between steel surfaces renders such SBCs inefficient at best, and impractical at worst as energy dissipators.

However, the use of brass insert plates significantly reduces the variations in slip force magnitude, almost completely eliminating the undesirable behavior.

SBCs can be implemented wherever butt-splice (or shear-splice or lap-joint) type connections are feasible. A natural location for use of SBCs in earthquake engineering is in the diagonal or chevron brace-to-gusset connection.

Research has continued since the original publication of the EERC/Steel Tips article. The

Pictured above is the assembled three-story, one-bay test structure supporting 30,000 lbs. per floor and standing more than 20-ft. high. The assembly featured moment resisting frames and Chevron braces connected with steel-on-brass SBCs at each level and on each of the two frames of the structure.

new series of tests include SBCs with $^3/_4$-in.-diameter A325 bolts. Such a test specimen is shown here in an isometric view. The displacement history, D, was imposed coaxially on this specimen with the MTS machine. The connection force response, F, and the resulting hysteresis loops also are shown. The behavior is seen to be, indisputably, elastic-perfectly-plastic.

Shake Table Experiments

A three-story, one-bay steel test structure supporting 30,000 lbs. per floor and standing more than 20-ft. high was tested on a shake table. The lateral force resisting system consisted of two moment resisting frames and Chevron braces connected with steel-on-brass SBCs at each level and on each of the two frames of the structure. The design slip loads for the SBCs were determined by computer simulation of the structure's response to various seismic inputs. The design called for slip forces of 15 kips for SBCs at the first level and 7.5 kips at the second and third levels.

Hysteresis diagrams for the six SBCs on each of the two frames were similar to those obtained with SBCs testing in the MTS testing frame. Calculations showed that nearly 75% of the input energy is dissipated by the SBCs for the case presented in the paper.

The results obtained from the shake table testing verify the practical performance of SBCs. It is evident that SBCs with steel on brass frictional surfaces possess significant advantages in terms of efficiency as energy dissipators and ease of modelling. As such, and with low material and fabrication cost, SBCs exhibit great potential as an alternative choice for energy dissipation in seismic design and retrofit of structures.

Currently, the authors of the report are continuing their studies with the aim of establishing design guidelines for the use of SBCs in real structures.

Blind Bolting

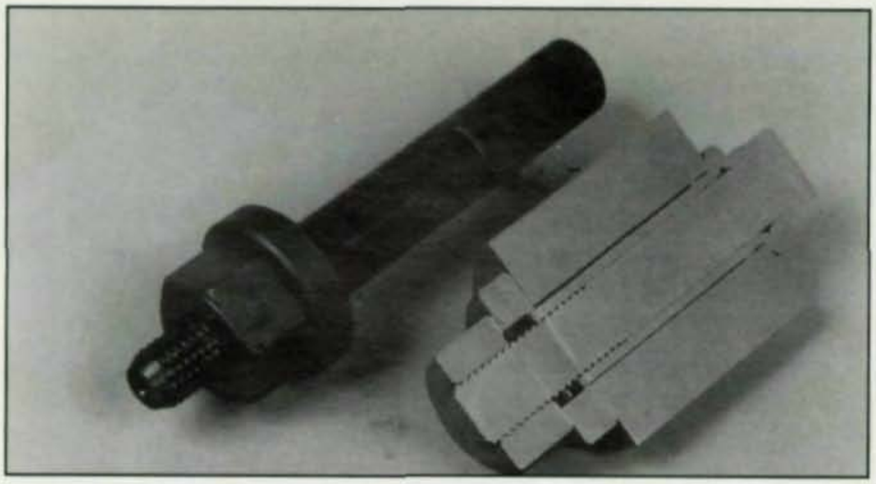
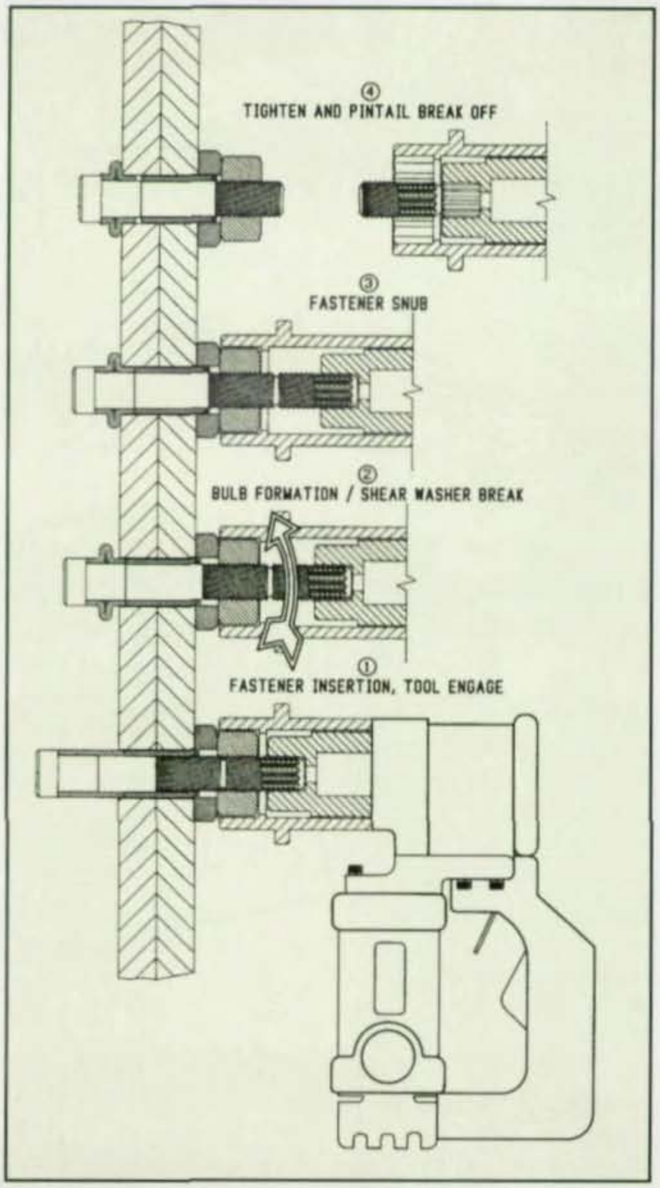


Figure 1 (above): *TBB fastener and an installed cross section of the bolt.*

Figure 2 (right): *Drive sequence of the twist-off blind bolt.*



Twist-off blind bolting can provide an alternative to welding for tubular steel

By Shahriar Sadri

During the past few years, structural tube sections have become increasingly popular, not only for their visual impact and superior strength, but also for their potential cost savings—including savings on material and transportation costs, as well as the savings realized by leaving a structure exposed. In practice, however, the process of joining the tubular sections have proven to be a cumbersome and costly task that often offsets a large portion of the potential cost savings.

Common practices for joining tubular sections consist of:

•Complete field welding of the sections;

•Bolting by cutting access holes in the beams;

•Bolting by using nut plates pre-welded in the shop;

•A combination of the above.

However, there are disadvantages with all of those methods. Welding is expensive, requires skilled labor, slows down the cycle time, is difficult to inspect, and cannot be done under certain weather conditions. Cutting access holes in the closed sections makes the structure weak and the associated joining process is often extremely awkward and slow. Tack welding nuts to a plate then welding it to the inside of the tubular section also is very slow, as well as hazardous, and may result in hole alignment problems in the field.

Post assembly inspection of the joints poses another challenge for the user and is mostly inaccurate and slow.

A new technique, high strength blind bolting—which is used for joints where there is access only from one side of the structure—is being introduced as a practical and economical method for joining tubular sections. Although blind fasteners have been around for many years, they have rarely been used in structural applications due to their lack of a proper strength level that would allow them to make a one-to-one substitute for A325 bolts. However, Huck has recently introduced a new fastener, the Twist-Off Blind Bolt (TBB) that is designed to meet the tensile strength and tension requirements of the A325 Specification.

The TBB is made up of five steel components, each designed to carry out a specific function during the installation and final setting of the fastener in the joint. The drive tool is a standard electric shear wrench used for installation of conventional twist-off (Torque-Controlled T-C) bolts. An installation drive sequence for the TBB is shown in figure 2. Current installation cycle for a 7/8-in. fastener diameter using a standard TONE S60EZ tool is around 35 seconds. A new, faster tool now under development is expected to have a 20 second continuous installation cycle. Bolt snubbing also is possible and is done using a Shear Wrench with a torque controller set at the proper snub level. The installation curve for a 7/8-in. TBB fastener is shown in figure 3.

The installation consists of three distinct cycles: bulbing; snub; and final tightening. All three cycles are identified on the curve in figure 3. The tension development mechanism is "built-in" in the fastener and is not operator or tool dependent, thus eliminating the common under/over torquing problems in the field. Inspection of the

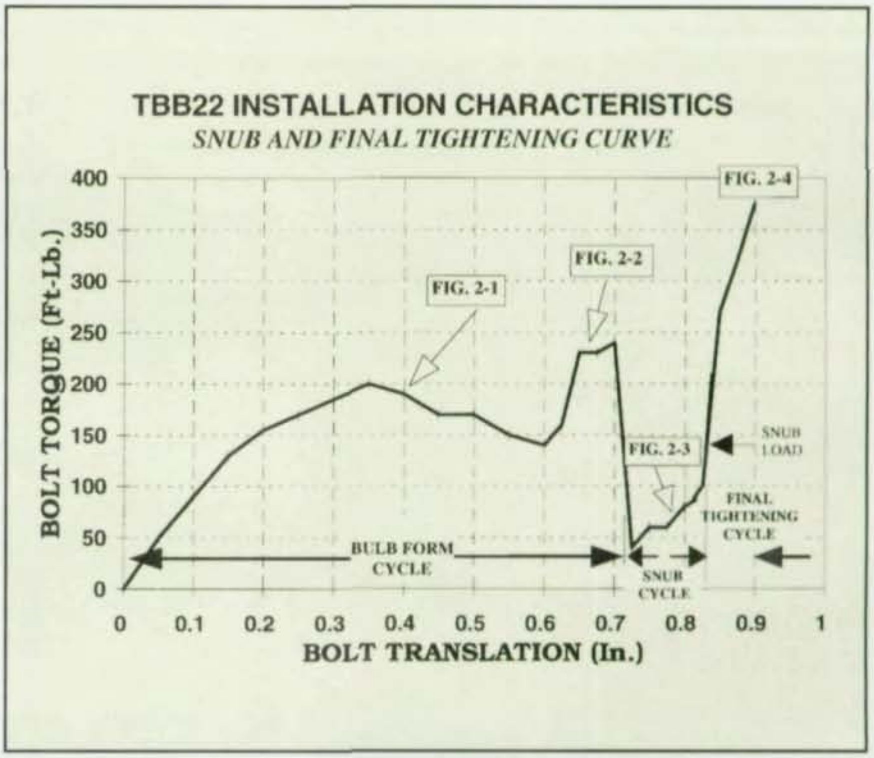


Figure 3 (top): *Installation curve of TBB: bolt torque vs. bolt translation. The installation segments are marked and can be visualized by referring to figure 2.*

Figure 4 (above): *Column-to-column connection. An operator is installing the TBB fasteners. The columns have been sectioned for demonstration of back side formation of the fasteners.*

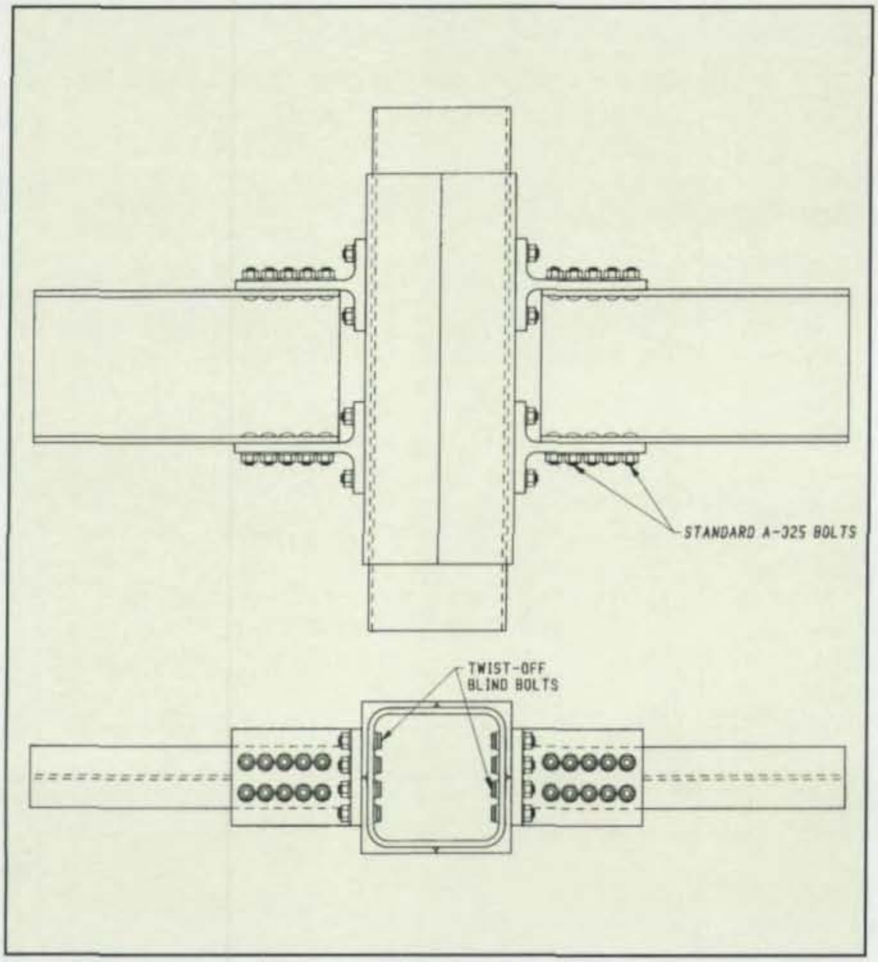
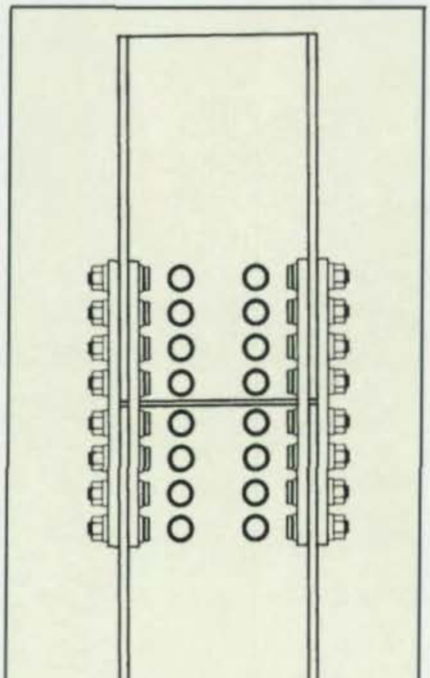


Figure 5 (above): H beam to tubular column connection.

Figure 6 (right): Column-to-column connection using TBB fasteners.



installed fasteners is easy and is done by measuring the protrusion of the bolt after the installation. Too high or too low protrusions will flag the operator of an installation problem. Also, unlike the conventional fastening systems, outside exposure of the TBB prior to installation has no effect on the bolt's prevailing friction factor since the thread system is "protected" within the assembly. Figure 4 shows an actual installation. Note that a section of the tubular column has been cut out for demonstration of fastener function and blind side formation. In this particular joint, conventional twist-off bolts were used for the bottom half of the connection and TBB fasteners were used for the top half.

Applications

In general, twist-off blind bolts could be used in all applications were access to one side of the structure is difficult or restricted. Examples of column-to-beam and column-to-column connections using TBB fasteners are shown in figures 5 and 6. Productivity improvements in the order of two to three times are expected from using the TBB system. The improvements are especially noticeable for the beam-to-tubular column connections, where besides the elimination of welding, diaphragms and cumbersome assembly procedures associated with them will no longer be needed.

The joining process simplification offered by the TBB system allows users to either partially assemble the connections in the shop and finish it on the job site, or do the complete assembly on the job site, thus giving more latitude in terms of transportation options and material handling.

Huck expects the TBB fasteners to be in full production by the end of the first quarter of 1994.

Shahriar Sadri is director of product development with Huck International, Inc.

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.

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.

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.

Type I And Type III Bolts

St. Louis Screw & Bolt operates the only facility in North America offering a full range of Heavy Head High Strength Bolts, including A325 Structural Bolts from 1/2-in. through 1-1/2-in. diameter in all lengths. Everything is produced from steel melted and made in the U.S. to fulfill both Type I and Type III (weathering steel) requirements and feature an "SL" trademark on its heads. For more information, contact: St. Louis Screw & Bolt, 6900 N. Broadway, St. Louis, MO 63147 (800) 237-7059; fax (314) 389-7510.

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. All steel inventory is mill certifiable and made in the U.S. Certification is provided upon request. Also, Fabsco features cold forming capacity up to 3/8-in. diameter in lengths up to 3-in. For more information, contact: Fabsco Corp., 1745 West 124th St., Calumet Park, IL 60643 (708) 371-7500; Fax (708) 371-7524.

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.

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.

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.

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.

Rapid Delivery

Specializing in meeting the needs of structural steel fabricators, 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. Three locations offer next-or second-day delivery to two-thirds of the U.S. Contact: Mid South Bolt & Screw Co., Inc., 499 Cave Road, Nashville, TN 37210 (800) 251-3520.

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.

Threaded Fasteners

Portland Bolt is a domestic manufacturer of headed and threaded fastening hardware up to 5-1/2-in. diameter and 40 ft. in length. The material is produced and certified to the latest ASTM and AISC specifications. The company specializes in carbon, alloy, non-ferrous fasteners and hot-dipped galvanized coatings. For more information contact: Portland Bolt & Manufacturing Co., Inc., P.O. Box 2866, Portland, OR 97208 (800) 547-6758.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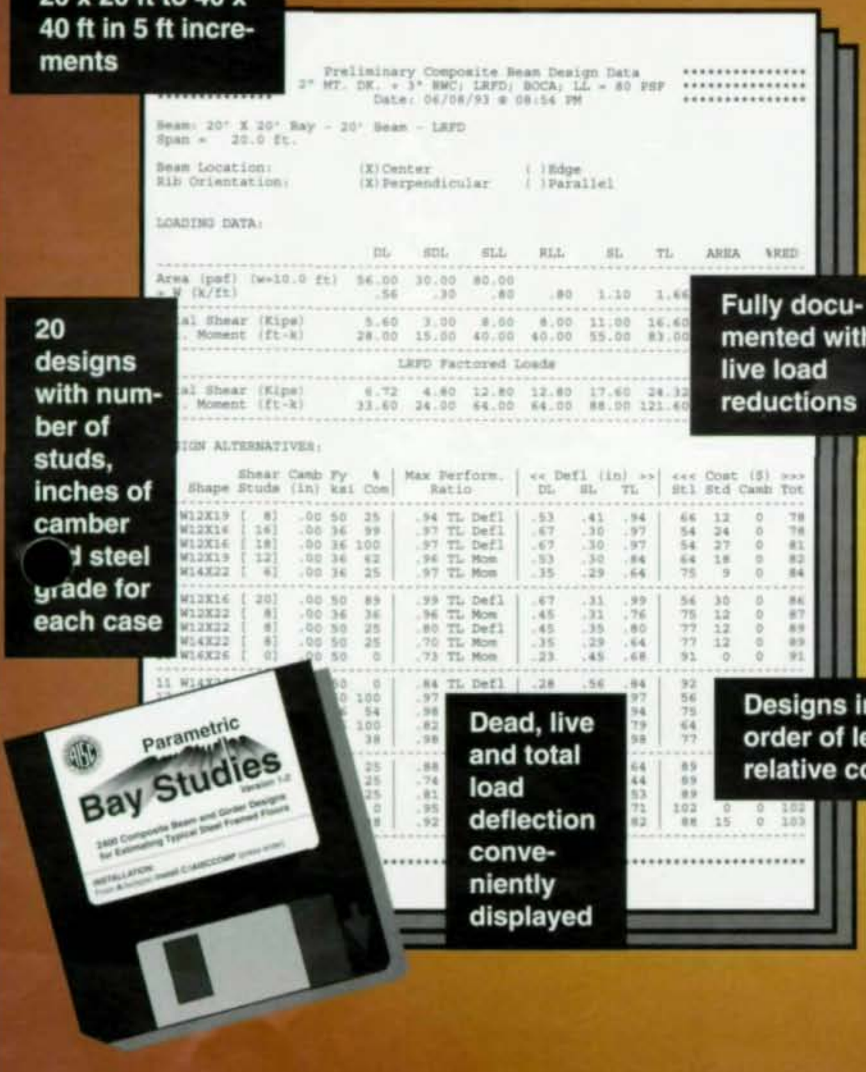

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