MODERN STEEL CONSTRUCTION

October 1993

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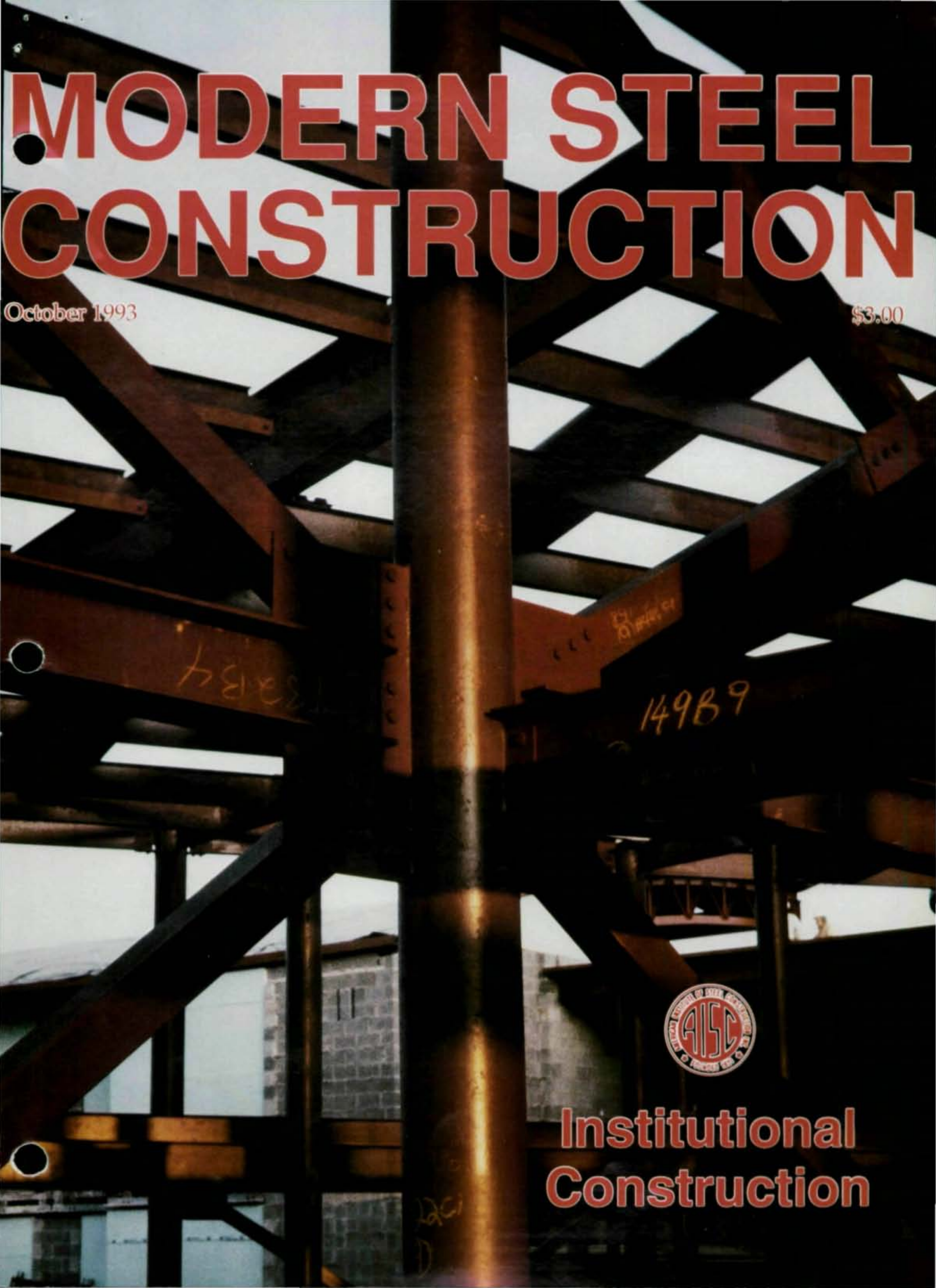

Institutional Construction

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JOIST DESIGN DATA SHEET
No. 2

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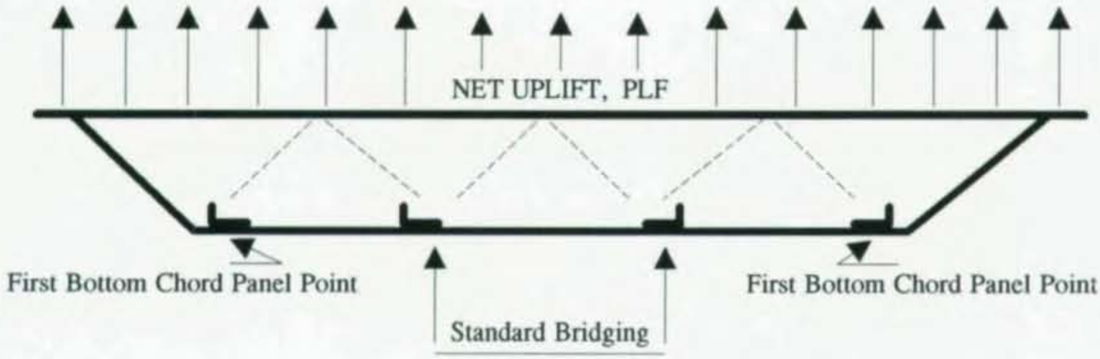
Wind uplift forces must be determined by the design professional and shown on the contract drawings as NET UPLIFT. (The net uplift force on the roof joist is the gross uplift minus the dead load including the joist weight.) This temporary reversal of loading creates compression forces in the bottom chord 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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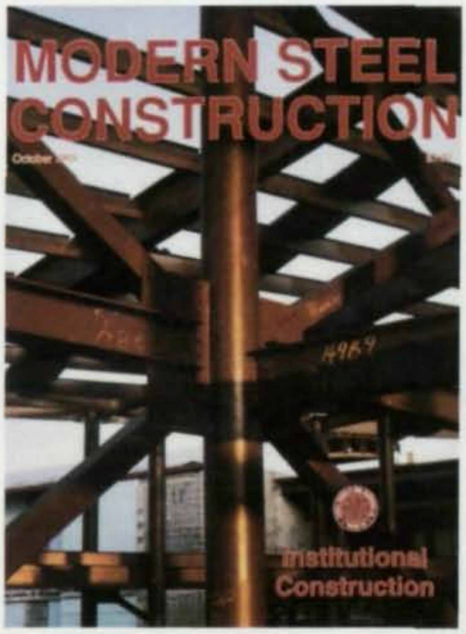
MODERN STEEL CONSTRUCTION

Volume 33, Number 10 October 1993



The designers of a new 160-acre educational campus for the Perry (Ohio) School District utilized both pipes and wide flange, much of which was left exposed to add architectural interest. Photo by William Sheck.

FEATURES

NEWS AND DEPARTMENTS

Modern Steel Construction (Volume 33, Number 10). ISSN 0026-8445. Published monthly by the American Institute of Steel Construction, Inc. (AISC), One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Advertising office: Pattis/3M, 7161 North Cicero, Lincolnwood, IL 60646. (708) 679-1100; fax 708/679-5926

Subscription price:
Within the U.S.—single issues $3; 1 year $30; 3 years $85.
Outside the U.S.—single issues $5; 1 year $36; 3 years $100.

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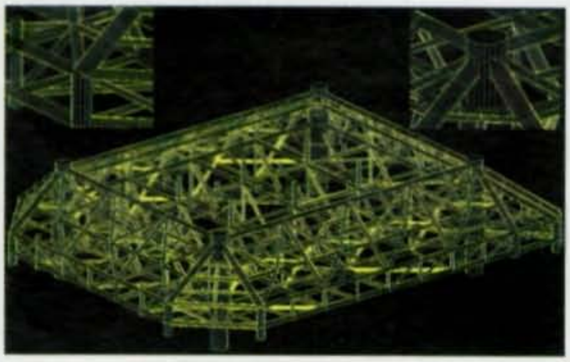
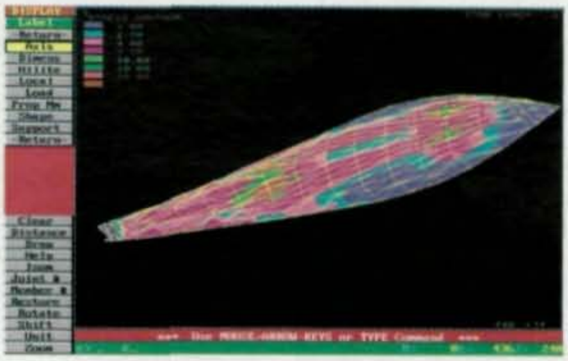
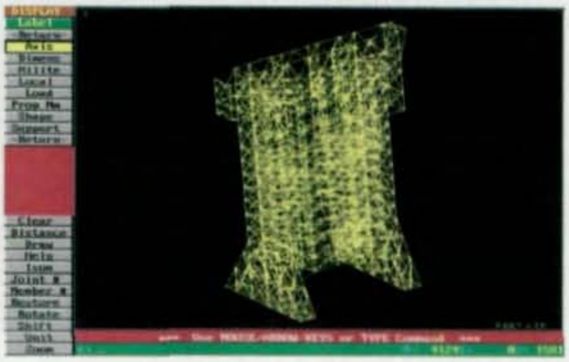


Offshore structure. Courtesy AKER Engineering, Norway



Concrete Canoe, Courtesy Drexel University, Pennsylvania

Roller Coaster. Courtesy Arrow Dynamics, Utah



NASA Launch Pad. Courtesy Brown & Root, Texas

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Scott Melnick,
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Patrick M. Newman, P.E.,
Senior Technical Advisor
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Editorial Offices
Modern Steel Construction
One East Wacker Dr.
Suite 3100
Chicago, IL 60601-2001
(312) 670-5407

Advertising Sales
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Instant Info

Regular readers of this magazine are probably all too familiar with my abhorrence of complicated computerized phone systems that don't provide the information a caller needs and offer no option for reaching a live human being. So it may come as a surprise that I'm now expressing optimism over a new phone information system set up by AISC.

The new system allows callers using a pushbutton phone to dial a toll-free number, punch a couple of buttons, and receive—via a fax machine—information about available AISC literature and software, including manuals & supplements, specifications & codes, design guides, technical & fabricator publications, and conference proceedings.

My support of this system goes beyond the basic consideration of who signs my paycheck. I like this system for what it does—and doesn't—do. It's not designed to answer questions about AISC, steel design, or who shot JFK. Rather, it has a single, clearly stated purpose: To provide information about AISC publications and software.

I also like this system because it represents an actual improvement over the existing situation. Previously, if you wanted an AISC publications list, you'd call AISC's general number, a receptionist would answer the call, you'd make your request, she'd transfer you to the publications department, you'd repeat your request, and someone would finally take down your name and address and mail you a publications list. Maybe.

Now, the system will be completely automated. Minutes after punching in your request and your fax number, the information you request will be available to you. No waiting, no hassles.

The service is now available. You can give it a try by calling (800) 644-2400.

If you have any interest in bridge design or construction, I strongly urge you to attend The National Symposium on Steel Bridge Construction (November 11-12 in Atlanta). The program is one of the strongest I've ever seen for any national meeting. In addition to the expected topics (painting strategies, weathering steel, cost effective design, seismic design, short span bridges), the symposium will feature an international panel on innovative designs. This is an opportunity to hear some of the world's leading bridge experts talk about design. Also, the day before the symposium officially starts, Bob Nickerson will present a full-day course on "Cost Effective Steel Bridges." In addition, a second course, "Painting Strategies for Maximum Economy and Useful Life," will be offered.

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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 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:

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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).

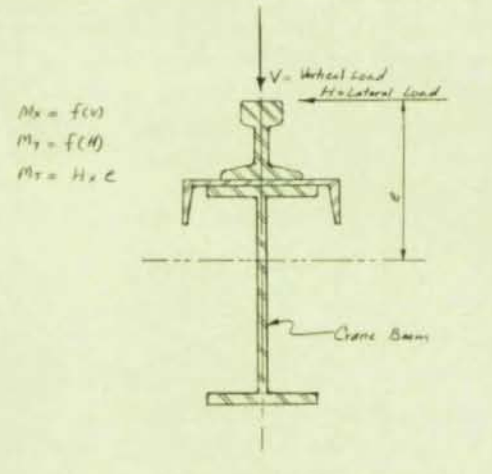
The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer,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

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

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

Under what circumstances does the designer have to consider torsion in the design of a beam?

An example of torsion in the design of a beam would be a beam supporting crane runways. According to the AISC Manual, the lateral force on crane runways (20% of the sum of the weights of the lifted load and of the crane trolley) is applied at the top of rail. This horizontal force (see Figure 1) will result in a moment in the weak axis (M_y) and a torsional moment (M_t = He). In absence of the torsional moment, the beam would be checked for combined stresses as follows:



TYPICAL CRANE RUNWAY BEAM
FIGURE 1

$$f_b = f_{bx} + f_{by}$$
$$= \frac{M_x}{S_x} + \frac{M_y}{S_y}$$

where M_x is the moment in the strong axis caused by gravity loads

To include the effect of torsion, complex calculations can be performed. Or, a simple procedure for handling torsional moment, for a simply-supported beam would be to halve the I_y (or double M_y) since the horizontal load is acting on one flange. Thus the total combined stresses can be calculated as follows, and compared with the allowable bending stress:

Total bending stress

$$= \frac{M_x}{S_x} + \frac{M_y}{S_{y/2}} = \frac{M_x}{S_x} + \frac{2M_y}{S_y}$$

This approximate method can be applied safely to relatively short members because results will be overly conservative in case of long spans.

Vijay P. Khasat, P.E.
Ohio Edison
Akron, OH

How can one take into account blast effects in the design of steel structures?

I am certain that the recent bombing of the World Trade Center in New York has generated a substantial amount of interest in this subject. Obviously, a great deal of local failures occurred in the vicinity of the explosion, but when considered as a whole, the structure performed admirably. One concept that must be grasped, is that the structure will have to absorb a tremendous amount of energy, in a very short period of time. Pressures in the immediate vicinity of the detonation can reach into the thousands of psi, and last for fractions of a millisecond. There are a number of important parameters that must be examined before one can dive headlong into the study of structural response to explosive incidents. For example:

- What type of explosion is being considered? In addition to high explosives, such as TNT or some of the other common energetic materials used by the military, a structure can be subjected to a gas or vapor explosion; dusts, such as coal or grain; or, rupture of a pressure vessel, such as a boiler. Each of these incidents have distinctive "pressure-time" history relationships. which influence the magnitude and duration of the loading and the response of the structure. Although each of these occurrences exhibit a loud "bang", which we experience as an "explosion", accompanied by heat, fireball, smoke, flying debris, etc., they are actually quite different.

- Where does the load originate? Does the explosion come from within the structure, or is it external? If the explosion occurs from within, a residual gas pressure, with a duration substantially longer than the initial pressure pulse, is formed, adding to the total energy the system must absorb. This gas pressure must be vented through the openings in the structure, similar to the air escaping from the neck of an open balloon. For an explosion external to the structure, the gas pressure does not form, making the loading less demanding. What does become a concern is the orientation of the element under investigation with respect to the source of the explosion. If the shock front passes parallel to the element under consideration, the peak pressure is substantially less than if the shock front hits normal to the element.
- What type of support conditions are present in the structure? This will affect how rapidly the system responds to the loading, and hence, how much load the system will experience. What about fragments? Small, shrapnel sized primary fragments generated from the explosives' casings can reach velocities similar to those created by high powered rifles. Light metal panels, curtain walls, even masonry walls, will provide little resistance to these fragments. Large, relatively slow moving secondary fragments can be "chunks" of concrete, masonry, steel columns, bolts, building furnishings, etc., and cause gross structural damage.

The subjects that could be discussed regarding the behavior of steel structures under blast loading can and have filled volumes on the subject. Several good texts exist on the topic, the great majority of them the result of research conducted by the Department of Defense. Of particular note is TM 5-1300, "Structures to Resist the Effects of Accidental Explosions, Vol. 5, Structural Steel Design." This volume, together with its five companion manuals, serves as a basis for the analysis, design, and detailing of structures under blast loading.

Richard P. Linck, P.E.
Booker Associates, Inc.
St. Louis, MO

If a pin hole in a lifting lug is flame-cut, should the net section be reduced to compute the capacity of the lug?

The author of the question is concerned with tension failure at the sides of the hole. This is the only one of four possible lug plate failure modes which would be influenced by the method of producing the hole. An electronic or template-guided cut, when made in the shop under controlled conditions using oxy-fuel or plasma, will produce a clean, well-finished cut. A careful ironworker can manually produce a satisfactory hole. The heat affected zone bordering the cut has no mere adverse affect that does that bordering a weld, and should not be a concern in lug design.

However, pin holes are often required to be burned or enlarged in the field under adverse conditions. Such holes are apt to be irregular and have sizeable notches. If the notches are in the sides of the hole where the tension stresses are highest, problems could develop.

It should be remembered that lifting lugs are a fracture critical part of the load path. They are often overloaded and abused. It is prudent to make sure their strength is adequate. A lifting lug accounts for such a small portion of the overall steel weight that it is unwise to skimp on the design.

Further information on lug design and references can be gotten from "Design and Construction of Lifting Beams", Engineering Journal, 4th Quarter 1991 (Vol. 28, No. 4)

David T. Ricker, P.E.
Payson, AZ

If a pin hole in a lifting lug is flame-cut, should the net section be reduced to compute the capacity of the lug?

It appears that the difference between machining and flame cutting is a factor. "Design of Machine Elements", M. F. Spotts, 4th Edition, Prentice-Hall, 1971 implies a surface hardening because of the flame cut.

Dennis C. Delonay, P.E.
Dennis' Design Service
Rib Mountain, WI

New Questions

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

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

Are there any references on designing composite columns made up of wide-flange shapes encased in concrete to form a round column?

A tubular shape is used to support a sign, the closed shape is very torsionally stiff and the design works. However, an access hole is needed in the column which creates an open shape that is not resistant to torsion. How big an access hole can be used before torsion must be considered?

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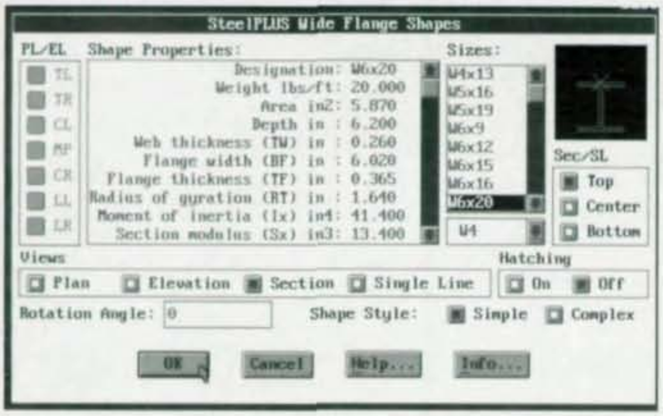

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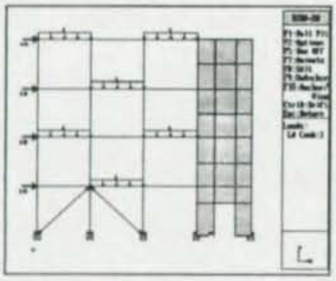

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The seminars are scheduled for:

Phoenix	10/13
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Columbus	10/20
Memphis	10/20
Cincinnati	10/21
Des Moines	11/3
El Paso	11/16
Oklahoma City	11/18

For more information, call AISC at **(312) 670-2400**; fax 312/670-5403.

It's also not too late to attend the "Steel Design Seminar Series: Design of Steel Connections", conducted by the **Steel Structures Technology Center**. The one-day, professional level program discusses joint analysis methods, design criteria and methods, constructability and economical design. Seminars are scheduled for:

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North Haven, CT	10/5
Boston	10/6
Portland, ME	10/7
Portland, OR	10/25
Seattle	10/26
Kansas City	11/4
Costa Mesa, CA	11/29
Los Angeles	11/30
San Francisco	12/2
Sacramento	12/3

The fee for the seven-hour seminar is $145. For more information, contact: Steel Structures Technology Center, 40612 Village Oaks Dr., Novi, MI 48375 **(313) 344-2910**; fax 313/344-2911.

Although the Colorado Rockies' inaugural season is all but over, structural engineering and baseball fans alike will still want to attend a breakfast seminar on "The Development and Design of Colorado Rockies Ballpark on October 5. The talk, to be held in Westminister, CO, will feature both the structural engineer and a member of the Denver Ballpark Commission. For more information, call Jim Anders at **(214) 369-0664**.

Roberto **Leon's** long-awaited T.R. Higgins Lectures on "Partially Restrained Connections" commence in October. Leon, a professor of civil engineering at University of Minnesota, gave his initial lecture at the National Steel Construction Conference last March to a wildly enthusiastic audience. Kicking off the series, the Kansas City Area AISC Fabricators hosts Leon's presentation on October 21 in Kansas City. The next day, Leon will lecture at the Texas Structural Steel Institute Annual Conference in Houston. Then, on March 15, 1994, he'll give two lectures. The first, a breakfast seminar, is hosted by the Steel Institute of New York. That evening, he'll give a dinner dinner in Boston, sponsored by the Boston Society of Civil Engineers Section of the ASCE. For more information, contact the host groups or AISC at **(312) 670-2400**.

Tubes continue to be a hot issue with structural engineers and fabricators. **Donald R. Sherman**, Chairman of the Department of Civil Engineering at the University of Wisconsin (Milwaukee) will give two talks in October on "Connections to Tubular Columns Without Through-Plates." The first will be October 1 in Chicago, and the second will be October 15 in St. Louis. For more information on the Chicago presentation, call Bill Liddy at **(708) 527-0770**, and for information on the St. Louis presentation, call Ann Stewart at **(314) 638-5000**.

AISI and AISC Marketing are sponsoring a bridge training course featuring **Robert L. Nickerson**, former Chief of the Structures Division of FHWA. The full-day course, "Cost Effective Design of Steel Bridges," includes four modules: Design & Detailing; Material Selection; Fatigue and Fracture—Design & Retrofit; Joints, Scuppers & Innovative Design. Course material is based on actual case histories. The courses currently are scheduled for:

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Baton Rouge, LA	10/26
Austin, TX	11/4
Jefferson City, MO	11/19
Sacramento, CA	11/30
Olympia, WA	12/2 & 12/3
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S T E E L   C A L E N D A R

Bridge engineers and fabricators in the Nashville area won't want to miss the next meeting of the **Steel Bridge Forum** on October 28, which will feature **Nickerson**'s presentation on "Cost Effective Design of Steel Bridges" (see above). For more information, contact: Camille Rubeiz, American Iron and Steel Institute, 1107 17 Street, NW, Suite 1300, Washington, DC 20036-4700 **(202) 452-7100**; fax 312/372-0873.

For high-rise building aficionados and fans of building fiascoes, the **Chicago Committee on High-Rise Buildings** is sponsoring an all-day seminar on "Amoco Building Recladding—Lessons Learned." For more information on this November 11 meeting, call Ian Chin at **(312) 372-0555**; fax 312/372-0873.

One of this year's highlights for anyone interested in bridge design or fabrication is **The National Symposium on Steel Bridge Construction** in Atlanta Nov. 11-12. Topics include: state plans for implementing metric conversion to meet FHWA mandates; paiting strategies for maximum economy and useful life; weathering steel success stories; cost effective design and details; seismic design; bridge research; and short span steel bridges. In addition, the symposium will feature an international panel on innovative designs featuring: **Yuhshi Fukumoto** from Japan; **Jean Muller** from France; **Ken D. Price** from Canada; **William Ramsay** from Great Britain; **Charles Seim** from the U.S.; and **Leo Spaans**, also from the U.S. Continuing education credits will be offered for attendees at the various lectures. Also, two exciting workshops will be held the day before the Symposium begins. **Bob Nickerson** will present a full-day course on "Cost-Effective Steel Bridges" (see above) and **Eric Kline** from KTA-Tator will moderate a full-day workshop on painting. Registration costs $275 plus $50 for the optional pre-symposium workshops. For more information, contact: AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001 **(312) 670-2400**; fax 312/670-5403.

If you're already planning your schedule for 1994, you might want to mark February 23-25 on your calendar. The **NJDOT** and the **FHWA** are sponsoring "Structural Materials Technology: An NDT Conference." The conference organizers' aim is to acquaint as many people as possible with the state-of-the-art in nondestructive testing and evaluation and to demonstrate where the technology is heading. Both steel and concrete will be covered. To receive a registration packet, contact: Conference Registration Committee, P.O. Box 77352, Trenton, NJ 08628; fax 609/633-6924.

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Complicated Curves

While the curved wings of the Jefferson County Government Center create a welcoming architectural image, they greatly complicated the structural design

By Richard G. Weingardt, P.E., and John F. Davis, P.E.

By Richard G. Weingardt, P.E. and John F. Davis, P.E.

As if dealing with the high winds, harsh temperature changes and devastating expansive soils on a high Colorado plateau wasn't enough of a challenge, the designers of the new $60 million Jefferson County Government Center also opted to design a curved structure.

Despite these obstacles, however multi-story administrative center and courthouse was completed on time and within budget.

The Government Center is the focal point of a 200-acre county government campus serving a rapidly growing county of 450,000 people located between the City of Denver and the Rocky Mountains. Its two curved wings—one administrative, the other judicial—are joined by a dramatic 12-story-high, glass-enclosed lobby rotunda.

Much of the design is symbolic of the building's functions. The glass domed rotunda acts as a beacon to visitors during the day and a glowing lantern at night. The tow wings were intended to "reach out" and welcome visitors; to symbolize the idea that government is a servant of the people. The configuration represents the cooperation between the judicial and administrative branches and the return of the county seat to its former prominence as the heart of the community.

The 531,000-sq.-ft. building's semi-circular pan provides the greatest opportunity for dramatic views while also creating very efficient operations. The administration wing houses 26 departments and the courthouse wing contains 27 courtrooms. In addition, the curved design fits in with the Jefferson County Human Services Building, which had earlier been constructed in the complex (see November 1992 *MSC* page 24).

One of the design innovations incorporated into the building is a sophisticated circulation plan for the judicial wing. To increase and simplify security, the courts building has three separate circulation systems: one for the public, which feeds into the courtroom area and assures controlled access and secu-

The circular plan of the 531,000-sq.-ft. Jefferson County Government Center was designed both to maximize views and to create a very efficient layout for court and administrative operations. Photo opposite by Ed Leland.

rity in each courtroom; one for prisoners who are brought through a concealed underground tunnel and secured elevators from the jails to secured holding areas connected to the courts; and one for the judges, attorneys and clerks. The only time the public, court staff and prisoners are in the same room is while court is in session.

"It was a very complex building to design, especially because of the three separate circulation systems," according to Curt Fentress of C.W. Fentress J.H. Bradburn Associates, Denver, the project's architect.

The design sophistication of the circulation plan continues throughout the building. Extending from the rotunda, public corridors serve both wings and give the visitor both building orientation and spectacular views of the landscape. Elevator lobbies on each floor provide overlook points for viewing into the lobby rotunda, thereby making the visual continuity of the public circulation path complete. Externally, these public circulation paths are delineated with differentiations in massing, window size and trans-

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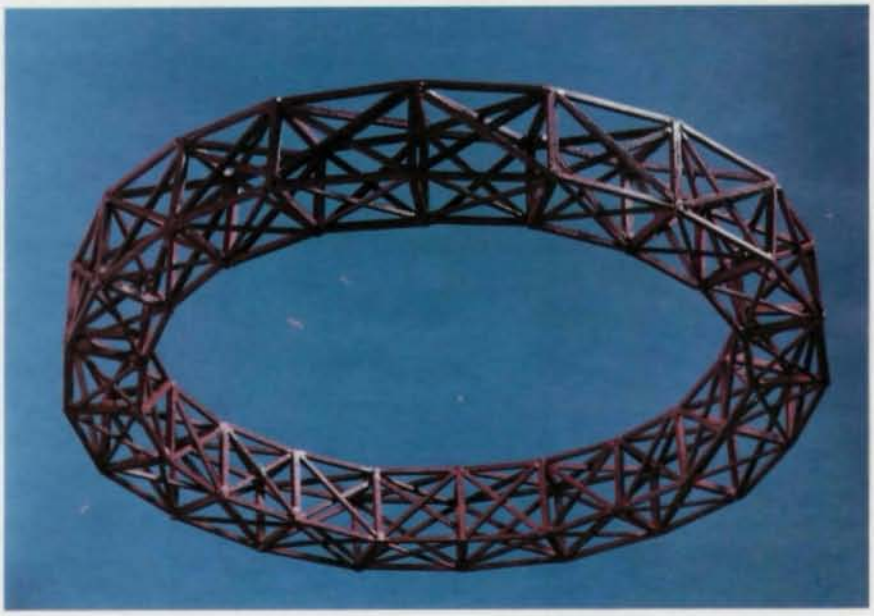


parency, and building skin color. Architectural precast concrete panels were used as exterior cladding with colors selected to reflect those of the surrounding landscape.

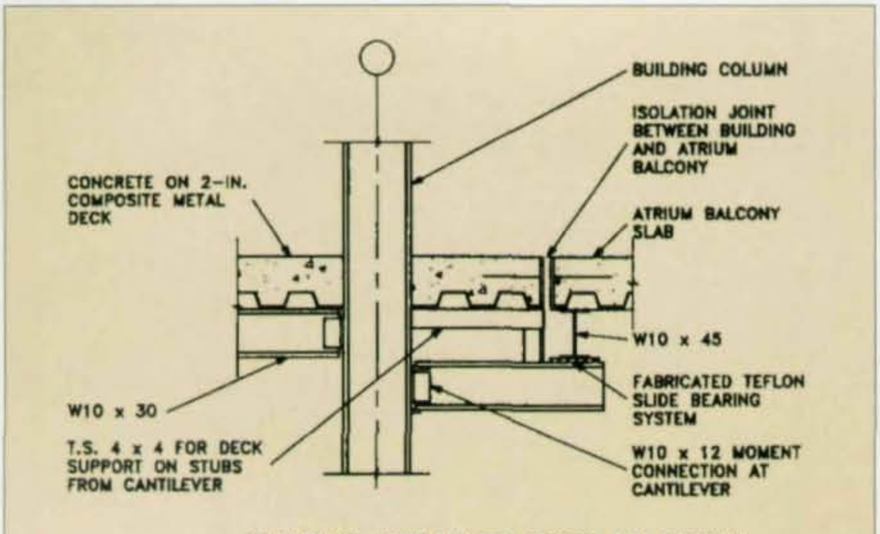
Completing the circulation plan is a drive-up window, which allows residents to conveniently pay fines and taxes and to complete motor vehicle registration renewals.

Complicated Structure

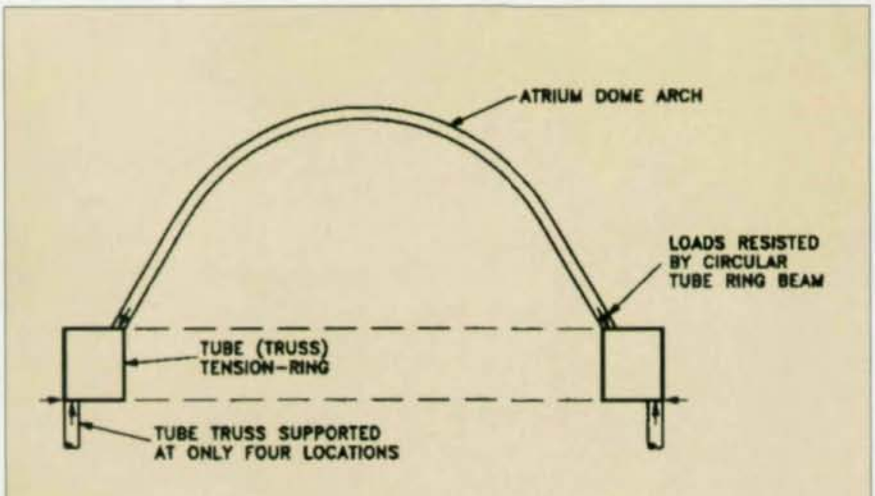
While the circular plan of the structure enhanced the building's appearance and functionality, it complicated the structural framing. There are virtually no right angles—and no room for error. All layouts required precise geometry and advanced surveying techniques. Most of the 3D frame analysis and design was done using STAAD-III.

Also, the massive structure was engineered to be supported on drilled piers, located to miss the seams in the highly fractured bedrock at the site. Special structural connections for curved beams had to be designed to provide torsional restraint, in addition to resistance of the usual vertical forces.

Perhaps the most unusual design feature is the rotunda dome, which features a horizontal tube



BUILDING ISOLATION JOINT AT ATRIUM



CROSS-SECTION OF DOME

tension ring beam. The ring beam is hollow to allow mechanical ducts to pass through unhindered. The circular tube consists of three-dimensional trusses spanning between only four main supports, 100′ apart. The tension ring tube truss consists of TS 8x8x¼ chords with TS 7x7x¼ and TS 14x10x½ webs. Extremely tight fabrication tolerances had to be maintained to allow for field erection and connection. The ring was fabricated in quarter segments, erected on the support cores, and field welded with full penetration welds. Following tube erection, the dome arch members and cladding support elements were constructed and erected.

Fabricator on the project was AISC-member PKM Steel Service, Inc.

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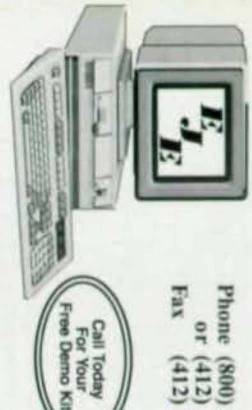
The use of the tube ring-beam



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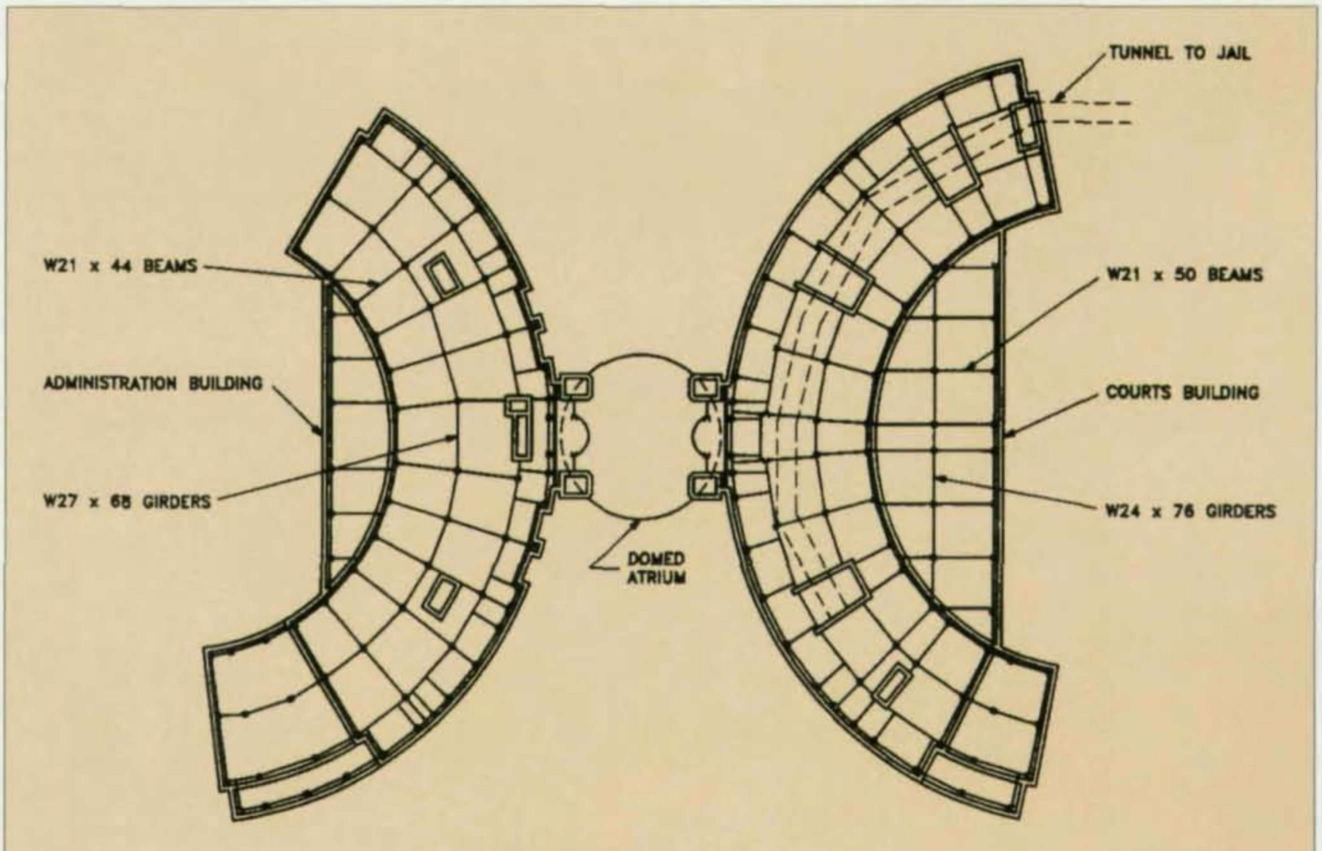

TYPICAL FLOOR FRAMING PLAN

was very cost effective and made the award-winning architecture of the building possible.

The structural frame for the two wings is generally comprised of A572 Grade 50 rolled steel sections. Floor framing is composite construction utilizing light-weight concrete on metal deck supported by steel beams and girders. High-strength wide flange columns transfer the vertical loads to the foundation walls and interior drilled piers. The semi-circular plan orientation of the administration and courts building resulted in the purlins being laid out in a modified radial configuration with the supporting girders being chord segments of the circular longitudinal grids. Due to this configuration, beam spacing varied from 6′ to 10′ on center. The roof level framing follows the same general configuration; however, the concrete topping is not used and open web steel joists are substituted for the wide flange sections.

Typical floor framing beams are W10x30 for the outside bays and W18x35 for interior bays. Typical perimeter girders are W21x50 and interior girders are W27x84. Roof joists are typically 26K6 at 5′ on center. Typical interior columns are W14x159 while those in the exterior walls are W8x67.

Additional complexity was created by the need to provide large open ares that varied from floor to floor. Long span W36x300 girders were required to provide these column-free spaces.

The project included 3,300 tons of structural steel, joists and decking. General contractor on the project was PCL Construction Services, Inc.

Foundation Design

Due to the highly expansive clay soils on this site, a drilled pier foundation system was utilized. These reinforced concrete piers extend up to 35′ into the claystone bedrock, the elevation of which varies greatly across the site. Steps in the building height, along with the framing plan configuration, resulted in the need for careful attention to bedrock penetration to minimize differential uplift concerns between lightly loaded and heavily loaded portions of the building. The need for very deep foundation excavation for the 580′-long tunnel connecting the Courts Building with an adjacent jail facility resulted in deep back fill areas that needed to be considered during the drilled pier design and construction.

Connection Design

Lateral resistance for wind and seismic loading is provided by a combination of moment resistant steel framing and concrete shear walls. The length of the circular main buildings resulted in their being divided into three segments, each separated by building isolation joints. Each segment was considered with its own lateral system independent of adjacent segments. However, to minimize differential lateral movement under design loading conditions, a shear connection detail was developed that allowed longitudinal movement between segments but limits lateral movements across the joint.

The lateral systems were then reevaluated for their performance assuming shear transfer across these joints. Double columns were not used at the isolation joints; rather, single columns with slip joint haunches were designed. Teflon pads were used between the girders and the haunches to accommodate structural movements.

Another important structural consideration was the design of the courtyard and plaza site construction elements. These highly visible and detailed elements also needed to be protected from excessive movements caused by the expansive clay soils. Thus, many of these elements were designed with drilled pier foundations extending into the bedrock strata. The location of isolation joints between structural segments of these elements were coordinated with the landscape architect. A major feature of the entry courtyard is massive stone slabs that appear to jut from the soil at differing acute angles. These cantilevering stones are supported by large footing elements to resist overturning and utilize a two-stage construction sequence to allow positioning of the stones at the proper angle prior to completing the support foundation work.

Expansion capacity has been provided for the next 20 years. As currently constructed, the center is able to serve more than 2,000 visitors per day. However, the structure also is designed to accommodate an 80,000-sq.-ft. addition.

Though only completed in January 1993, the building has already won two major awards: a national AIA Citation for Excellence and a CECC Engineering Excellence Award.

Richard G. Weingardt, P.E., is president and John F. Davis, P.E., is vice president of Richard Weingardt Consultants, Inc., a large Denver-based structural engineering firm.

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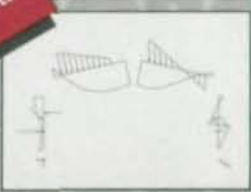

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Fresh Start

A new educational campus in a Cleveland suburb makes extensive use of exposed structural steel

By Dena R. Traina, P.E., and Robert S. Macholl, P.E.

In a bold move for a small community, a single 160-acre educational campus is being developed to replace the aging school facilities of Perry, OH. The project consists of separate facilities for grades K-4, 5-8 and 9-12, a physical education wing that doubles as a community fitness center, and outside athletic facilities for a variety of sports. Each school functions as an individual facility, with its own administrative offices, classrooms, laboratories, dining halls and gymnasiums. Art and music programs and the libraries are housed in special arc-shaped wings.

Designed by prime consultant Burgess & Niple, Ltd., Columbus, OH, with architectural consultant Perkins & Will, Chicago, the Community Education Village for Perry Local Schools in Lake County, OH, will house the school districts' entire student population as well as serve as a recreational, fitness and cultural center for the entire community.

The village is being developed in phases. The first phase, begun in April 1991, is composed of the high school, physical education/community fitness center, central mechanical plant and maintenance facility. Designed for a future high school student population of 1,000, the two-story, 390,000-sq.-ft. building includes an 850-seat auditorium with complete audiovisual capabilities and a 2,000-seat gymnasium. Recently completed, the high school welcomed its first students in September. Current high school enrollment is 350.

The design of the second phase, which includes the middle and elementary schools, has been completed and construction began in June.

Design Concept

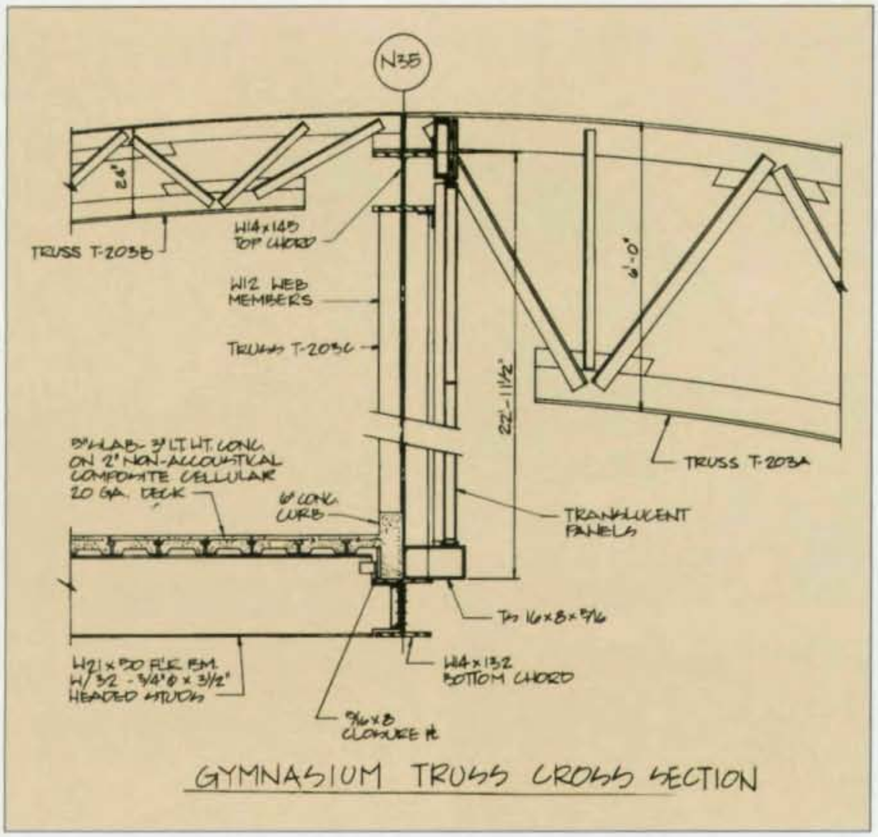
The architectural design of the village leaves much of the structural steel exposed to provide visual texture. As a result, an extensive design effort was necessary from the engineers to provide the desired visual effects. Every exposed detail was scrutinized to ensure that it conformed to the architectural intent, and often the design was dictated by proportion rather than strength.

The new 160-acre Community Education Village in Perry, OH, includes modern classroom facilities, as well as a theater and recreational areas. Many of the special purpose spaces, such as the arts/music/library wing pictured above and on the opposite page, utilize non-parallel columns and required a coordinate system for layout. The 850-seat theater, shown at left, is designed to incorporate all of the qualities of a professional facility. X-bracing was necessary to minimize drift. Finished photography by William Sheck.

The exterior design is an interplay between light steel segments and massive masonry elements. The connections between the steel columns and beams are clean and unobtrusive to maintain a sleek, narrow appearance. The beam-to-column connections are moment connections to resist wind forces. The connections are created with full penetration welds in the top and bottom flanges. A shear tab on the unexposed side of the beam was used to erect the beam. Once in place, welds were made and backer plates were removed and ground smooth. This provides a simple, visually strong connection that meets both the structural engineers' and the architects' requirements. The engineers used STAAD-III for design and analysis.

The clean appearance extends to the top of the structure. Roof edges were limited to an 8" dimension, which included the depth required for the insulation, roof deck and the structure supporting the deck. In places, the roof overhangs its supports by 5'.

WT sections are cantilevered from the structure to support the



deck. The WT sections are located at 8' on center to match the spacing of the exposed structural steel, which results in two conditions: either the WT is directly above a beam or, more critically, the WT is perpendicular to the main beams. In the latter case, the load from the WT results in a torsional force in the main beam. Each sheet of deck had to be cut to fit within the WT sections. The overall structural depth of the WT and deck is 3", leaving room for the insulation and roofing. The result is a sleek, visually appealing roof edge.

Interior Details

Interior space within the village is composed of two-story areas, such as the dance studios and locker rooms, and one-story areas, such as the natatorium and gymnasium.

The one-story spaces are quite tall, with heights ranging from 22' to more than 50'. As a result, X-bracing was necessary in certain lo-

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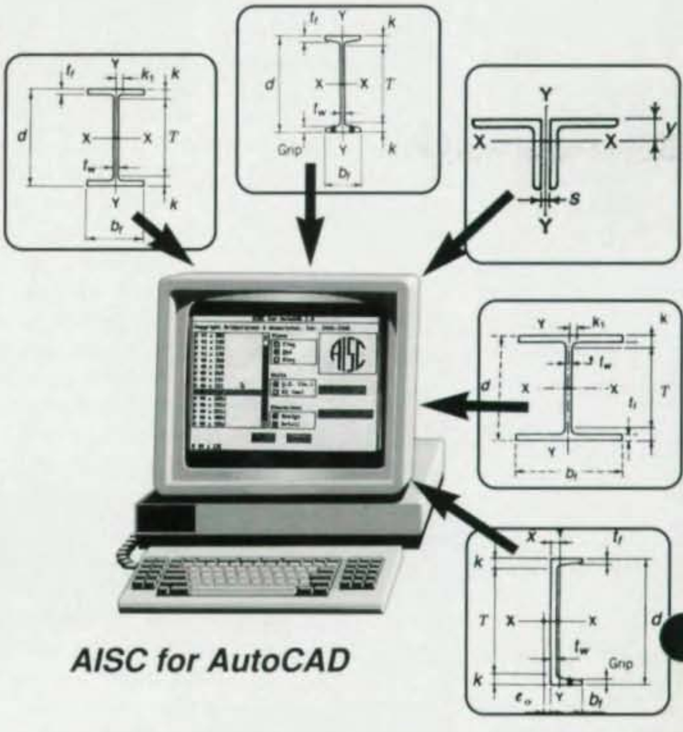

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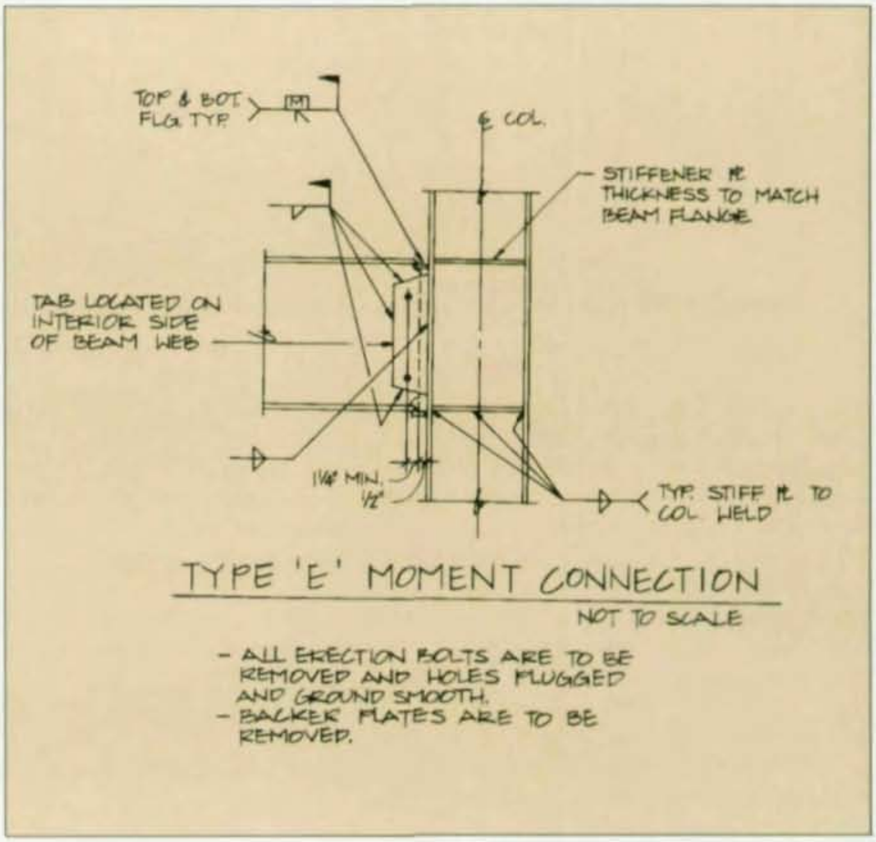
cations to minimize drift. Some of the bracing has been left exposed and included as architectural elements, while other bracing is hidden in walls. The exposed X-bracing is composed of steel rods, joined in the middle with a circular plate to add texture and to avoid chattering of rods moving across one another. This treatment is repeated throughout the village to provide architectural continuity.

A variety of methods was used in many of the tall spaces to visually break up the height. For example, in some areas canopies are located approximately 10' above the finished floor. To maintain visual unity, the canopies were designed to reflect the appearance of the roof edges. However, in some areas the canopies support the windows and walls above, making it difficult to achieve the same effect as the roof edge due to the greater loads and restrictions on deflection. To achieve the desired thin appearance, the depth of the structural



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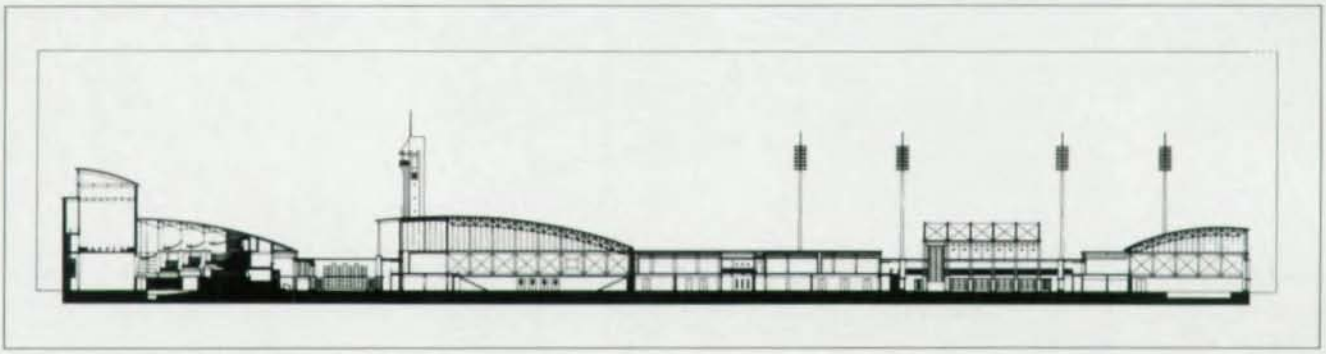

members was limited to a maximum of 6", with the majority of the members being 4" channels. Tubes were utilized to support the weight of the glass and the wind load.

The main lobbies are two-stories high and create an open atmosphere for the adjoining second floor walkways. The structural steel for these walkways is designed as an architectural feature, which meant the supporting steel needed smooth, clean lines. The connections were welded with the backer plates removed and ground smooth, as described earlier. Handrails on these walkways are custom designed to keep in concept with the remainder of the building. Stiffeners were added to the walkway support beams to align with the handrails and were designed to ensure that they not only met code requirements but would also maintain their appearance under heavy use.

Other significant areas include the pressbox, art/library/music wing, theater and gymnasium.

Pressbox

The pressbox for the 4,000-seat football stadium has space available for the press, a conference room, videotaping room, and strategy rooms for the coaches.

The large overhanging pressbox creates a visually exciting effect and is focal point for the village. The wind uplift load on the overhang was carefully analyzed to ensure the dead load of the structure could counterbalance the uplift load. Additional concrete was added to the roof structure to ensure adequate balance. This was necessary because rods supporting the overhang cannot withstand any

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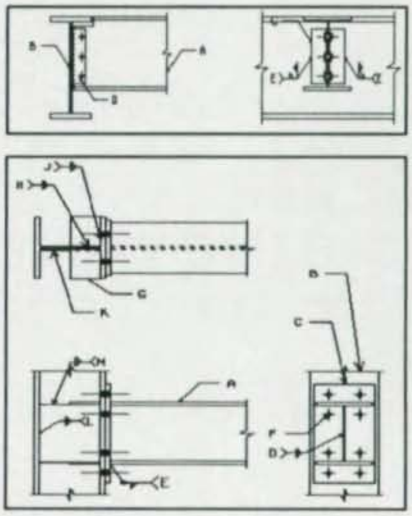

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compressive forces. The proportions and weight of the overhang results in the rear columns being in uplift. This is counterbalanced by using large, deeply buried footings.

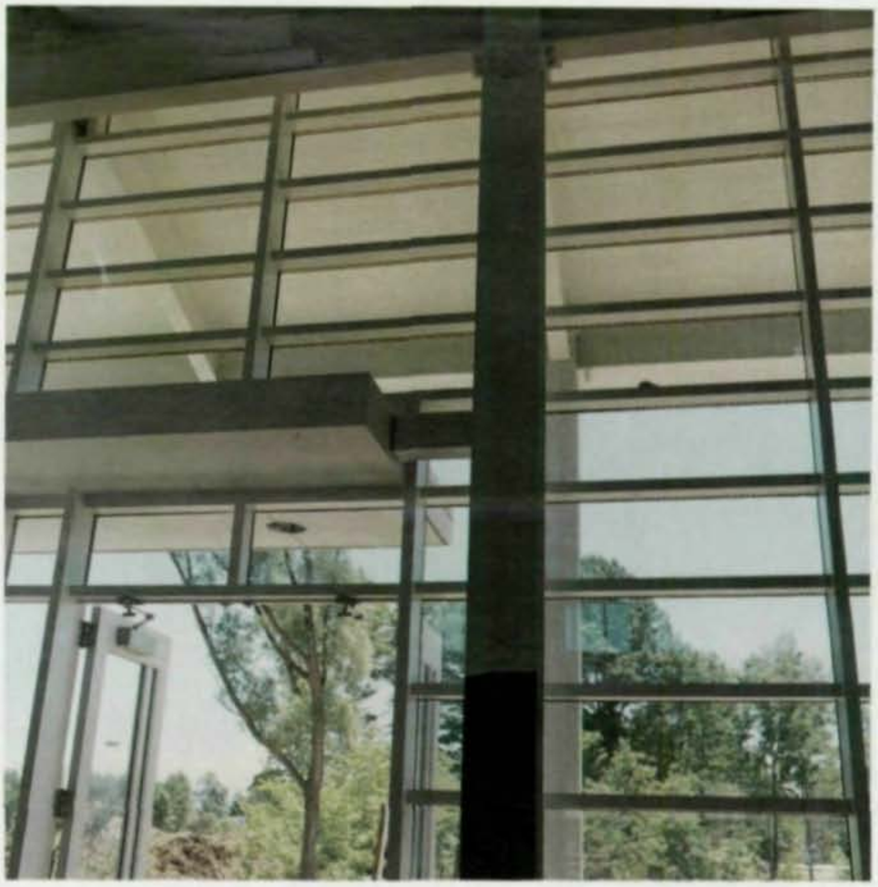
Architectural design called for custom-made, tapered columns to support the rods. The columns, 14" at the base and tapering to 7" at their top height, were created by cutting W14 shapes lengthwise and rewelding them to specifications.

Art/Library/Music Wing

Arc-shaped wings, resulting in a lack of parallel walls, are conceptually repeated in each of the three schools of the village, and house the art and music classrooms and libraries. The music rooms benefit from this design, because the non-parallel walls break sound waves, creating superior acoustics. Clerestory lighting throughout the wings provides dramatic natural lighting for the art classrooms and display areas.

The geometry of the wings is so complex both in plan and in elevation that columns were laid out using a coordinate system. Roof beams slope at different rates and in both directions, creating a warped roof surface as well as dramatic visual effects on both the interior and exterior of the building.

Pipe columns are utilized throughout this wing due to the lack of grid orientation and to simplify connections. The column diameter increases as the grade level increases.



Canopies located approximately 10' above the finished floor serve a dual purpose. In addition to providing shade, they visually interrupt the tall spaces. To maintain visual unity, the canopies were designed to reflect the appearance of the roof edges. Photo by William Sheck.

Theater

The tallest, most dramatic structure on the campus is the 850-seat theater, which was designed to incorporate all of the qualities of a professional facility. The theater wing is complete with audiovisual capabilities, covered orchestra pit, dressing rooms, theater classroom with production studio, scenery workshops and an elaborate catwalk system. Acoustic "clouds" create a false ceiling that deflects and disperses soundwaves, reducing echoes and improving sound quality.

The balcony floor is supported by 57"-deep plate girders, which span across the theater. The roof is supported on 80' trusses, with top and bottom chords composed of W14x61 members. Catwalks and rigging supports hang from the trusses and a 90' fly loft stores scenery panels, curtains and lighting, and speaker towers above the acoustic clouds.

Gymnasium

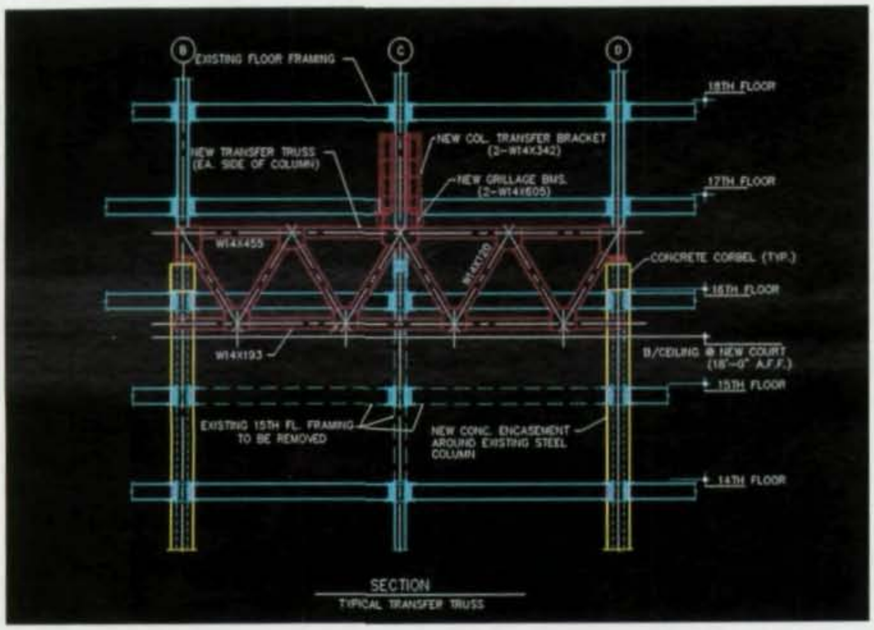
The roof of the 19,000-sq.-ft. gymnasium was constructed with two curved trusses, 32' and 128' long respectively, rising from 31' to 51' above grade. A 112'-long, 20'-deep truss was designed to span the gym and to support both the roof trusses and the third floor mechanical room. The 32'-long composite floor beams for the mechanical room floor were connected to the bottom chord of the truss to provide an open space for the second floor auxiliary gym.

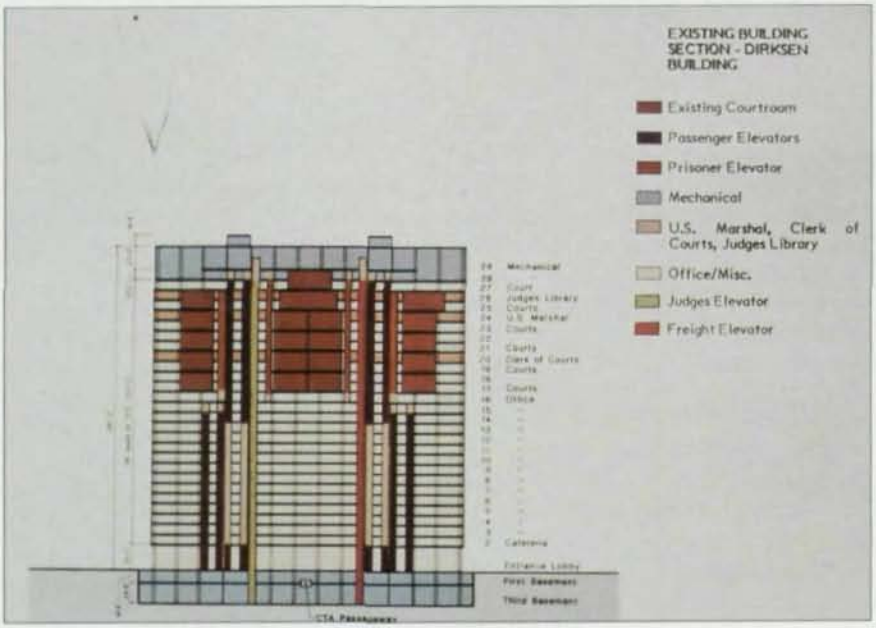
Phase 1 of the Perry Community Education Village cost $52 million and took two years to complete. The Albert M. Higley Co. served as construction manager. AISC-member Amthor Steel supplied 2,200 tons for the high school and AISC-member Kilroy Structural Steel Co. supplied 1,200 tons of steel for the physical education/community fitness center.

Dena R. Traina, P.E., is a structural engineer with Burgess & Niple, Ltd., Columbus, OH, and serves as section director for architectural projects. Robert S. Macholl, P.E., is the structural department head engineer for the company's Painesville, OH, office and is the project structural engineer on the Perry Community Education Village project.

Classic Expansion

A massive transfer truss was needed to create new courtroom space in the landmark Mies van der Rohe-designed Dirksen Federal Building





Even the best laid plans of Mies and men can go astray over a 30-year period. When the Dirksen Federal Building, a Chicago masterpiece designed by Ludwig Mies van der Rohe, opened in 1964 it included 14 courtrooms and an expansion capability, which was utilized in 1980, to accommodate six more. But by the 1990s, it wasn't enough.

The building is currently in the last phases of a $23 million renovation, which includes the addition of eight two-story high courtrooms. The problem facing the project's structural engineer, Chris P. Stefanos Associates, Oak Lawn, IL, was not just how to accommodate an 18'-high ceiling in a building with 12' floor-to-floor heights, but also how to create the large column-free spaces needed for a courtroom.

"Our solution was to remove four interior columns and then to support the floors above the courtroom on a large transfer truss," explained John Conroyd, S.E., Stafanos' project manager for the renovation. The new courtrooms are occupying space on what formerly were floors 12-14 of the 30-story building. During this phase of the renovation, only one level of courtrooms are being added. However, the renovation also will accommodate a future, second level of courtrooms. The full-story transfer truss was placed on the 14th floor, directly above the new courtroom space and picks up the weight of the upper 15 stories. If a second level of courtrooms is eventually required, the 12th floor can be built-out using steel plate girders to transfer loads since they'd only be carrying the weight of that one level, Conroyd noted.

The bottom chord of the 56'-long truss is a W14x193, while the top chord is a W14x426, and the web members are W14x120 and W14x132. Each of the trusses is 13' deep and weighs 38,000 lbs. All of the steel was A572 Grade 50 and steel fabricator was AISC-member Munster Steel Co.

Given the massiveness of the members, erection was a key challenge—especially since the structure remained occupied throughout the renovation. Individual truss members needed to be hoisted through 4' x 8' windows on the 15th and 16th floors and then reassembled in their final positions.

"We used a 250-ton crane to hoist the members up and onto a steel conveyor system," explained Joe Leslie, project manager with AISC Associate-member Broad Vogt and Conant, the project's erector. In addition to running alongside the column line inside the building, the conveyor was cantilevered 13' out the window. "We had to calculate the center of gravity of each piece so that we could know how far we had to start it into the window before we could release it from the crane." Once inside the building, the erection crew used chain falls and come-alongs to maneuver the pieces into final position.

The original design was all-welded, but that was switched to an all-bolted design to reduce construction time and minimize any fire hazard. Leslie pointed out that even if the truss was shop welded, it still would require a field splice. Using bolted connections also allowed pre-assembly to ensure correct fit-up.

During erection the trusses were preloaded with four 500-ton hydraulic jacks to minimize dead load settlement and strain gauges monitored the forces induced in the trusses and building members. Once the trusses were preloaded, the column transfer brackets were locked in their final position and the columns were cut and removed to provide the column-free, 40' x 55' courtrooms. The four trusses were brought into the buildings over the course of two weekends to

All of the steel was brought into the building during a weekend using a 250-ton crane. Members were hoisted up to the 15th floor and placed on a steel conveyor system, which extended 13' out of the window. Top photo by Boshke Photography.

Photo by Bosche Photography, Chicago

minimize disruption to building tenants and to avoid having to close a busy street during the business day.

Another crucial part of the project was reinforcing the columns supporting the transfer truss.

"We decided to use a composite column design to accommodate the added loads," Conroyd explained. On the first floor of the structure, the columns had been wrapped in concrete for fireproofing, while above that point the columns depended on drywall for fire protection.

Floor-by-floor, the drywall encasing the columns was removed. Headed shear studs were welded to both sides of the column flanges and webs and horizontal and vertical steel reinforcing bars were placed around the column core, which was then encased in concrete for a total width of 34". "The work required a lot of coordination with the tenants in those spaces," Conroyd said. "The contractor was only allowed to work on one or two floors at a time, and even then they were only given a small space in which to work. It was very labor intensive and we had to place concrete in wheelbarrows and transport it using the building's freight elevator." General contractor on the project was Power Contracting & Engineering Corp., Chicago.

On the first floor, core sample were taken of the concrete encasing

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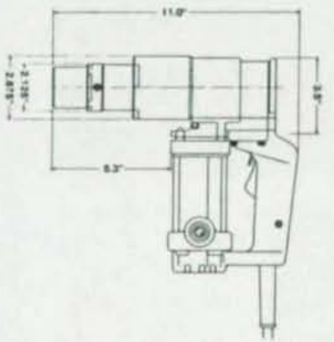

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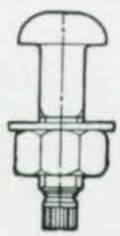

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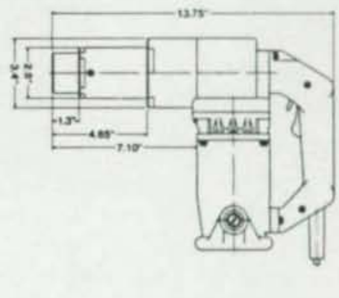

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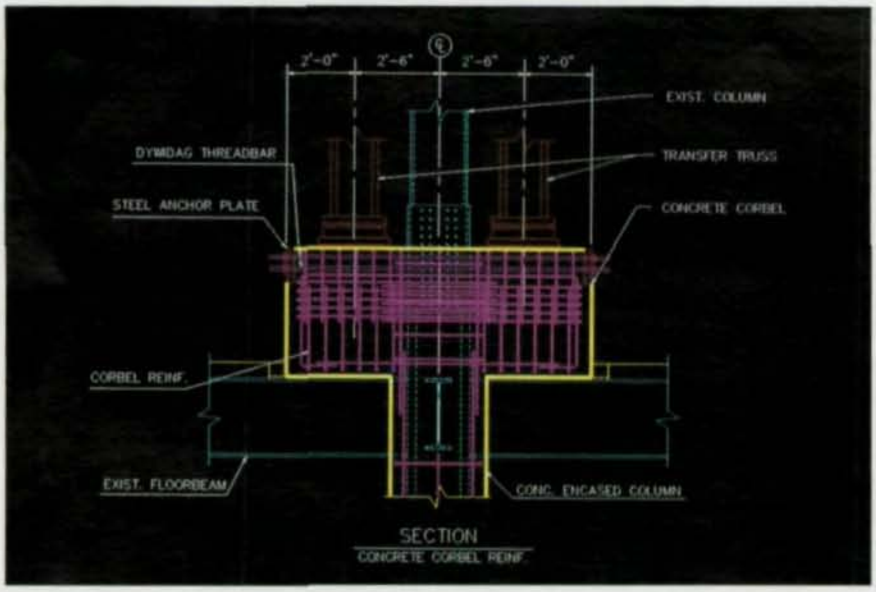

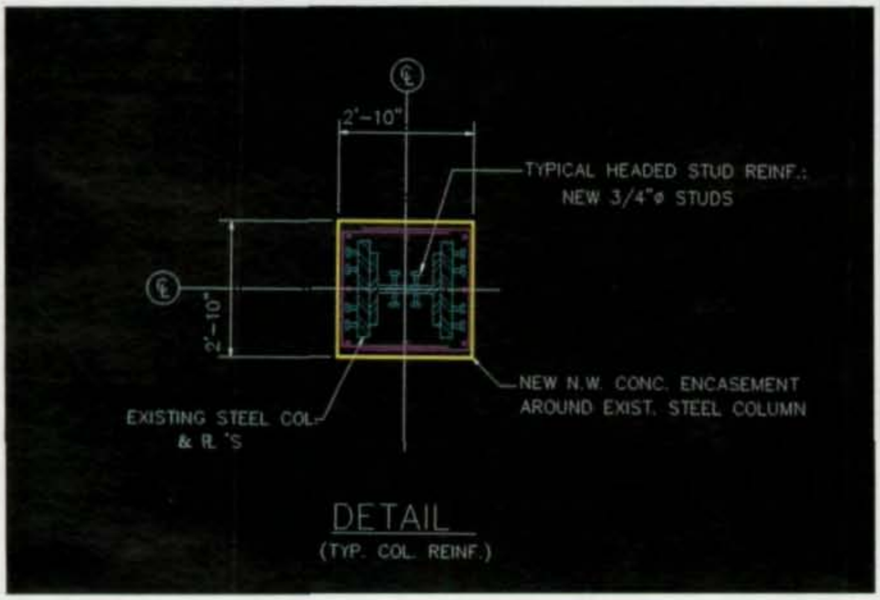

the columns to determine compressive strength. In general, it was adequate for the additional loads, though some of the concrete needed to be removed and replaced, Conroyd said.

The only other major modification to the building occurred on floors three through five, where a connecting stairway was installed for the U.S. Attorney's office, according to Nezin Hedlund, AIA, project designer with Lohan & Associates, Chicago, the project's architect. While installing the stair mostly involved cutting an opening through the existing concrete slab, designing it was much more complicated.

Because of the building's significance as a Modernist masterpiece, any changes must be made with great care. As a result, the stair was clearly designed in a Meisian style, and features an open stair of painted metal and polished stainless steel. As one reviewer wrote in a recent issue of *Inland Architect*: "It's easy to believe that, had the programmatic requirements been different in 1959, this stair might have been built in just such a way by Mies himself."

The structural work on the project is complete and the engineer recently won an award for it in the respected Structural Engineers Association of Illinois Awards Program.

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Turmoil constantly swirls around the topic of bolt specifications. To help answer questions about bolt buying, specifying and quality assurance, the Research Council on Structural Connections has issued a series of bulletins, the first four of which are printed here.

RESEARCH COUNCIL ON STRUCTURAL CONNECTIONS
Education Committee

Bulletin No. 1

Project Specifications For High-Strength Bolts

Trouble-free construction using high-strength bolts begins with the engineer's project specifications, which contain provisions to assure fasteners of the specified strength are supplied and that they are installed by procedures that assure the reliable performance in the completed structure. The following are suggested provisions to be incorporated in the project specifications:

Material Standards

The publications listed below form a part of this specification to the extent they are referenced. Publications are referred to in the text by basic specification designation only.

American Society for Testing and Materials

ASTM A325—*High Strength Bolts for Structural Joints.*

ASTM A490—*Heat-treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength.*

Research Council on Structural Connections

Specification for Structural Joints Using ASTM A325 or A490 Bolts. Endorsed by AISC.

Evidence Of Conformity

The supplier shall certify that bolts, nuts and washers furnished comply with all of the appropriate requirements of the applicable specifications, and shall provide complete manufacturer's mill test reports (Manufacturer's Inspection Certificate). For fasteners to be accepted, certification numbers shall appear on the product containers and correspond to the identification numbers on the mill test reports (Manufacturer's Inspection Certificate).

Manufacturer's symbol and grade markings shall appear on all bolts and nuts.

Field Erection And Shop Assembly

1. High-strength bolt installation and tightening shall be in accordance with Section 8 of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts.*

2. Regardless of installation requirements for the project, whenever high-strength bolts are to be installed, not less than three bolt, nut and washer assemblies from each lot supplied shall be tested in a tension measuring device at the job site at the beginning of bolting start-up to demonstrate that the bolts and nuts, when used together, can develop tension not less than that provided in Table 4 for the size and grade. The bolt tension shall be developed by tightening the nut. A representative of the manufacturer or supplier should be present, if required by the Engineer of Record, to assure that the fasteners are properly used, and to demonstrate that the fastener assemblies supplied satisfy the specification requirements.

Inspection

Inspection procedures should be in accordance with Section 9 of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts.* The inspector shall confirm that the materials meet the project specification and that they are properly cared for. He shall confirm that the faying surfaces have been properly prepared before the connections are assembled. He shall observe the specified job site testing and calibration and confirm that the procedure to be used does provide the required tension. He then monitors the work to assure the tested procedures are routinely followed on the joints that are specified to be fully tensioned.

Bulletin No. 2

Factors Meriting Special Attention By The Engineer

Incorporation by reference of standard specifications in project specifications makes for efficiency, completeness and uniformity of understanding and application from project-to-project. However, sight is often lost of the fact that, as standards, they cannot cover all eventualities, and as "boiler plate," their automatic incorporation in project specifications may lead to diminished recognition of the content and intent of the standard. Experience has shown that periodic review of standards and the Commentary thereto plus attention to the following items avoids misinterpretation of the Engineer's intentions and job site problems. The list is not all inclusive.

1. The current Specification for Structural Joints Using ASTM A325 or A490 Bolts is dated November 13, 1985. It is significantly different from earlier versions and is supported by a Commentary that was prepared

to provide background for the Specification and to cover the questions routinely asked. Proper design using high-strength bolts and their proper use in the field and shop can be assured by knowledgeable application and adherence to the requirements of the current specifications.

2. Routine adherence to demanding specifications assures the performance you contemplate in your design and saves money because corrective work is much more costly than work done correctly from the start.

3. Numerous past bolting problems were the result of fasteners that did not meet ASTM Specifications, but which fact became known only be testing after installation. Strict adherence to specified bolt and nut marking requirements and job site testing prior to installation of bolts are essential to preventing the installation of non-conforming or "counterfeit" bolts in your structures.

4. Some bolt tension indicating devices cannot be used to directly test short grip bolts; however, the required testing of short bolts can be accomplished in any convenient steel plate by the use of a washer-type direct tension indicating device. The direct tension indicating washer must first be tested using a longer bolt in the testing device to prove that they are not over nor under strength. Alternatively, a tightening torque may be determined in a tension measuring device using a longer bolt with a hardened washer under the turned element. This torque may then be used for testing shorter bolts with a hardened washer under the turned element in a steel plate provided lubrication and condition of threads for the long and short bolts are similar.

5. The Council Specification recognizes alternative-design fasteners with design features intended to control the installed bolt tension. The bolt tension control feature is dependent upon the manufacturer's quality control. Therefore, it must be confirmed by on-site testing before bolting start-up that the supplied alternative-design fasteners induce tensions as specified in Table 4 of the Council Specification.

6. Alternative-design fasteners may require different clearances than conventional bolts to accommodate the special tools. Check details.

7. The torques required to install A490 bolts more than 1" in diameter in slip-critical conditions is beyond the capacity of installation wrenches usually available. Do not use A490 bolts larger than 1" in diameter without careful consideration of the necessity and special attention to the problems of installation in order that your intended performance may be assured.

8. The mixing of A325 and A490 bolts of the same diameter should be avoided to assure that the A490 bolts are installed in their proper location.

9. Installation and/or testing of tension in bolts using standard tables or formulas for torque-to-tension relationships is highly unreliable and violates Specification requirements. If torque is to be used as an indicator of tension, it must be determined by specific test at the job site using project bolts.

Bulletin No. 3

Recommendations For Purchasing, Receiving And Storing A325 And A490 Bolts

The steel fabricator normally bears the cost of the high-strength fasteners on a project. "Purchase price" and "delivery" are the main criteria used in purchasing; however, purchase price is the true measure of cost to the fabricator only if the bolt manufacturers and vendors can be counted upon to fully comply with the purchase specifications. Unfortunately, with some manufacturers and vendors, knowledge and attention to specification requirements, proper manufacturing and testing procedures and quality control have been neglected or disregarded to meet tough competition.

Therefore, carefully prepared purchase orders that clearly and completely specify requirements for manufacture, testing, delivery and storing of high-strength bolts are essential to keep the fabricator's true costs under control while assuring proper fastener performance.

Purchasing Requirements

The purchase order for all ASTM A325 and A490 high-strength bolts must include the following:

1. The ASTM grade—A325 or A490

2. The type—Type 1, 2 or 3

3. A copy of the project specification for the manufacturer and vendor

4. "Ordering information" as required by ASTM, Volume 15.08 *Fasteners*, Pages 56 and 98.

Evidence of Conformity

Additionally, the purchase order should require:

1. The vendor to provide certification that the bolts,

nuts and washers furnished conform to all requirements of the referenced ASTM specification.

2. That certified manufacturer's mill test reports be supplied that clearly show the applicable ASTM mechanical and chemical requirements together with the actual test results for the supplied fasteners.

3. That the bolt heads and the nuts of the supplied fasteners must be marked with the manufacturer's identification mark, the strength grade and type as specified by ASTM specifications.

4. That, for projects requiring slip-critical connections, the lubricated bolt, nut and washer be preassembled to assure proper fit of the bolt and nut and the assembly tested for strength to meet the requirements of Table 4 of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts* prior to shipment to the purchaser.

Receipt And Verification

For acceptance, verify the delivered fasteners comply with purchase requirements upon receipt:

1. That bolts and nuts are marked as specified.

2. That manufacturer's mill test report reflects that the chemistry of the fasteners supplied comply with requirements for the type bolts and nuts specified.

3. That certification numbers appear on the product containers and correspond to the certification numbers on the mill test reports for the fasteners.

4. That mill test reports are supplied to both the purchaser and the testing laboratory responsible for quality control.

A representative of the fastener supplier should be present. The inspector shall be present and a tension measuring device shall be available in the shop and at the jobsite at the beginning of bolting start-up. Tests of representative samples of the fasteners received shall be conducted to confirm that the fastener assemblies, including lubrication if required, when tensioned by tightening the nut on the bolt, satisfy the installed tension requirements of Table 4.

Storage

All fastener components shall be stored in a manner that affords complete protection from moisture, heat and dirt contamination. These precautions are necessary to avoid corrosion, loss of effectiveness of the lubricant and dirt contamination that will increase the needed torque and preload scatter ranges. Each day, upon removal from storage, each bucket of fasteners will be visually inspected for corrosion, contamination with dirt and condition of lubricant. Any fastener found to be corroded, dirty or lacking the coating of lubricant present when delivered to the job site will be deemed unacceptable for installation.

Bulletin No. 4

Recommended Erection And Field Inspection Procedures For High-Strength Bolts In Structural Steel Assemblies

This bulletin provides brief guidance for proper installation of high-strength fastener assemblies, and emphasizes the most reliable methods for inspection of bolts installed by each of the four recognized installation methods. Additionally, it explains why the recommended receipt, storage, installation and inspection procedures are important to the reliable use of high-strength fasteners.

Threaded high-strength fasteners are an assembly made up of an externally threaded screw (commonly called a bolt) and a nut. If a washer is required, it is an essential part of the assembly; a direct tension indicator also may be specified. Each part is covered by separate ASTM Specifications that assure uniformity of strength and quality of each part, largely independent of the other elements.

The ASTM Specifications are adequate and appropriate for the strength and quality of the separate parts, but the ASTM tests may not be representative of the factors and conditions that determine the performance of the fastener assembly during installation and service as they are used in construction. For one example, the bolt part of the assembly is tested for strength by screwing the bolt into a standard testing fixture and subjecting it to *pure tension*. In usual applications, bolt tension is induced by torquing a nut on the bolt. During this operation, the bolt is subjected to *combined torque and tension*, which may cause the bolt to fail at a load less than the pure tension load. The highly variable torque component of the combined stress must not be so large that, for bolts required to be pretensioned, it prevents the development of the specified pretension.

The components of a fastener assembly come together and may be evaluated reliably as an assembly from the standpoint of the effect of fit of the nut threads with the bolt threads, the condition of the threads and the effectiveness of the lubrication and the function of the assembly in a structure only at the time of installation. The *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, and various specifications that invoke the RCSC Specifications provide the authoritative rules that ensure suitability and proper installation of fastener assemblies in structural applications.

Requirements Applicable To All High-Strength Bolt Installation

The assurance of service performance of fastener assemblies as contemplated in the design and intended by the Engineer must be the primary goal of all members of the construction team. This goal can be reliably achieved only by following the RCSC Specifications. The following summary emphasizes the essential procedures that are required but often neglected.

1. Job site quality control procedures for receipt of fasteners must verify that lot numbers on the kegs, boxes or bags correlate with the lot numbers on the test certificates and that the information on the certificates indicate that all lots of bolts, nuts, washers (if used), and DTIs (if used) are within the specification requirements. The procedures for receipt also shall verify that fastener assembly components are properly marked.

2. Special attention must be given to galvanized fastener assemblies to assure compliance with latest ASTM Specification revisions, such as rotational capacity tests and lubrication of galvanized nuts.

3. Reaming of poorly aligned holes, which may be required to avoid damage to threads by driving bolts, should be accomplished in accordance with AISC *Code of Standard Practice*, paragraph 7.12. Modifications to clear major misalignments should be recorded in detail with date, time and location.

4. When the requirements for receipt of fasteners and the requirements for verification of installation procedure is followed by installation of fasteners in the work according to the pretested procedure under the surveillance of the inspector as required in the *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, the installed fasteners are deemed to be properly installed and to satisfy the requirements of the specification without further inspection or testing.

Installation And Inspection Of Bolts Not Requiring Pretension

1. Bolts in connections not identified as being slip critical *nor* subject to direct tension *nor* otherwise identified as requiring pretension should be tightened to pull all plies into firm contact but need not be tightened to induce a specific pretension.

2. Inspection should not include testing for bolt tension in connections not requiring pretension.

3. Painting of the faying surfaces of these connections is permitted.

Requirements Applicable To All Methods For Bolts Requiring Pretension

The following requirements are mandatory for all RCSC Specification approved field installation and inspection procedures for connections requiring pretension, such as slip-critical connections and fasteners subject to directly applied tension and connections covered by AISC Specification paragraph J1.11.

1. All fastener components shall be stored at all times prior to installation in a manner that affords protection from moisture, excessive heat and contamination by dirt.

2. Only the number of fasteners required for one shift of work shall be removed from storage at a time. At the end of the work day, all fasteners not installed are to be returned to suitable storage.

3. Tension measuring devices are necessary erection tools that are required by the erector at the job site where fasteners requiring pretension are being installed. The erector shall assemble the as-received fastener components and test the assemblies, as they will be used, to confirm that the installation procedure to be used for tensioned fasteners develops tension 5% higher than required tension given in Table 4 of the RCSC Specification.

4. During the project, fasteners found to be corroded, dirty or lacking adequate lubricant to satisfy test requirements shall be collected in a designated container, cleaned and re-lubricated with an approved lubricant and re-tested. Fastener assemblies that cannot demonstrate an installed tension 5% higher than minimum required tension shall be deemed unacceptable for use.

5. Bolts shall be installed in holes of a connection and brought to the snug-tight condition. Snug tightening shall begin at the most rigid part of the connection and progress to the free edges.

6. After the snug tightening operation, all bolts shall be tightened further in a systematic manner beginning with the most rigid part of the connection and progressing to the free edges of the connection.

7. Surveillance by the inspector of start-up testing and performance according to the tested procedure on work in progress provides greatest assurance of proper installation and bolt tension. After-the-fact testing is inherently less reliable.

8. Tensions in excess of those given in Table 4 are not cause for rejection.

Additional Requirements For Turn-Of-Nut Tightening Method

1. At the start of the work, the installation crews shall demonstrate that their routine procedure for turn-of-nut method properly establishes the snug tight condition and turns required for full tension.

2. Bolt tightening to final tension may be accomplished with greatest assurance by match-marking the outer face of the nut with the protruding bolt point after the snug tightening operation.

3. The specified turns from snug tight to final tension must be applied in a systematic manner beginning at the most rigid part of the connection.

4. The inspector shall observe the testing of the representative samples and verification demonstrations of the method to be used, and shall exercise surveillance to assure that the proven method is routinely followed in the work.

Additional Requirements For Tension Control Bolts

1. *The sheared-off splined end of an individual bolt* only indicates that, at the time the splined end was torqued off, enough torque had been applied to the bolt to fracture the break neck. Specified tension is assured for all bolts in a connection required to be tensioned only if the bolts have been systematically snugged and tightened as specified.

2. Tightening to final tension and shearing the break neck shall not be accomplished in a single continuous operation, especially in large joints.

3. The greatest assurance of properly installed and tensioned bolts will result if the inspector exercises surveillance inspection of work in progress to assure that procedures (5) and (6) of the tensioning requirements applicable to all methods are routinely followed before the break neck is sheared.

4. Specific and proper lubrication of "tension control bolts" is essential to the reliable use of these fasteners. Contact the manufacturer prior to applying any lubricant to these fasteners.

Additional Requirements For Calibrated Wrench Tightening

1. At least once *each working day*, using the tension measuring device and representative samples of the fasteners to be installed that day, the erector will calibrate the wrenches and demonstrate that the installation procedure and the equipment being used achieves 1.05 times the specified installation tension.

2. Any time a component of the installation process is changed (operator, impact wrench, compressor, fastener component combination, etc.) this test is to performed to revalidate the procedure.

3. In the work, the bolts shall be installed with hardened washers under the element to be turned in tightening.

4. Following the initial tightening, the connection shall be tightened using the calibrated wrench beginning with the most rigid part of the connection and progressing systematically to the free edges until the tightening torque for all bolts reach at least the torque established by the demonstration tests for the day. Several cycles may be needed.

5. The inspector shall observe the calibration of wrenches and/or required installation torques, as appropriate, for the fastener assemblies being used, and shall use the torques so determined to check the tightening of bolts in the work.

6. So-called "standard" torques, or torques determined by formula or tables, shall not be used.

Additional Requirements For Assemblies With Direct Tension Indicators

1. At the start of work, in addition to General Requirement (1), representative samples shall be checked in the tension calibrating device to demonstrate that the DTIs supplied are within the installed tolerances in Table 3 of ASTM F959.

2. The presence of a DTI with the protrusions compressed to the specified gap merely indicates that at some time an adequate load has been applied. Specified tension is assured only if the connection is systematically snugged and tightened as required for all methods for tensioning bolts.

3. Tightening of individual bolts with DTIs to final gap shall not be accomplished in a single continuous operation. First, all fasteners shall be tightened so as to compress the DTI protrusions to two times the specified final gap.

4. Tightening to final specified indicator gap shall be accomplished by systematic tightening of fasteners beginning at the most rigid part of the connection and progressing to the free edges, especially in large connections. Several cycles may be required.

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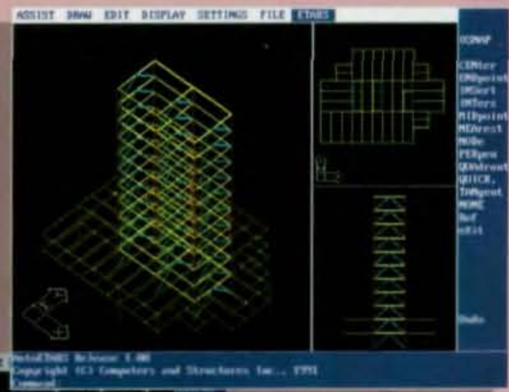
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