MODERN STEEL CONSTRUCTION

August 1995

$3.00

Steel

UNITED STEEL DECK, INC.

deck design data sheet #23

ASTM specification change

The American Society for Testing and Materials (ASTM) recently issued new material specification numbers for galvanized products. The new specification is A653-94*, *Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process. There are two important changes:*

(1) The new material designation uses the grade to define the yield point.

(2) The coating designation (G30, G60, G90) is now part of the A653 specification.

The coating designation was previously part of A525 but A525 has now been replaced with A924-94 which is a general specification covering all metal coatings (aluminum, zinc, lead-tin) applied by the hot-dip process.

Old Grade Name Under A446
grade A
grade C
grade E

⇩

New Grade Name Under A653
Structural Quality (SQ)
grade 33(230)
grade 40(275)
grade 80(550)

The numbers after the grade (33, 40, 80) are the minimum yield strengths in ksi. The bracketed numbers are metric (MegaPascals) and will be used instead of the ksi values in metric specifications.

Example: ASTM A653 SQ grade 33 with coating designation G60 replaces ASTM A446 grade A with coating designation G60.

The ASTM number for uncoated steel products (and for products painted over uncoated steel such as roof deck painted in our factory) is still A611 and the grades C, D, and E are still used.

The SDI thickness tolerance, our load tables, and our section properties have not changed. We will change the ASTM numbers in our catalogs as they are printed. The new ASTM numbers will start to appear in job specifications, but it will probably be a number of years before the old ones are completely gone. We will simply have to endure the confusion during the transition period.

* The last two digits (94) indicate the adoption year for the specification.

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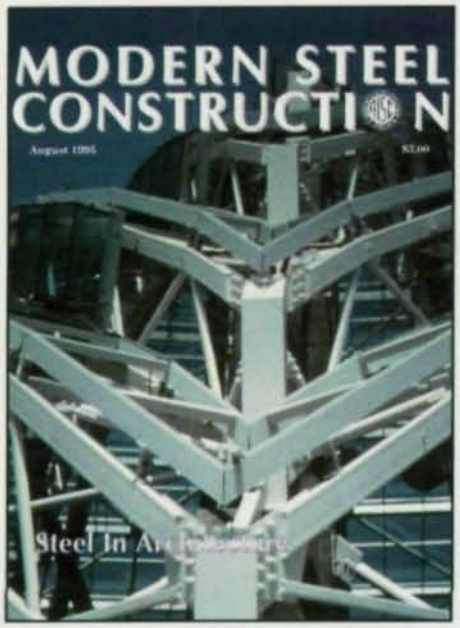
Volume 35, Number 8 **August 1995**



The brightly colored 152,000-sq.-ft. Florida Aquarium is more than just the latest in a frenzy of aquarium construction across America. The story behind this state-of-the-art, awe-inspiring facility that is one of the nation's largest and most exciting aquatic facilities begins on page 18.

FEATURES

DEPARTMENTS

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Curious Origins

WHEN MY WIFE AND I WERE JUST STARTING TO DATE, she lived in Toronto. Back in the early days of our relationship, we visited exclusively by flying back and forth. But one weekend, I decided to surprise Judy with a visit. By the time I had made my decision it was much too late to get a reasonable air fare, so instead I drove the 530 miles to Toronto. Unfortunately, being a little more than somewhat absent-minded, I neglected to bring a map. Fortunately, finding the city itself was pretty simple (just take I94 to Detroit and the 401 to Toronto). It was finding her house that was a bit more challenging.

I exited from the highway where I thought it was appropriate, but I wasn't quite sure I was in the right place—until I saw a tower crane looming over a recently begun apartment complex. I made a quick right, a quick left, another left, a right turn and I was there.

During the next year or two, I often made that drive, and I always used that crane as my final landmark. I watched as the building grew and grew, and finally, on one September trip, I noticed a Christmas tree atop the highest girder. I remember thinking that the tower crane—my perfect landmark—would soon be coming down, but at the time, I didn't give the tree much thought. But a recent conversation with Bob Lorenz, AISC's Director of Education and Training, brought the image back. Bob was recently asked how and where the traditional topping-out ritual (either putting a tree or a flag atop the final girder) originated. No one at AISC could answer the question, and we couldn't find any references to it in any of our library books.

So if anyone out there knows the origins of this curious custom, let me know. Either drop a note to me at *Modern Steel Construction*, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001, or fax me a note at 312/670-5403. **SM**

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

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

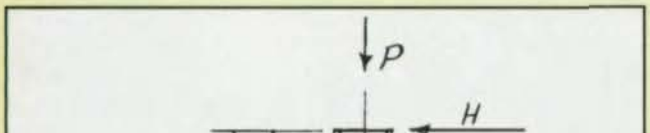
Answers and/or questions should be typewritten and double-spaced. Submittals that have been prepared by word-processing are appreciated on computer diskette (either as a Wordperfect file or in ASCII format).

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

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

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

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



Since the r's (radius of gyration) are known, the key to the calculation of F_a is to find the K in the allowable stress equation for axially loaded compression members.

Per AISC Design Guide, "Industrial Buildings, Roofs to Column Anchorage" Appendix B: Calculation of Effective Lengths of Stepped Columns, the values of K_1 and K_2, representing the effective length factors for upper and lower segments of column, respectively, can be attained by solving the equations proposed by J.P. Anderson and J.H. Wodward (AISC Engineering Journal, October 1972, pp. 157 - 166) or by entering Table 1 (included in Appendix B) with the values of I_1/I_2, L_2/L_T, and P_2/P_T under the appropriate end condition category.

In this case, by using Table 1, L_2 = 10', L_T = 30', P_2 = 0.0, P_T = p, then L_2/L_T = 0.33, P_2/P_T = 0.0 and the end condition is "Fix-Free"; therefore, by entering the value of I_1/I_2 (the ratios of moments of inertia of upper and lower segments, respectively) the values of K_1 and K_2 can be determined by interpolation.

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

Another answer:

The L/r ratios do not work well for this type of column. Instead, calculate the theoretical Euler buckling load, and then check both the elastic and inelastic allowable buckling loads.

There are several procedures to calculate the Euler buckling load. Simple hand calculation methods include Finite Differences, integration, or Newmarks Method. Example calculations for the first two methods are available in " Principles of Structural Stability Theory" by Alexander Chajes, and the last in "Theoretical and Applied Mechanics" edited by N.M. Newmark.

After finding the theoretical Euler buckling load (P_{cr}), determine whether elastic or inelastic buckling controls. If the Euler buckling load (P_{cr}) is less than $P_y/2$ (where $P_y = AF_y$) then elastic (Euler) buckling controls. This is the basis for the AISC ASD equation for C_c, see the AISC ASD Commentary. If elastic buckling controls, simply multiply the theoretical buckling load by $^{12}/_{23}$ and you have the allowable axial load.

If inelastic buckling controls (i.e. $P_{cr}>P_y/2$) it becomes a little more complicated. Lets dissect AISC ASD Equation E2-1. The denominator is simply a sliding factor of safety that results in $^{23}/_{12}$ where $Kl/r = C_c$, and results in $^{5}/_{3}$ when $Kl/r = 0$. For the moment, call the denominator the safety factor, SF. Now we can rewrite equation E2-1 as:

$F_a = [1 - ((Kl/r)^2/(2C_c^2))]F_y/SF$

This equation can be rewritten in terms of force s follows:

$$P_a = [(1 - AF_y)/4P_{cr}]AF_y/SF$$

$$\text{since}\, P_{cr} = \pi^2 E/(Kl/r)^2$$

$$\text{and}\, C_c = \sqrt{2\pi^2 E/F_y}$$

Now, having calculated P_{cr} from one of the methods noted above, and knowing all the other terms, you have only to determine the appropriate safety factor, SF. The term $(Kl/r)/C_c$ can be rewritten in terms of force if one remembers that:

$$C_c = \sqrt{2\pi^2 E/F_y}$$

Substitute this into the equation for $(Kl/r)/C_c$ and you will finally get:

$$(Kl/r)C_c = \sqrt{[(F_y A)/2P_{cr}]}$$

Substitute this into the denominator in place of $(Kl/r)/C_c$ and you have an equation rewritten in terms of force. Use the smaller section to determine steel area, A, since inelastic buckling will occur there first.

Remember that K is already included in P_{cr}, since you determined the buckling load P_{cr} directly with its actual boundary conditions.

Duane L. Siegfried, P.E.,S.E.
Ralph Hahn and Associates, Inc.
Springfield, IL

Another response:

The most common approach is to use the formula $P_{cr} = K_1 \pi^2 E_1 I_1 / L_2$, where E_1/I_1 is the stiffness of the smaller section, L the total length of the column, and K_1 depends not only on end constraints, but also on the ratio of stiffness and lengths for the two sections.

Values of K1 are tabulated for different end constraints and ratios of stiffness and lengths in "Formulas for Stress and Strain", by Raymond J. Roark and Warren C. Young, McGraw-Hill, New York (various editions).

For the case in question (cantilever column), K_1 varies between and 0.5, with the larger section up to twice as stiff as the smaller one.

Behrouz Jazayery, P.E.
Mueser Rutledge Consulting Engineers
New York, NY

What is the flatness tolerance for webs of welded plate girders?

Since for statically loaded (buildings) structures,, web flatness does not affect the structural integrity of a girder, neither the LRFD Specification not the AISC Code of Standard Practice provides a limitation on the maximum out-of-flatness of girder webs. AWS D1.1 Section 8.13.2 does, however, provide such requirements for welded plate girders. Problems arise, however, when these tolerances are applied to girders with thin webs. Specifically, in girder webs less than $^{5}/_{16}$-in. Thick, they do not account for operational difficulties caused by shrinkage resulting from web-to-flange welds and/or welds that attach stiffeners to the web. Because of this, in some cases, flatness within AWS tolerances cannot be practically provided.

AISC recommends that, for statically loaded (building) structures, the dimensional tolerance for deviation from flatness of a girder web less than $^{5}/_{16}$-in. Thick, without stiffeners or with stiffeners on one or both sides, be determined by the larger of ½-in or AWS Section 8.13.2. If architectural considerations require special flatness tolerances, such special requirements must be identified on the engineering drawings and stipulated in the bid documents.

American Institute of Steel Construction
Chicago, IL

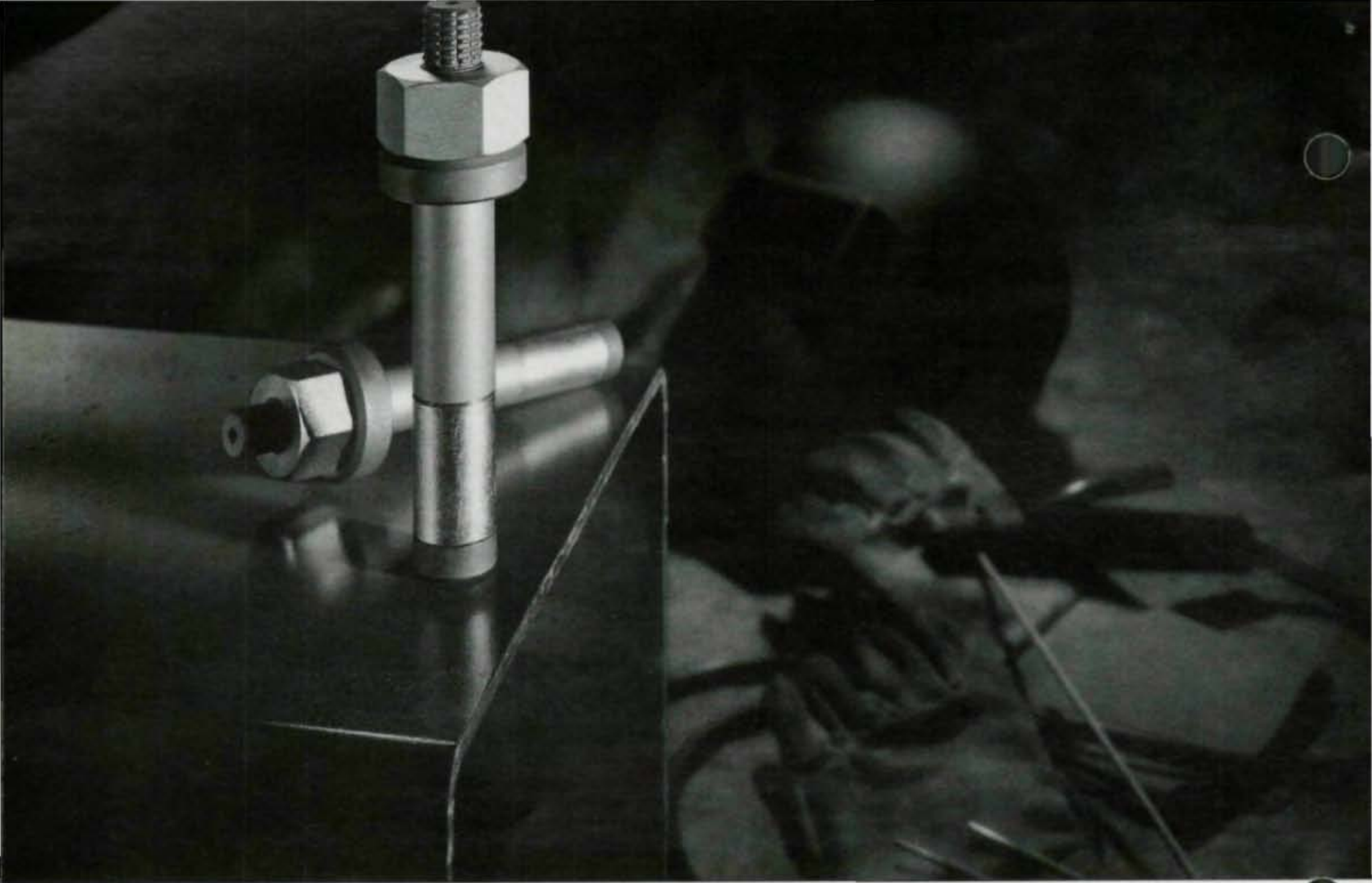
New Questions

Are there special requirements for the design of High-strength A325 or A490 bolts that are going to be in a high temperature area?

Alice Leich
S.E.S Environmental, Inc.
Knoxville, TN

If a W-shaped column is made up of three welded plates, how does one design the welds connecting the plates together?

Correction: *The answer by James McCarthy in the June issue referred to the wrong question. The correct question is: In a partially cover-plated column, how would you analyze the column for governing l/r ratio? We regret the error.*



Special Report:

Wide Flange Shipping Times Expected To Improve

As of mid-July, lead times for shipping wide flange sections by mills to structural steel fabricators were approximately twice as long as historical levels, but all of the major mills state that by the end of the year lead times will return to normal. And already, there have been some indications that shipping times are beginning to improve.

Longer lead times this summer were the result of a confluence of factors. Foremost was a surge in demand that hit in late March. But adding to this problem was the scheduling by some mills of summer downtime for annual maintenance and for capital improvements. For example, Chaparral Steel Company shut its mill for two weeks beginning June 19 for maintenance and upgrading. "We're a stocking mill, but our inventories were severely depleted during our shutdown. However, we don't have another scheduled shutdown for 52 weeks," explained Jeff Werner, senior vice president of sales at Chaparral. Another factor has been increased purchases by speculators who are concerned about potential shortages, especially as Bethlehem Steel Corporation begins phasing out production of larger wide flange shapes, including W40, W36, W33, W30, and W27 shapes plus selected W24, W21, W18, W16, W14, and W12 shapes.

All of the mills are taking independent steps to reduce shipping times. At Chaparral, they are closing their rolling orders at 70% of capacity in order to use the remaining 30% to build inventory. However, Werner pointed out, as of mid-July most of the August and September rollings were not yet at 70% of capacity. In addition, both the large and medium size mills at Chaparral have added additional shifts—and as a result of the recent capital improvements productivity on these mills has increased. This capacity increase is substantial enough that Chaparral has begun purchasing billets and blooms rather than depending solely on the hot metal produced by its own furnaces. "We anticipate increasing productivity 8% during the next 12 months for structural products," Werner stated. Chaparral also is shifting products between its mills to additionally increase productivity. For example, all of the W12 and W14 shapes, which previously were rolled on the medium mill, are being shifted to the larger mill. "This will free up rolling tonnage for beams in the 10-in. and down range and will increase availability of lightweight beams."

> **We anticipate increasing productivity by 8% during the next 12 months—Jeff Werner, Chaparral Steel**

Nucor-Yamato Steel Company is taking similar steps, including shifting production. For example, HP12s, the most popular piling size, is being shifted from Nucor-Yamato's large size mill to a smaller-sized mill. "In essence, what we have done is add enough extra capacity to replace the loss of Bethlehem's capacity in the sections we make in common," explained Bob Johns, sales manager at Nucor-Yamato. "We've opened a tremendous quantity of mill time to replace the void left by Bethlehem in closing their large-section mill." Nucor-Yamato also is taking steps to reduce speculation. "We're trying to take the speculative tonnage out of the backlog. If we find companies are blocking tons based on bids rather than sold projects, we won't accept additional orders from that company without proof of a contract."

As of mid-July, Nucor-Yamato

was booked through September. Additionally, the mill will shut down for semi-annual maintenance for one week at the end of September. "After that, though, we expect the cycle to tighten and we expect to return to historic lead times by the end of the fourth quarter," Johns said. Part of the reduction in lead times will be accomplished through increases in capacity. "We've added a second crew to our big mill and a second strand to the caster. As a result, we're rolling more raw tons."

Crew additions also are in the works at Northwestern Steel and Wire Company. "We are currently hiring and training a fourth crew for our Houston mill," said Ray Bauer, Manager/ General Sales, Steel Division at Northwestern. "Also, this fall we will install our second ladle metallurgy furnace and late this year we will begin the conversion to ultra-high-power transformers at our melting facility in Sterling. These improvements will further enhance our hot-metal and semi-finished steel capabilities to feed our rolling mills."

As of mid-October, Bethlehem will permanently close its largest mill. In addition, it will be closing its combination mill, which produces structural shapes up to 24 in., sometime during the fourth quarter for maintenance and modernization. However, Bethlehem reports that the combination mill will be back in operation in December and that availability will improve at that time.

Jumbo shape availability was adversely affected earlier this year by the temporary closing for modernization of TradeARBED's heavy beam mill. "ARBED has now completed the modernization of their Differdange heavy beam mill that adds a continuous caster and electric arc furnace," said Greg DePhillis, product manager with TradeARBED. "Production of medium sections will be via continuous casting while heavy Tailor-Made Beams (WTMS) and Jumbos will be ingot cast. In view of the closing of Bethlehem's heavy beam mill, ARBED has geared up to give Jumbo columns (W14x426 through 730) a top priority in terms of rolling and delivery." August and September rollings remain open for orders as of mid-July. For medium shapes (14-in. through 36-in. to about 400 lbs.), ARBED currently has limited availability due to the heavy orderbook at the mill, caused partly by downtime from the modernization and also from heavy worldwide steel demand. "However, a special priority will continue to be given to HISTAR Grade 65 (ASTM A913/65) in all sizes. We hope to return to more normal offerings of medium shapes during the fourth quarter," DePhillis stressed.

IN ESSENCE, WHAT WE HAVE DONE IS ADD ENOUGH EXTRA CAPACITY TO REPLACE THE LOSS OF BETHLEHEM'S CAPACITY IN THE SECTIONS WE MAKE IN COMMON—BOB JOHNS, NUCOR-YAMATO

Foreign purchases of steel from U.S. mills, however, has not affected lead times, according to Johns. "Exports, as a percentage, haven't changed one iota at Nucor-Yamato. But we have seen some U.S. fabricators increasing their amount of international work." Werner concurred that foreign purchases are not a big factor in U.S. shipments, though exports are up very slightly at Chaparral.

Despite the increase in mill orders, Werner stressed that industry-wide demand is actually less than it was during 1989. "The U.S. steel industry shipped 3.6 million tons of wide flange in 1989 and 1990, but the demand today is 20% less, about 3 million tons at our current shipping levels." And even if demand continues to increase, Werner estimates that current existing capacity is 3.4 to 3.5 million tons per year.

Producers indicated that increased demand has not fueled a sharp run-up in steel prices. The increases that did occur earlier this year merely brought steel prices back to approximately 1989 levels, or even lower, depending on member size. These relatively low prices occurred despite substantial material costs increases, such as an increase in scrap metal prices of $40 per ton since 1989. Any additional increases this year, according to Werner, will likely only be the result of material cost increases.

STEEL SEMINARS CONTINUE

FROM DISCUSSIONS OF BOLT INSTALLATION TO THE DEVELOPMENT OF A NEW HIGH-STRENGTH STEEL, AISC's 1995 Seminar Series is designed to provide practical information for structural engineers, fabricators, and others involved in the steel construction industry. As an added bonus, the seminar will offer a session on "Answers to the Most Commonly Asked Questions" of AISC staff engineers.

Each seminar begins at 2:00 p.m. and ends at 8:15 p.m. Cost for the seminar, which has a CEU value of 0.45, is $120 ($90 for AISC members). The fee includes the lectures, numerous handouts, LRFD educational software, and dinner.

For more information, call AISC at 312/670-5422.

1995 STEEL SEMINAR SCHEDULE

August 8 ....Rochester, NY
August 9 ....Albany
August 10 ....Portland, ME
August 15 ....Washington, DC
August 17 ....Philadelphia
August 22 ....Chicago
August 24 ....Milwaukee

September 7 ....New York City
September 12 .....Meriden
September 14 .....Boston
September 19 .....Dallas
September 21 .....Houston
September 26 .....Denver
September 28 .....Kansas City

October 3 ....Birmingham, AL
October 5 ....Atlanta
October 10 ....Detroit
October 12 ....Indianapolis
October 17 ....Cleveland
October 18 ....Columbus
October 19 ....Cincinnati
October 24 ....Memphis
October 26 ....Nashville
October 31 ....Pittsburgh

November 2 ....Edison
November 7 ....New Orleans
November 9 ....Albuquerque
November 28 .....Miami
November 30 .....Orlando

For more information, call 312/670-5422 or FAX 312/670-5403

T.R. HIGGINS LECTURE SCHEDULE

CONNECTION DESIGN IS A COMPLEX SUBJECT FOR BOTH ENGINEERS AND FABRICATORS. This year's T.R. Higgins Lectureship Award was presented to William A. Thornton, Ph.D., P.E., for his work on safe and economical connection design.

In his lecture, Thornton, chief engineer of Cives Steel Company and President of Cives Engineering Corporation, both of Roswell, GA, discusses bracing connections, shear connections and moment connections. The presentation features descriptions of both "good" and "bad" connections, as well as presents "rules of thumb" for designing connections.

The lecture has a C.E.U. value of 0.1.

Six have so far been scheduled for 1995:

- **Chicago—Sept. 27** (call 708/527-0770)
- **Milwaukee—Sept. 28** (call 414/251-5110)
- **Denver—Oct. 17** (call Arla Zimmerman at 303/294-0180)
- **Kansas City—Oct. 18** (call Gordon Finch at 816/373-1203)
- **St. Louis—Oct. 19** (call 314/638-5000)
- **Albany—Oct. 27** (call Anne Sylvester at 518/785-3221)
- **Atlanta—Nov. 21** (call 404/642-9707)

Additional lectures will be held in New England.

INTERNATIONAL BRIDGE CONFERENCE SCHEDULED FOR AUGUST

DESPITE INDICATIONS OF INCREASED INFRASTRUCTURE INVESTMENT, the number of proposed projects still exceeds the available funds. The Fourth International Bridge Engineering Conference, to be held in San Francisco August 27-30, will include sessions on both project management and engineering.

Conference sessions inlcude: bridge management systems (condition rating, PONTIS, BRIDGIT, and safety assurance); bridge aesthetics; bridge performance; bridge construction; long-span bridges; bridge loads and dynamics; FRP composites and other materials for bridges; bridge rehabilitation; seismic response of bridges; bridge bearings, joints and details; prestressed concrete bridges; bridge structural systems; bridge substructures (scours and ship impact); and wood bridges. In addition, a special session will be devoted to bridge performance during the Kobe earthquake in Japan.

Scheduled speakers include engineers in private practice, academics from the U.S. and several foreign countries, domestic and international government officials, and state bridge engineers.

Registration is $350. For more information, contact: Reggie Gillum, Transportation Research Board, Box 289, Washington, DC, 20055; 202/334-2382; fax: 202/334-2003.

New Manual Aids Renovation of Steel Joists

THE 318-PAGE STEEL JOIST INSTITUTE 60-YEAR JOIST MANUAL CONTAINS A CHRONOLOGICAL COMPILATION OF ALL SPECIFICATIONS AND LOAD TABLES of SJI steel joists manufactured between 1928 and 1988. This new Manual is designed to answer questions about the load-carrying capabilities of steel joists in existing buildings requiring renovation.

In addition to pertinent data on joists, the $59 Manual includes information on various building documents required for renovation, investigative procedures and a complete listing of commonly used live and dead loads for the past 60 years.

For more information, contact: Steel Joist Institute, Suite A, 1205 48th Ave. N., Myrtle Beach, South Carolina, 29577; 803/449-0487.

New Publications Catalog Available

The American Institute of Steel Construction has published its 1995/1996 Publications list. This document contains a short synopsis of books, design guides, specifications, software and periodicals available from AISC, along with ordering information.

The catalog also lists a new ordering address for AISC publications: AISC, Dept. 77-5245, Chicago, IL 60678-5245. AISC's main address remains unchanged.

To receive a free copy of the publications list, call 800/644-2400.

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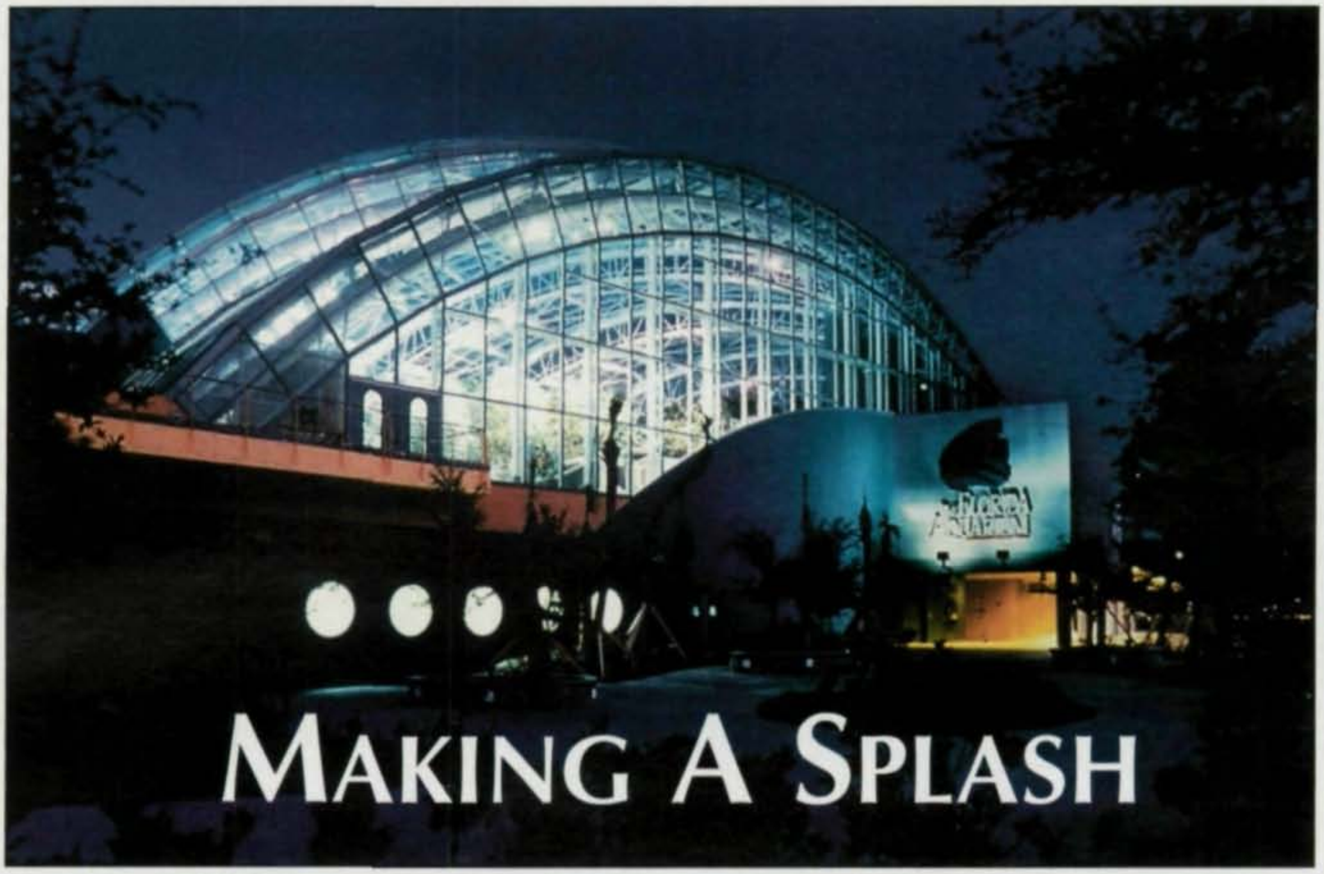


Making A Splash

Large steel truss arches span across the new Florida Aquarium to help create a light, airy feel

By David W. Landis, P.E., and Blair K. Hanuschak, E.I.

WHEN THE $84 MILLION FLORIDA AQUARIUM OPENED ITS DOORS IN TAMPA on March 31, 1995, it made quite a splash. The brightly colored 152,000 square foot aquarium is more than just the latest in a frenzy of aquarium construction across America. The Florida Aquarium is a state of the art, awe-inspiring facility that is one of the nation's largest and most exciting aquatic facilities.

The aquarium's signature glass-covered, shell-shaped dome roof spans 140 feet over 20,000 square feet of lush exhibit space representing various types of Florida wetlands habitat. Striking architecture, engineering prowess, and cutting edge aquatic technology came together in spectacular fashion to create southwest Florida's newest landmark and tourist destination.

The aquarium commands a prominent site along the waterfronts of Ybor and Garrison Channels, just southeast of downtown Tampa. As part of the Garrison Seaport Center, which also includes a new cruise ship terminal, the Florida Aquarium is within walking distance of the Tampa Convention Center, the Harbour Island master planned residential, retail, and hotel community, and the new NHL arena currently being built. Planned development for this growing area includes a high-rise hotel, an outdoor music venue, restaurants, and other entertainment and retail amenities. Construction of the aquarium has been a key factor in the revitalization of this area of town.

The Florida Aquarium's mission is to tell the Florida water story, from the inland stream and swamp to the coast and open sea, in an educational and entertaining way. Exhibits are arranged to follow the journey of a drop of water from Florida's inland freshwater springs and aquifers, through marshes, swamps, and wetlands, out to bays and beaches, through coral reefs, and finally, out to the open ocean. The exhibits are organized into four main areas: Florida Wetlands, Florida Bays and Beaches, Florida Coral Reefs, and Florida Offshore. The Florida Aquarium is unique in that it focuses specifically on the rich diversity of plant, animal, fish, and marine habitats found in Florida.

4,300 Specimens

The aquarium is home to more than 4,300 specimens of 550 fish, animal, and plant species native to Florida. The Florida Wetlands takes the visitor through springs, streams, saw grass marshes, rivers, hammocks, cypress swamps, and mangrove forests. The Florida Bays and Beaches exhibits allow the visitor to examine the marine habitats of Florida bays, sea grass beds, bay bottoms, bridges, and beaches. The Florida Coral Reefs simulates a 60-foot dive off the Florida Keys, beginning at the shallows of a near-shore reef, passing through an underwater tunnel, and ending at an enormous viewing window into the huge 500,000 gallon coral reef tank, alive with a myriad of brightly colored coral, fish, sharks, and rays.

The building's unique architecture, and the engineering systems that support it, are an integral part of the visitor's experience. The most notable architectural element is the aquarium's signature roof, which was inspired by and mimics the form of a scallop shell. The glass-covered, shell-shaped dome roof encloses the 20,000 square foot, 65-foot high column-free Florida Wetlands exhibit space. Eight arched trusses spring radially from a central buttress at the shell's apex to form the dome's thematic shape. The use of arched trusses, wind tunnel tests, a refined three-dimensional analysis, tube shapes, and intricate detailing made possible the light, airy structure that complements the exhibits below.

Architect on the project was

Photo courtesy of Aerial Innovations

Photo courtesy of Aerial Innovations

HOK / EHDD, a joint venture of Hellmuth, Obata & Kassabaum, Inc., Tampa and St. Louis, and Esherick Homsey Dodge & Davis, San Francisco, and wind consultant was Cermak Peterka Peterson, Inc., Fort Collins, CO.

Because the many live plants and trees require large amounts of natural light to thrive, the roof structure was required to be as open and unobstructive as possible. At the same time, the glass-covered dome has to stand up to the 100 mph hurricane winds expected along the Florida Gulf coast. In response, the structural engineer worked closely with the architect to develop an ethereal but strong structural system that is free from large gusseted connections and other obstructions. Various truss schemes were investigated, including trusses of wide flange shapes, tubes, and pipes. Tube trusses were selected because of their strength, architectural appeal, ease of connection, and relatively low cost.

Arched Trusses

The arched trusses, which span as much as 140 feet, vary in depth along their length from four feet at the apex to eight feet at the base. Truss top and bottom chords are segmented at panel points to follow the curved roof geometry. The chords are 8-in. x 4-in. tubes with wall thicknesses varying from $^1/_4$-in. to $^1/_2$-in. as required for strength. The segmented chords are full-penetration welded at each panel point. The truss web diagonals are tubes as well, varying from TS2.5x2.5x$^3/_{16}$ to TS3.5x3.5x$^1/_4$. Connections of web diagonals to truss chords are fillet welded or partial penetration welded, depending upon diagonal size. This eliminated the need for gusset plates, resulting in aesthetic, unobtrusive connections.

Structural engineer on the project was Walter P. Moore and Associates, Inc., Tampa, and mechanical/electrical engineer was Syska and Hennessy, New York City.

Photo courtesy of Florida Aquarium

The arched trusses are supported at the shell's apex by one towering concrete buttress column. The trusses come to a point at the apex and are field-bolted to heavy W12s that are embedded in the concrete buttress. Each truss is supported at its far end (the shell's base) by a curved concrete buttress. A simple anchor bolt connection is used at these supports. Each of the arched trusses was delivered to the site in two pieces, spliced together on the ground with full-penetration welds, and then lifted into place. Two cranes were used to erect the trusses.

To create the scallop shell shape, the radial roof trusses are sequentially stepped. As a result, the roof purlins span from the top chord of each lower truss to the bottom chord of the adjacent higher truss. Faceted "horizontal" trusses between the chords of adjacent arched trusses provide lateral resistance and overall roof stability. These trusses follow the curved profile of the roof and share a common chord with the arched vertical trusses.

Tube purlins spanning between the roof trusses vary in size from TS 6x4 near the shell's apex to TS 20x12 at the shell's base. For aesthetic reasons, the purlin's depth is always oriented perpendicular to the curved roof surface, resulting in biaxial bending and torsional loads. Additionally, because the purlins help to form the lateral-load resisting trusses, they are also subjected to axial loading. Purlin connection design was complicated because it was necessary to accommodate the biaxial shears, axial loads and torsion, and yet maintain small, aesthetic, and easily constructable connections. Connection plates that are only slightly larger than the tube dimensions are shop-welded to the purlins and trusses; the field connections are bolted with slip-critical bolts.

Steel fabricator and erector on the project was AISC-member Tampa Steel Erecting; detailing was performed by Structural Technics in Irondale, AL. General contractor was Turner/Kajima, a joint venture of Turner Construction Company, Orlando and Kajima Construction Services, Atlanta.

COMPUTER-AIDED DESIGN

Close coordination among the designers and builders helped to accurately analyze, detail, and construct the complex geometry of the roof system. The architect developed the roof geometry using a three-dimensional CAD model developed on STAAD-III. Top and bottom truss chords are segmented to form non-concentric circles. The architect's CAD

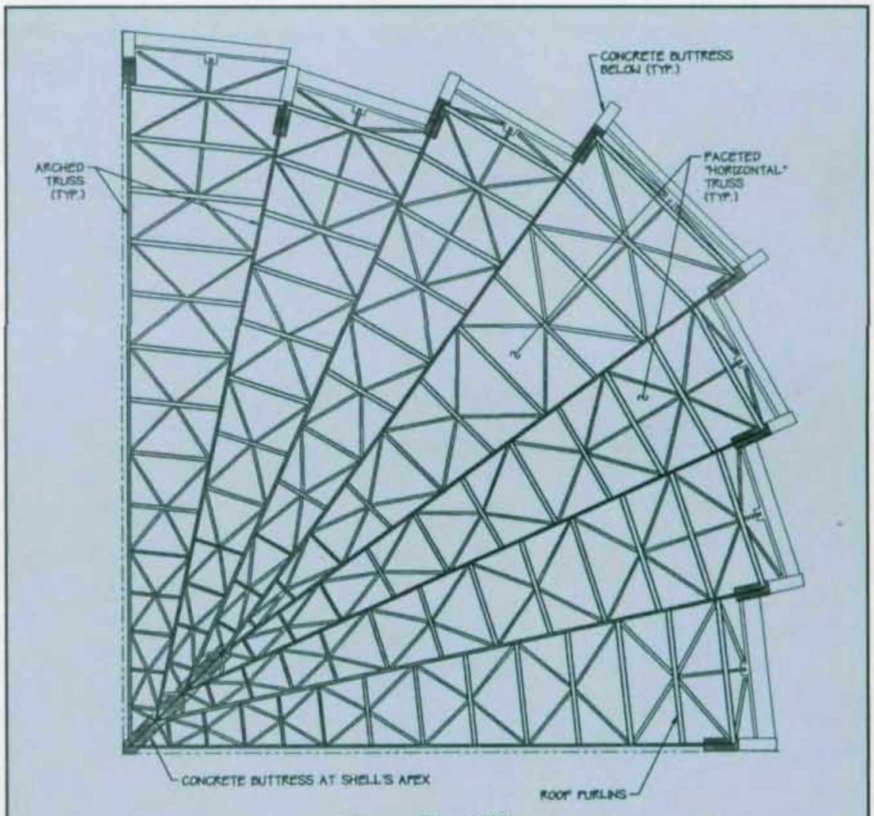
geometry model was then used as a basis for a three-dimensional computer model for the structural analysis and design. The structural computer model consisted of more than 900 members and 1,900 degrees of freedom. The steel detailer used the structural computer model and CAD files as an aid in generating the shop drawings. In addition, some two dimensional drawings were developed using RISA 2D.

To assure high reflectivity, durability, and ease of maintenance, the roof steel is painted with an off-white high-build epoxy-based paint system. Each roof component was primed with a zinc primer and then painted in the shop with two coats of paint. To protect the paint, nylon lifting straps were used to erect the steel. Touch-up work and bolted connections were painted in the field prior to the application of a final 2 mil semi-gloss finish coat to the entire in-place dome structure. The overall paint system thickness is 18 mils.

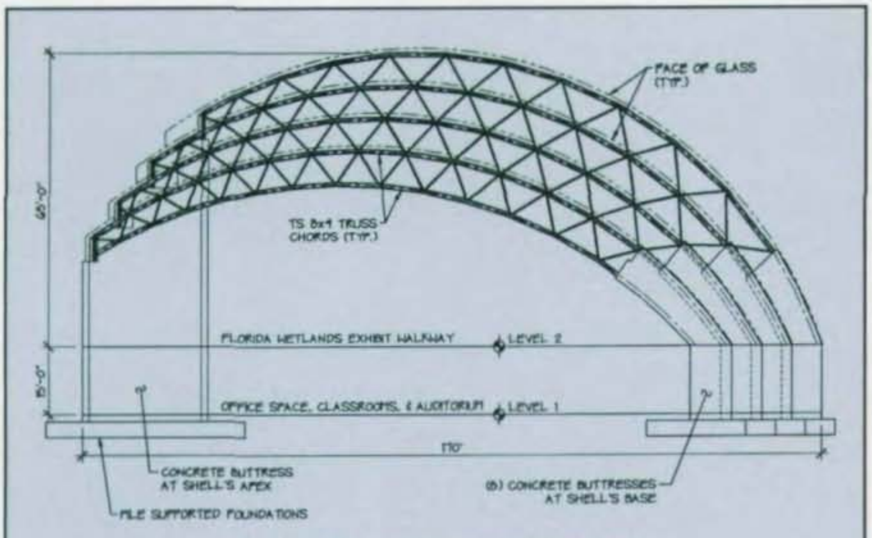
Because the majority of the structural steel is over 20 feet above the floor, fireproofing was not required. The small amount of steel below the 20-foot fire-protection threshold was protected by increasing the density and capacity of the sprinklers and using sprinklers that are activated at a temperature lower than conventionally required.

Comprehensive wind tunnel tests were conducted on the uniquely shaped roof. The tests were based on historical wind speed data for the Tampa Bay area and a 100-year recurrence interval. While the wind tunnel tests did reveal some localized areas of high pressure, the pressures for the most part were less than code-prescribed values. This allowed the use of reduced design wind loads, resulting in a cost savings for both the structure and the glass and glazing system. The lower wind pressures, in conjunction with an optimized three-dimensional structural analysis, reduced the structural steel weight by 25 percent compared to preliminary designs based on the higher code-prescribed values.

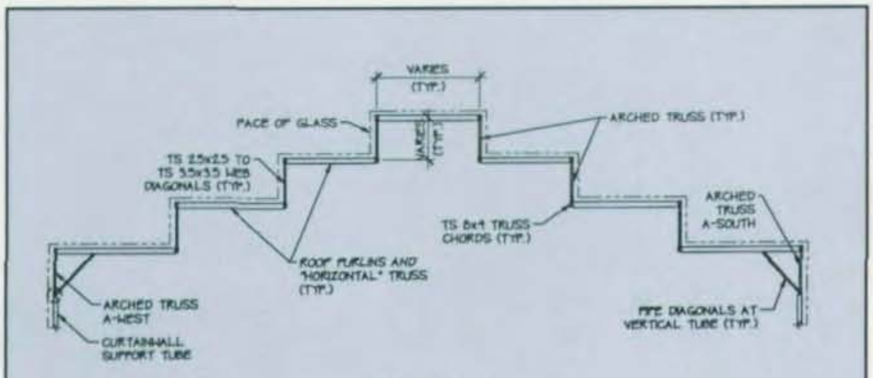
The shell-shaped dome contains over 110 tons of steel tubes and connection material, equating to a relatively modest structural steel weight of 11 psf. Another 9 tons of steel tubes support the glass-covered sides



Dome Roof Plan



Arched Truss Geometry Elevation



Roof Section

of the dome. The dome is enclosed with 1,100 laminated glass panels, most of which have different geometries. Despite the complicated roof geometry, construction of the shell-shaped dome went smoothly and was completed ahead of schedule. Erection of the structural steel was completed in only four weeks. Glass and glazing installation took five months to complete.

In addition to the dome roof, 43 tons of structural steel create a skeleton for fabric-covered parasail-shaped canopies that protect open-air saltwater exhibit tanks from the sun and elements. Curved TS 8x3 purlins shape the canopies and are supported by tubular beams spanning over the tanks. Translucent vinyl fabric is stretched over the curved tube frames. The steel tubes are protected from the corrosive saltwater environment with the same durable coating system used for the dome roof.

SUPPORTING THE EXHIBITS

The aquarium's large exhibit tanks also required careful design. The tanks are constructed of high-performance reinforced concrete for durability and water-tightness. Various reinforcing steel corrosion protection schemes were investigated, including silica fume concrete, corrosion inhibitors, epoxy coated reinforcing steel, and cathodic protection. The selected scheme incorporates high-performance concrete and uncoated reinforcing steel. The concrete mix design includes the use of a high cement content, fly ash, and silica fume for all tanks, resulting in a very dense concrete matrix. Additionally, a corrosion inhibitor was incorporated into the concrete for all brackish and saltwater tanks. Extra concrete cover, special concrete placing and curing procedures, and a crystalline forming water-proofing system further enhance tank durability.

The Coral Reef tank, the largest in the aquarium, is up to 26 feet deep and holds 500,000 gallons of saltwater. The concrete tanks are highly irregular in shape, with numerous viewing windows penetrating the tank walls. Accommodating the acrylic windows required special detailing and close coordination with the architects, exhibit designers, and acrylic window manufacturer. The largest acrylic panel, 14 feet tall by 43 feet wide, is 12" thick, weighs 24 tons, and provides a panoramic view of the colorful coral reef habitat. Over 100 acrylic panels are incorporated in tanks throughout the aquarium.

The project was financed by the sale of an $84 million bond issue backed by the City of Tampa. The aquarium is projected to draw over 1.8 million visitors annually, creating 1,700 jobs and a $210 million impact to the local economy. The success of the aquarium is aided by the fact that there are over 5 million people within a 100 mile radius and the Tampa Bay area is a popular tourist destination.

The Florida Aquarium's grand opening on March 31, 1995, was a memorable event full of fanfare, complete with marching bands and fireworks. In line with its educational theme, 350 school children participated in the opening ceremony. The aquarium made its debut on national television's Good Morning America as a crowd of over 1,000 looked on, and 10,000 visitors toured the aquarium on opening day. The facility has been well received, accommodating over 150,000 guests in the first month. The Florida Aquarium provides visitors with an exciting and entertaining educational experience, made possible by extraordinary design and construction teamwork, creative architectural design, and structural ingenuity.

David W. Landis, P.E., is a Senior Associate and Blair K. Hanuschak, E.I., is an Associate with Walter P. Moore and Associates, Inc. Landis and Hanuschak served, respectively, as structural project manager and project engineer. Walter P. Moore and Associates provided all structural engineering services for The Florida Aquarium project. Walter P. Moore and Associates maintains offices in Houston, Dallas, Tampa and Atlanta, and provides diverse building-related structural, civil and traffic engineering services throughout the US and internationally.

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Herman Miles Trucking Inc., El Paso, Texas, 8,500 sq. ft.;
Architect: Joe M. James, AIA; Structural Engineer: Pedro Herrera; General Contractor: Balboa Enterprises, Inc.; Steel Fabricator: Area Iron & Steel

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Please circle # 37

GALVANIZING REDUCES MAINTENANCE CONCERNS

The use of galvanized steel tubes for a new conservatory provided strength, aesthetics and a maintenance-free structural system

By Timothy W. Porritt, P.E.

WHEN THE WEST MICHIGAN HORTICULTURAL SOCIETY FIRST BEGAN PLANNING A MAJOR BOTANICAL AND SCULPTURAL GARDEN, they envisioned a facility that would not just attract visitors from around the world, but would also serve as a valuable educational resource for the community. That vision is now a reality on 70 acres of land purchased with private funding on the east side of Grand Rapids, MI.

The focal point of the facility is the glass conservatory, which rises the equivalent of five stories above the site and has a floor area of 15,000 sq. ft. Within the glass walls are beautiful landscapes of plants and trees, meandering walking paths, and a waterfall cascading over a cave and into a stream that flows below footbridges and rock formations. The structure consists of insulated panels supported by an aluminum frame, with a conventional modular greenhouse roof including powered vents, sun screens, misting systems and other mechanical systems to control the tropical environment.

Next to the conservatory is a 17,400-sq.-ft building housing the facility's entrance, administration space, tour information/reception area, gift shop, a cafe with outdoor terrace, classrooms/banquet facilities, rest-rooms, multimedia center and library. The building is a conventional steel-framed structure with a masonry and limestone exterior.

Connecting the two main structures is a gallery/corridor for displays and art shows. It also provides access to the specialty greenhouses. The long, curved gallery is constructed of masonry and conventional steel framing, and highlights the very unique reinforced concrete trees that serve as support columns. The specialty greenhouses, which house other climates and activities, are modular systems. The gallery floor area is 4,700 sq. ft. and the specialty greenhouses total 8,000 sq. ft.

THE CONSERVATORY

The entire conservatory was designed around a modular, standard greenhouse system, with a modular vented-glass roof system supplied by a greenhouse manufacturer. However, the shear size of the space, coupled with a desire for the highest level of natural light possible, mandated the design of a unique structural system. In addition, the northern location of the conservatory made wind and snow loadings critical, while strict deflection criteria had to be met because of the glass systems. The deflections were determined by applying lateral and gravity loadings to a computer model generated and analyzed using SAP 90.

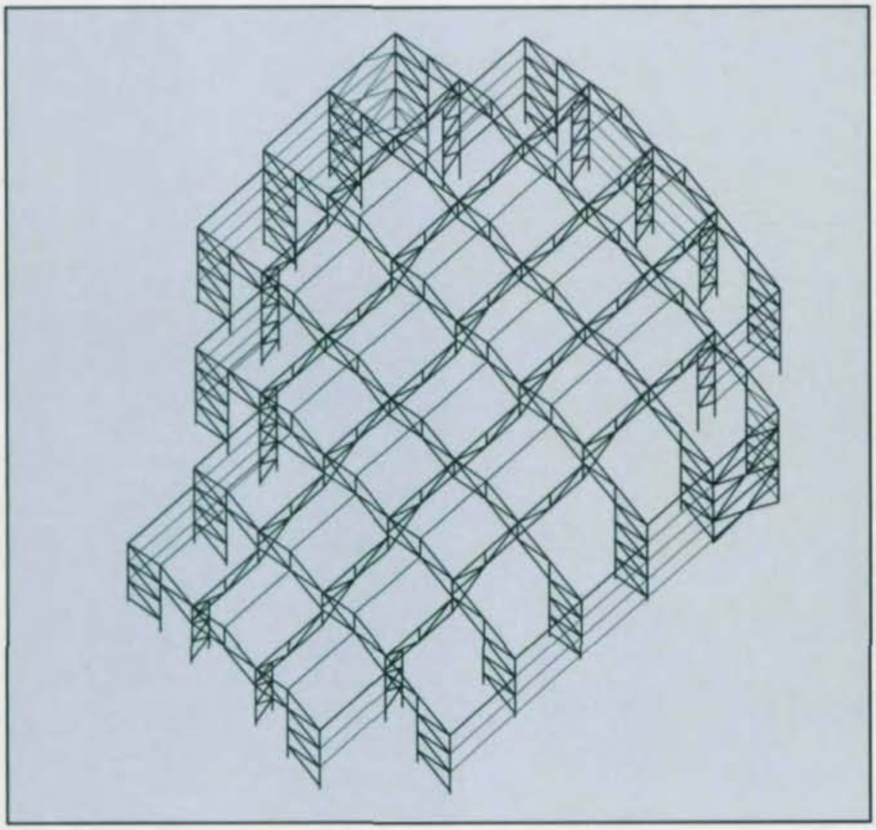
The interior environment influenced the selection of materials and the manner in which they integrated with each other. The atmosphere within the conservatory is tropical and a high level of moisture is present year-round. Structural steel tube trusses were selected for the structural members not just for their performance characteristics, but also because it minimized exposed surface area and created an attractive appearance. However, because of the desire to create a maintenance-free structure—and because of

the potential for damage to foliage in any re-painting operation—the structural tubes were hot-dipped galvanized. Galvanizer on the project was Columbus Galvanizing-Voigt & Schweitzer, Inc. and Voigt & Schweitzer Galvanizers, Inc.

From the exterior, the glass-enclosed conservatory presents a steeped appearance. The glass is supported by a series of truss/frames of varying sizes with spans ranging from 63 ft. to 126 ft. These truss/frames are constructed of TS8x8 and TS8x4 structural tubes. The components for the Frederik Meijer Gardens are the largest possible, taken into account limitations for shipping, galvanizing and erection. The depth between compression and tension chords is typically 10-ft.-6-in., which also is the dimension of the greenhouse manufacturer's modular roof components. The top of these truss/frames forms a saw-tooth configuration. Optimizing the design required close coordination between the steel fabricator, galvanizer and erector.

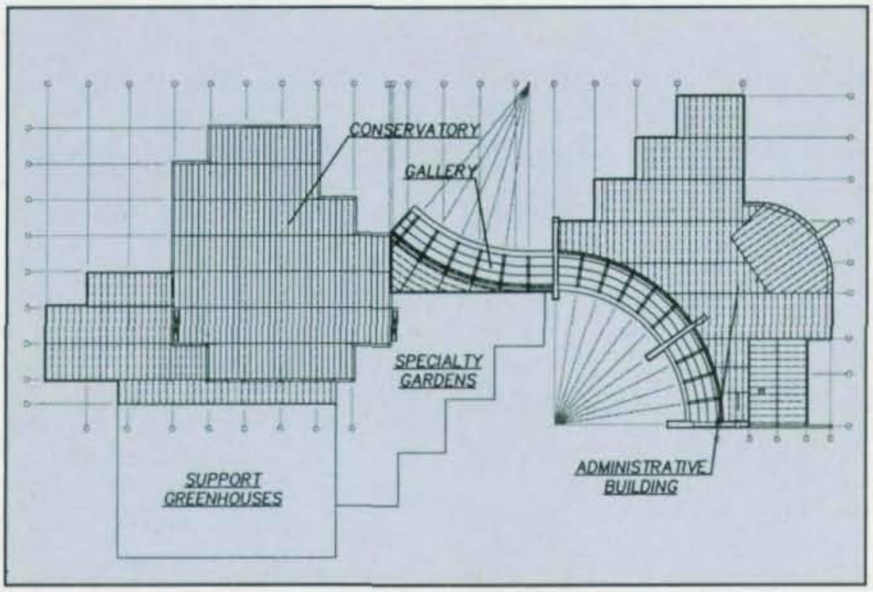
Web members of the truss/frames were continuously welded to the main chord members. Prior to welding, small-diameter ports were drilled between members so the steel could be coated galvanically on the inside of all tube surfaces. This also prevents components of the trusses from floating in the dip tanks. Connections between truss components are field-bolted butt plates with galvanized bolts; all other connections are through-bolted tubes attached to welded clip angles. No field cutting or welding was allowed, eliminating possible damage to the galvanized coating, which could lead to discoloring and staining of the structural steel over time. The glass systems are supported by structural tube purlins at the roof and along the walls between truss/frames. These tubes are designed to transfer both wind and gravity loads to the truss/frames.

The main truss/frames are lat-



erally supported by a combination of wind truss/frames and cross bracing where frames were not physically possible. The truss/frames are located in the interior of the building spaced 21-ft. o.c. The overall depth between compression and tension chords is 8-ft.-3-in. Cross braces are used along exterior walls and perpendicular to the main truss/frames.

There is a concrete mechanical tunnel 8-ft. wide by 12-ft. deep below grade following the perimeter of the conservatory that also functions as the foundation for the truss/frames. This tunnel carries water, air and heat to the conservatory and acts



as a retaining wall to support as much as 23 ft. of soil.

Administration Building

The administration building is a single-story structure of steel framing, steel joists and metal deck enclosed by brick, limestone and glass. LRFD was used to design the framing for this structure. It was a challenge to come up with a framing system that would conceal the steel and still offer some economical advantages of modular spacing of structural members. The structural steel members include W16 wide flange beams and 16K steel joists supporting $1^1/_2$-in.-wide rib metal deck. The corridor that links the gallery has exposed structural framing supported by on-site precast concrete trees. These unique tress are 22-ft. high and spaced 15-ft. o.c. Support for the concrete trees is provided by combined footings used grade beams and spread footings.

Gallery/Corridor

The gallery linking the conservatory to the administration building is constructed with more conventional steel beam and joist framing. The support for this framing also is provided by the reinforced concrete trees. The trees are a popular, repetitive theme that begins at the main entrance and ends at the doorway to the conservatory.

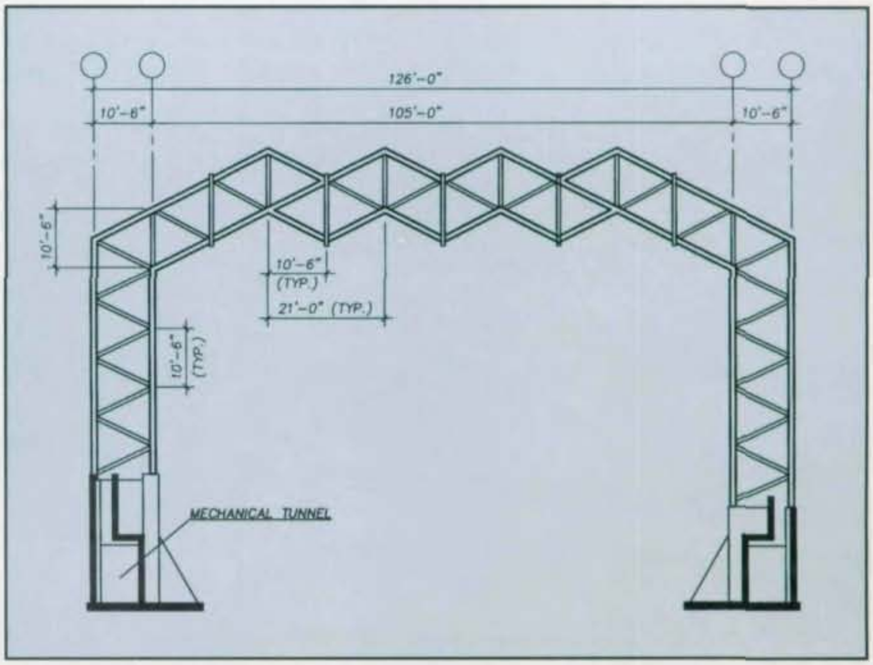
ERECTION

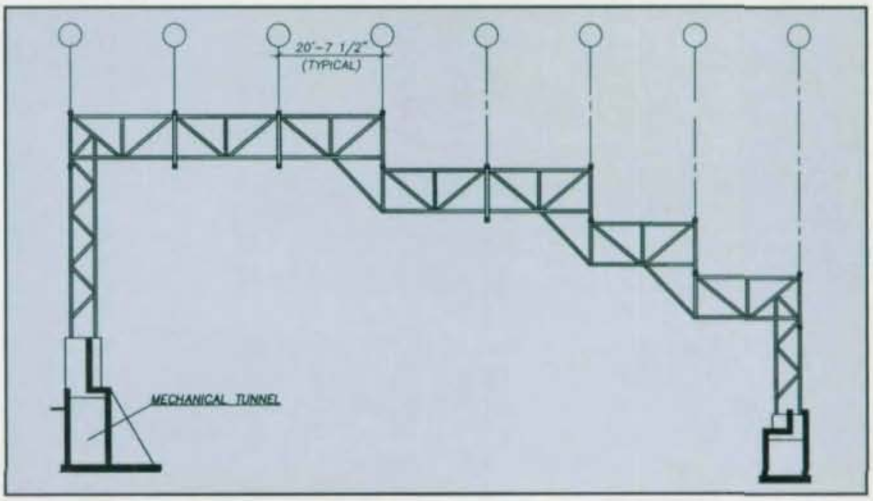
The erection went smoothly and began at the tallest end of the conservatory with the first main truss/frame. Each truss/frame was erected from the base on both ends, placing components sequentially, working up the frame then toward the middle, until the last segment was placed to form the full truss/frame. After the first truss/frame was installed, segments of the wind truss/frames were constructed to provide stability during erection. This sequence continued along the length of the structure until it was complete, with only minor shoring or bracing required.

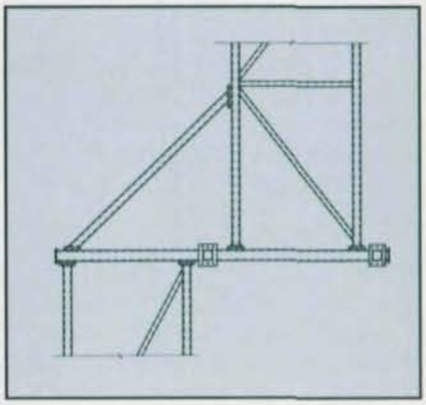
Approximately 186 tons of galvanized structural tubing were used in the conservatory and 104 tons to construct the remaining support facilities and administration building. The sloping corridor roof is protected by 7,300 sq. ft. of standing seam copper roofing. The remaining flat roof area is covered by 23,700 sq. ft. of $1^1/_2$-in. Type B metal deck. The project required 18 tons of steel roof joists, 280 tons of 4-in. split-face Wisconsin limestone, and 3,200 cu. yds. of concrete.

The overall project cost, including all the planting and sculptures, reached nearly $20 million. The constructed buildings had a price tag of approximately $8.5 million, with the structural steel, steel joists, and metal deck—including material and erection—costing approximately $665,000 or $13.75/sq. ft.

Timothy W. Porritt, P.E., is an associate with Fishbeck, Thompson, Carr and Huber, Inc.—Engineers, Scientists, Architects, headquartered in Grand Rapids, MI. The Frederick Meijer Gardens recently was honored by the American Galvanizers Association with both their 1995 Architectural/Building Construction Award and their 1995 Most Distinguished Project Award.







TIGHT FIT

Despite a narrow, 150-ft.-by-35-ft. site, the designers of a new collegiate fitness center were determined to to create an open feeling

By Eliot W. Goldstein, AIA and Glenn A. Wrigley

OVER THE PAST SEVERAL DECADES, UNION COUNTY COLLEGE in Cranford, NJ, has grown considerably, both in student population and physical plant. However, what has not—and could not—change was the size of its campus, which is limited to a narrow 48-acres. As a result, each new building on campus was constructed in close proximity to existing ones.

The latest addition is a new Fitness Center. The program called for a 16,000-sq.-ft. facility with the school's shipping & receiving center, central storage, maintenance workshops and print shop on the first floor, and a 5,000-sq.-ft. fitness center on the second floor, along with physical plant offices and several new classrooms. As a result of the campus' space limitations, the new building had to be constructed on a narrow, L-shaped site squeezed between existing campus buildings and inviolate parking lots. The highly constrained site included a long leg on a sliver of land 150-ft. long and just 35-ft. wide.

Several difficulties arose due to the unusual site constraints. The new building would wrap around portions of two existing buildings. However, for one of these buildings, the unsprinklered gymnasium, the loss of this "street frontage" meant that its actual floor area would be in excess of the area allowed by the existing building code for an unsprinklered building. To satisfy the building code, the existing gym had to be sprinklered and the new construction had to be separated from the existing one by rated walls.

Another unusual code problem emerged from the desire to add enough windows to bring substantial natural light into the building. The code constraint was related to the proximity of the addition's clerestory windows to the outside face of the existing building. The clerestory windows, at a distance of 4-ft. from that surface, were limited to 15% of the area of the wall in which they were located. Since the fire wall was offset from the plane of the clerestory, the overall area of the clerestory wall was relatively small, resulting in a minuscule allowable window area. However, if the clerestory were sprinklered both in and out, and the roof between the clerestory and the fire wall were of protected construction, the authorities were willing to consider the overall assembly as an offset fire wall, in which the area of the fire wall could be considered in the calculation of the allowable area of the clerestory windows.

Non-code related challenges included satisfying ADA guidelines for the toilets, locker rooms and shower rooms, and adding an elevator accessible from the existing buildings.

Design Concepts

The existing campus was constructed in several distinct phases, each with its own architectural vocabulary. The earliest buildings were characterized by large expanses of brick and simple, punched windows—some with decorative sunscreens. The second phase was Brutalist: glazing occupied the spaces between three-dimensional brick and concrete forms. The third phase, which began with the construction of the nearby Student Commons, was characterized by an exposed steel structure, large curtain walls, and facades divided into bases, middles and tops.

For the exterior of the new building, the designers opted to exploit its prominent location at the western end of the network of interconnected campus buildings by creating a distinctive curving roof on the long wing, which rises above the existing gym. The fitness center, occupying the entire area directly under this roof, is glazed on all four sides. On the high side, the glazing is in the form of a clerestory, filling the space between the existing roof of the gym and the higher roof of the new building. To the west, the view is toward an existing pond. The vocabulary of the new building's facades is similar to that of the nearby student commons: brick on the first floor, decorative concrete block above.

Given the forms and materials of contemporary exercise machines, and the static and dynamic loads imparted by their users, the designers decided that an exposed steel superstructure was appropriate for the center. The simplest way of spanning the space would have been to use inclined members running in the short direction. However, we were concerned that the result would be of little visual interest, a major issue in an environment where the users often spend long periods working out in place.

Instead, we explored a variety of roof forms, trying to find a balance between spatial richness and construction economy. The form we eventually settled on consists, in building section, of a tilted flattened "S", where both its arcs are identical but oriented in opposite directions.

The fitness center is essentially one large 35-ft.-by-150-ft. room. A single longitudinal row of interior columns separates the workout area from the corridor. An interior curtain wall separates the apparatus areas from

Building adjacent to an existing building was difficult not just because of the site constraints, but also because the new construction altered the fire rating of the existing building.

the aerobics area. The curve of the roof is extruded the entire length of the fitness center, and the underside of its structure is exposed to view. Acoustic metal roof deck spans between identical curved steel tubes spaced at 10-ft. o.c. and supported by longitudinal edge beams. Uplights illuminate the structure. Only the corridor has a suspended ceiling; all of the ductwork and much of the sprinkler piping is concealed above it.

The long narrow leg contains the large unpartitioned areas (fitness center on the second level, central storage area on the first level). The shorter leg contains a variety of partitioned functions. To maximize the efficiency of the plan, circulation was placed along the inside of the L, where it would be shortest. The elevator was located at the juncture of existing and new corridors, and two new fire stairs were located at opposite ends of the long wing.

Building Materials

To avoid concentrated loads near existing exterior walls, new columns were kept 5-ft. away from them, and the structure was cantilevered toward them. The new moment-resisting steel structure is supported on concrete footings, piers and foundation walls. Roofing consists of single-ply membranes over higher-performance insulation board. Interior finishes included painted drywall, decorative block, and in the fitness center, rubber flooring. High-performance glazing is used throughout.

To satisfy code constraints, it was necessary that sprinklering of the gym be completed prior to erection of the superstructure. Also, it was crucial that erection of the superstructure was completed prior to the fall 1994 semester. As a result, the project was fast tracked and constructed under two bids—one for site utilities, foundations and superstructure; the second for enclosure, operating systems and finishing.

Phase 1 general contractor was Tormee Construction, Tinton Falls, NJ, and Phase 2 general contractor was The Conklin Corporation, Franklin Lakes, NJ. Architect on the project was The Goldstein Partnership, West Orange, NJ, structural engineer was Klein Associates, Mountainside, NJ, and mechanical/electrical contractor was Syska & Hennessy Engineers.

STRUCTURAL DESIGN

"The compound curve roof form (sine wave) evolved from the architect's desire both for natural interior lighting at the interface of the existing building and for a fitting form for the fitness center," according to Allan Klein, P.E., principal and project engineer with Klein Associates. "To maximize volume, reduce acoustic echoes and reduce costs, an exposed acoustic metal deck was chosen as the roof diaphragm. The continuous run of clerestory windows provided light and transition from the curved ceiling to the existing roof."

The basic semi-rigid bent was composed of W8x67 (50 ksi) columns, W24x94 floor beams spanning 30-ft., and an 8x12x$^1/_2$ curved tube roof beam. The tubular rectangular roof beam was chosen as the most efficient shape, while the 8x12x$^1/_2$ tube provided the least weight, strongest section. In addition, the tube greatly reduced the distortion usually associated with a curved wide flange beam bent about its strong axis. Five-ft. cantilevered brackets were welded to the columns and span the gap between the new column line adjacent to the existing building and the code-mandated three-hour fire-rated masonry wall between and independent of both the existing construction and the addition, Klein explained.

"The excellent torsional rigidity provided by the tubes, he added, greatly reduced secondary stresses. Lateral bracing was provided by upturned ST6x17.5s welded to the tops of the tubes. Because this orientation concealed their vertical legs within the thickness of the roof deck, a clean unbroken reflected ceiling plan resulted. The tubular 8x12x$^1/_2$s were bent in one piece by Reco Nelson of Newark, NJ. The 8x12x$^1/_2$ tubular spandrel girders and intermediate curved roof beams completed the roof framing and simplified connections, while providing an efficient moment-resisting frame capable of accommodating wind and earthquake forces.

Within the wing of the building containing the fitness center, the second floor platform required a total of 14.2 psf of structural steel, while its roof required a total of 13.7 psf of structural steel.

Eliot W. Goldstein, AIA, is a partner and Glenn A. Wrigley is a project manager with The Goldstein Partnership, West Orange, NJ.

Third of three parts

Essentials Of LRFD

An overview of LRFD as found in Part 2 of the *Manual of Steel Construction* (1994)

NEARLY A DECADE AGO, THE AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC. (AISC) BEGAN AN INDUSTRY-WIDE TRANSITION from Allowable Stress Design (ASD) to Load and Resistance Factor Design (LRFD). While acceptance has been slow, momentum is gathering: A recent Gallup poll showed that industry acceptance of LRFD is growing and a majority of structural engineers now believe that LRFD is the steel design method of the future. Some people, however, have interpreted the existence of two specifications as an indication of an unclear direction. Therefore, the AISC Board of Directors has adopted the following resolution:

"Based upon expert input from its Committee on Specifications, the Board of Directors of AISC affirms that the *1993 Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings* is the preferred Specification for the fabricated structural steel industry. LRFD is a modern and technologically superior steel design specification. Its direct representation of ultimate structural behavior is especially relevant for seismic design, design of frames with partially restrained connections, and composite systems design. It offers engineers the opportunity to innovate in the analysis and design of highly reliable and competitive steel structures by encouraging the consideration of strength and serviceability criteria under appropriate combinations of gravity and lateral loads. In this way, LRFD is consistent with the prevailing trend toward limit-states design in all materials, both domestically and internationally."

This article is the third of a three-part summary of LRFD (part one appeared in June and part two appeared in July).

H. Members Under Combined Forces And Torsion

Symmetric Members Subject to Bending and Axial Tension

The interaction of flexure and tension in singly and doubly symmetric shapes is governed byEquations H1-1a and H1-1b, as follows:

For $\frac{P_u}{\phi P_n} \geq 0.2$,

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad \text{(H1-1a)}$$

For $\frac{P_u}{\phi P_n} < 0.2$,

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad \text{(H1-1b)}$$

where:

P_u = required tensile strength; i.e. total factored tensile force, kips

ϕP_n = design tensile strength, $\phi_t P_n$, kips

ϕ = resistance factor for tension, $\phi_t = 0.90$

P_n = nominal tensile strength as defined in Chapter D of the LRFD Specification, kips

M_u = required flexural strength; i.e., the moment due to the total factored load, kip-in. or kip-ft. (Subscript x or y denotes the axis about which bending occurs.)

$\phi_b M_n$ = design flexural strength, kip-in. or kip-ft.

ϕ_b = resistance factor for flexure, $\phi_t = 0.90$

M_n = nominal flexural strength determined in accordance with the appropriate equations in Chapter F of the LRFD Specification, kip-in. or kip-ft.

Interaction Equations H1-1a and H1-1b cover the general case of biaxial bending combined with axial force. They also are valid for uniaxial bending (i.e., when M_{ux} = *0 or* $M_{uy} = 0$). *In this case they reduce to the form plotted* in Figure H-1. Pure biaxial bending (with $P_u = 0$) is covered by Equation H1-1b.

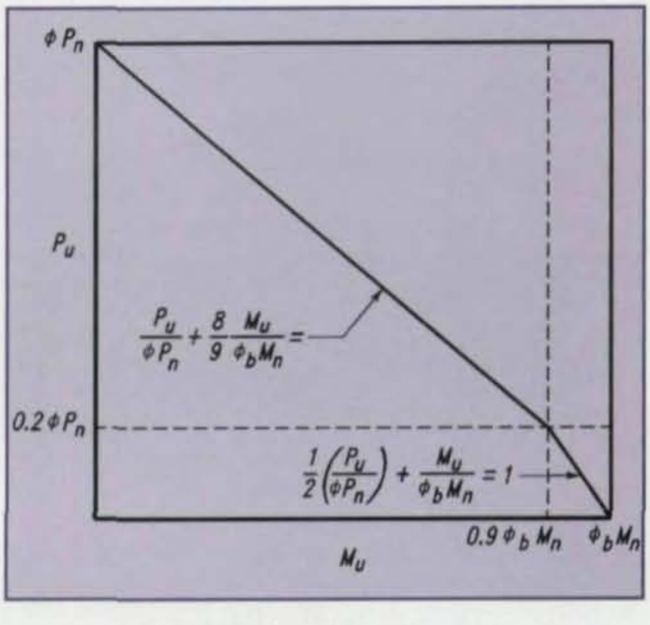


Figure H-1: Interaction Equations H1-1a and H1-1b modified for axial load combined with bending about one axis only.

Symmetric Members Subject to Bending and Axial Compression

The interaction of compression and flexure in beam-columns with singly and doubly symmetric cross sections is governed by Equations H1-1a and H1-1b.

ϕP_n = design compressive strength, $\phi_c P_n$, kips

ϕ = resistance factor for compression, $\phi_c = 0.85$

P_u = required compressive strength; i.e., the total factored compressive force, kips

P_n = nominal compressive strength as defined in Chapter E of the LRFD Specification, kips

M_u = required flexural strength including second-order effects, kip-in. or kip-ft.

$\phi_b M_n$ = required flexural strength, kip-in. or kip-ft.

ϕ_b = resistance factor for flexure = 0.90

M_n = nominal flexural strength from Chapter F of the LRFD Specification, kip-in. or kip-ft.

The second-order analysis required for M_u involves the determination of the additional moment due to the action of the axial compressive forces on a deformed structure. In lieu of a second-order analysis, the simplified method given in Chapter C of the LRFD Specification (and in Section C of the June issue) may be used. However, in applying the simplified method, the additional moments obtained for beam-columns must also be distributed to connected members and connections (to satisfy equilibrium).

Example H-1

Given: Check the adequacy of a W10x22 tension member of 50 ksi steel to cary loads resulting in the following factored load combination:

P_u = 55 kips
M_{uy} = 20 kip-ft.
M_{ux} = 0

Solution: From Section D for 50 ksi steel,

$$\phi P_n = \phi_t P_n = 45.0 \text{ ksi} \times 6.49 \text{ in}^2 = 292 \text{ kips}$$

$$\frac{P_u}{\phi P_n} = \frac{55 \; kips}{292 \; kips} = 0.188 < 0.20;$$

therefore Equation H1 – 1b governs.

For bending about the y axis only, Equation H1-1b becomes:

$$\frac{P_u}{2\phi P_n} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0$$

From Section F for 50 ksi steel, $M_n = M_p = Z_y F_y = 50$ ksi x Z_y for minor-axis bending (regardless of unbraced length).

$$\phi_b M_{ny} = 0.90 \times 50 \; ksi \times Z_y = 45.0 \; ksi \times Z_y$$

$$= 45.0 \; ksi \times \frac{6.10 \; in.^3}{12 \; in./ft.}$$

$$= 22.9 \; kip\text{–}ft. \text{ for a W10x22 member}$$

$$\frac{P_u}{2\phi P_n} + \frac{M_{uy}}{\phi_b M_{ny}} = \frac{0.188}{2} + \frac{20 \; kip\text{–}ft.}{22.9 \; kip\text{–}ft.} = 0.094 + 0.873$$

$$= 0.967 < 1.0 \quad \textbf{o.k.}$$

Example H-2

Given: Check the same tension member, a W10x22 in 50 ksi steel, 4.0-ft. long, subjected to the following combination of factored loads:

P_u = 140 kips
M_{ux} = 55 kip-ft.
M_{uy} = 0
C_b = 1.0

Solution: Again, ϕP_n = 292 kips

$$\frac{P_u}{\phi P_n} = \frac{140 \; kips}{292 \; kips} = 0.479 > 0.20;$$

therefore Equation H1 – 1a governs.

For bending about the x axis only, Equation H1-1a becomes:

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\frac{M_{ux}}{\phi_b M_{nx}} \leq 1.0$$

From Section F for 50 ksi steel, $M_n = M_p = Z_x F_y = 50$ ksi x Z_x for major-axis bending if $L_b \leq L_p$ for (C_b = 1.0).
Assume unbraced length, L_b = 4.0 ft.
By equation F1-4a in Section F, $L_p = 42.4 r_y$ for 50 ksi steel.
For a W10x22, r_y = 1.33 in., Z_x = 26.0 in.^{3}.

$$L_p = \frac{42.4 \times 1.33 \; in.}{12 \; in./ft.} = 4.7 \; ft.$$

$$L_b = 4.0 \; ft. < L_p = 4.7 \; ft.$$

Then M_{nx} = 50 ksi x Z_x

$$\phi_b M_{nx} = \frac{0.90 \times 50 \; ksi \times 26.0 \; in^3}{12 \; in./ft}$$

$$= 97.5 \; kip\text{–}ft. \text{ for a W10x22 member}$$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\frac{M_{ux}}{\phi_b M_{nx}} = 0.479 + \frac{8}{9} \times \frac{55 \; kip\text{–}ft.}{97.5 \; kip\text{–}ft.}$$

$$= 0.479 + 0.501 = 0.980 < 1.0 \quad \textbf{o.k.}$$

Bending and Axial Compression—Preliminary Design

The design of a beam-column is a trial and error process which can become tedious, particularly with the repeated solution of Interaction Equation H1-1a or H1-1b. A rapid method for the selection of a trial section is given in the LRFD Manual, Part 3, under the heading Combined Axial and Bending Loading (Interaction). As in earlier editions of the AISC Manual, the Interaction Equations are approximated by an equation which converts bending moments to equivalent axial loads:

$$P_{u\,eq} = P_u + M_{ux} m + M_{uy} m u$$

where:

$P_{u\,eq}$ = equivalent axial load to be checked against the column load table, kips

P_u, M_{ux}, M_{uy} are defined in the Interaction Equations for compression and bending

m, u are factors tablulated in the *LRFD Manual*, Part 3

As soon as a satisfactory trial section has been found (i.e., one for which $P_{u\,eq} \leq$ tabulated $\phi_c P_n$), a final verification should be made with the appropriate Interaction Equation, H1-1a or H1-1b.

Torsion and Combined Torsion, Flexure, and/or Axial Force

Criteria for members subjected to torsion and torsion combined with other forces are given in Section H2 of the LRFD Specification. They require the calculation of normal and shear stresses by elastic analysis of the member under the factored loads. AISC's *Torsional Analysis of Steel Members* (American Institute of Steel Construction, 1983) provides design aids and examples for the determination of torsional stresses. Extensive coverage is given there to wide-flange shapes (W, S, and HP), channels (C and MC) and Z shapes. For these members, the charts and formulas simplify considerably the calculation of torsional rotations, torsional normal and shear stresses, and the combination of torsional with flexural stresses.

In the LRFD Specification,

f_{un} = the total normal stress under factored load (ksi) from torsion and all other causes

f_{uv} = the total shear stress under factored load (ksi) from torsion and all other causes

The criteria are as follows:

a. For the limit state of yielding under normal stress,

$f_{un} \leq \phi F_y$, where $\phi = 0.90$ (H2-1)

For 50 ksi steel,

$f_{un} \leq 0.90 \times 50$ ksi = 45.0 ksi (2-24)

b. For the limit state of yielding under shear stress,

$f_{uv} \leq 0.60\phi F_y$, where $\phi = 0.90$ (H-2)

For 50 ksi steel,

$f_{uv} \leq 0.60 \times 0.90 \times 50$ ksi = 27.0 ksi (2-25)

c. For the limit state of buckling,

$f_{un} \leq \phi_c F_{cr}$ or $f_{uv} \leq \phi_c F_{cr}$, as applicable,

where $\phi_c = 0.85$ (H2-1)

For 50 ksi steel, values of $\phi_c F_{cr}$ are given in Table E-1, in Section E.

Torsion will accompany flexure when the line of action of a lateral load does not pass through the shear center. For wide flange and other doubly symmetric shapes, the shear center is located at the centroid. Singly symmetric shapes have their shear centers on the axis of symmetry, but not at the centroid. (The location of the shear center of channel sections is given in the Properties tables in Part 1 of the *LRFD Manual*.)

Open sections, such as wide-flange and channel, are very inefficient in resisting torsion; i.e., torsional rotations can be large and torsional stresses relatively high. It is best to avoid torsion by detailing the loads and reactions to act through the shear center of the member. In the case of spandrel members supporting building facade elements, this may not be possible. Heavy exterior masonry walls and stone panels can impose severe torsional loads on spandrel beams. The following are suggestions for eliminating or reducing this kind of torsion:

1.Wall elements may span between floors. The moment due to the eccentricity of the wall with respect to the edge beams can be resisted by lateral forces acting through the floor diaphragms. Torsion would not be imposed on the spandrel beams.

2.If facade panels extend only a partial story height below the floor line, the use of diagonal steel "kickers" may be possible. These light members would provide lateral support to the wall panels. Torsion from the panels would be resisted by forces originating from structural elements other than the spandrel beams.

3.Even if torsion must be resisted by the edge members, providing intermediate torsional supports can be helpful. Reducing the span over which the torsion acts will reduce torsional stresses. If there are secondary beams framing into a spandrel girder, the beams can act as intermediate torsional supports for the girder. By adding top and bottom moment plates to the connections of the beams with the girder, the bending resistances of

Example H-3

Given: Check the adequacy of a W14x176 beam-column, 14.0 ft. in height floor-to-floor, in a braced symmetrical frame in 50 ksi steel. The member is subjected to the following factored forces due to symmetrical gravity loads: P_u = 1,400 kips; M_x = 200 kip-ft., M_y = 70 kip-ft. (reverse curvature bending with equal end moments about both axes); and no loads along the member.

Solution: For a braced frame, $K = 1.0$; $K_xL_x = K_yL_y$ = 14.0 ft.

For a W14x176:

A = 51.8 in.^{2}

Z_x = 320 in.^{3}

Z_y = 163 in.^{3}

r_x = 6.43 in.

r_y = 4.02 in.

Kl/r_x = (14.0 ft. x 12 in./ft.)/6.43 in. = 26.1

Kl/r_y = (14.0 ft. x 12 in./ft.)/4.02 in. = 41.8

From Table E-1, $\phi_c F_{cr}$ = 37.4 ksi for Kl/r = 41.8 in 50 ksi steel.

$\phi_c P_n = (\phi_c F_{cr})A$ = 37.4 ksi x 51.8 in.2 = 1,940 kips

$$Since\ \frac{P_u}{\phi_c P_n} = \frac{1{,}400\ kips}{1{,}940\ kips} = 0.72 > 0.2;$$

Interaction Equation H1–1a ***governs***.

For a braced fram, $M_{lt} = 0$. From Equation C1-1:

$M_{ux} = B_{1x}M_{ntx}$, where M_{ntx} = 200 kip-ft.; and

$M_{uy} = B_{1y}M_{nty}$, where M_{nty} = 70 kip-ft.

From equations C1-2 and C1-3:

$$B_1 = \frac{C_m}{(1 - P_u / P_{e1})} > 1.0$$

where in this case (a braced frame with no transverse loading),

$C_m = 0.6 - 0.4(M_1/M_2)$

For reverse curvature bending and equal end moments:

M_1/M_2 = +1.0

C_m = 0.6 - 0.4(1.0) = 0.2

From Table C-1:

P_{e1x} = 420 ksi x A_g = 420 ksi x 51.8 in.^{2} = 21,756 kips

From Table C-1:

P_{e1y} = 164 ksi x A_g = 164 ksi x 51.8 in.^{2} = 8,495 kips

$$B_{1x} = \frac{C_{mx}}{1 - P_u / P_{e1x}} = \frac{0.2}{1 - 1{,}400\ kips / 21{,}756\ kips} = 0.2$$

Use B_{1x} = 1.0, per Equation C1-2.

$$B_{1y} = \frac{C_{my}}{1 - P_u / P_{e1y}} = \frac{0.2}{1 - 1{,}400\ kips / 8{,}495\ kips} = 0.2$$

Use B_{1y} = 1.0, per Equation C1-2.

M_{ux} = 1.0 x 200 kip-ft.

M_{uy} = 1.0 x 70 kip-ft.

From Equation 2-15 for 50 ksi steel,

$$L_p = 42.4 r_y = \frac{42.4 \times 4.02\ in.}{12\ in.\ /\ ft.} = 14.2\ ft.$$

Since L_b = 14.0 ft. < L_p = 14.2 ft., $M_{nx} = M_{px} = Z_xF_y$

$M_{ny} = M_{py} = Z_yF_y$

ϕ_bF_y = 0.90 x 50 ksi = 45.0 ksi

Example H-3, cont.

$$\phi_b M_{nx} = \phi_b F_y Z_x = \frac{45.0\ ksi \times 320\ in.^3}{12\ in./ft.} = 1{,}200\ kip\text{-}ft.$$

$$\phi_b M_{ny} = \phi_b F_y Z_y = \frac{45.0\ ksi \times 163\ in.^3}{12\ in./ft.} = 611\ kip\text{-}ft.$$

By Interaction Equation H1-1a

$$\frac{1{,}400\ kips}{1{,}940\ kips} + \frac{8}{9}\left(\frac{200\ kip\text{-}ft.}{1{,}200\ kip\text{-}ft.} + \frac{70\ kip\text{-}ft.}{611\ kip\text{-}ft.}\right)$$
$$= 0.72 + \frac{8}{9}(0.17 + 0.11) = 0.72 + 0.25 = 0.97 < 1.0$$

W14x176 is **o.k.**

Example H-4

Given: Check the adequacy of a W14x176 beam-column (Fy = 50 ksi) in an unbraced symmetrical frame subjected to the following factored forces: P_u = 1,400 kips (due to gravity plus wind); M_{ux} = 300 kip-ft. (due to wind only), M_y = 0 kip-ft.; and $K_xL_x = K_yL_y$ = 14.0 ft.
Drift index, $\Delta_{oh}/L \leq 0.0025$ (or $^1/_{400}$).
ΣP_u = 24,000 kips
ΣH = 800 kips

Solution: As in Example H-3, for a W14x176 with KL = 14.0 ft., $\phi_c P_n$ =1,940 kips.

$$Since\ \frac{P_u}{\phi_c P_n} = \frac{1{,}400\ kips}{1{,}940\ kips} = 0.72 > 0.2;$$

Interaction Equation H1-1a ***governs.***

Because $M_{ntx} = M_{nty} = M_{lty} = 0$ and $M_{ltx} \neq 0$, $M_{ux} = B_2 M_{ltx}$ and $M_{uy} = 0$.
M_{ltx} = 300 kip-ft.
According to Equation C1-4,

$$B_2 = \frac{1}{1 - \frac{\Sigma P_u}{\Sigma H}\left(\frac{\Delta_{oh}}{L}\right)} = \frac{1}{1 - \frac{24{,}000\ kips}{800\ kips}(0.0025)} = 1.08$$

M_{ux} = 1.08 x 300 kip-ft. = 324 kip-ft.
Because $L_b < L_p$ = 14.2 ft., $M_{nx} = M_{px} = Z_x F_y$; $\phi_b M_{nx}$ =1,200 as in Example H-3.
By Interaction Equation H1-1a:

$$\frac{1{,}400\ kips}{1{,}940\ kips} + \frac{8}{9}\frac{324\ kip\text{-}ft.}{1{,}200\ kip\text{-}ft.} = 0.72 + \frac{8}{9}0.27 = 0.96 < 1.0$$

W14x176 is **o.k.**

the beams can be mobilized to provide the required torsional reactions along the girder.

4.Closed sections provide considerably better resistance to torsion than open sections; torsional rotations and stresses are much lower for box beams than for wide-flange members. For members subjected to torsion, it may be advisable to use box sections or to simulate a box shape by welding one or two side plates to a W shape.

I. Composite Members

Chapter I of the LRFD Specification covers composite members. Included are concrete-encased and concrete-filled steel columns and beam columns, as well as steel beams interactive with the concrete slabs they support and steel beams encased in concrete. Unlike traditional structural steel design, which considers only the strength of the steel, composite design assumes that the steel and concrete work together in resisting loads. This results in more economical designs, as the quantity of steel can be reduced.

Compression Members

Composite columns (concrete-encased and concrete-filled) must satisfy the limitations in Section I2 of the LRFD Specification. The design strength of axially loaded composite columns is $\phi_c P_n$, where ϕ_c = 0.85 and the nominal axial compressive strength is determined from Equations E2-1 through E2-4 with the following modifications:

A_s replaces A_g, r_m replaces r, F_{my} replaces F_y, and E_m replaces E.

$$F_{my} = F_y + c_1 F_{yr}\frac{A_r}{A_s} + c_2 f'_c \frac{A_c}{A_s} \qquad \text{(I2-1)}$$

$$E_m = E + c_3 E_c \frac{A_c}{A_s} \qquad \text{(I2-2)}$$

r_m = radius of gyration of the steel shape, pipe or tubing, in. (For steel shapes it shall not be less than 0.3 times the overall thickness of the composite cross section in the plane of buckling.)

where

$$E_c = w^{1.5}\sqrt{f'_c}$$

and

F_{my} = modified yield stress for the design of composite columns, ksi
F_y = specified min. yield stress of the structural shape, ksi
F_{yr} = specified min. yield stress of the longitudinal reinforcing bars, ksi
f'_c = specified compressive strength of the concrete, ksi
E_m = modified modulus of elasticity for the design of composite columns, ksi
E = modulus of elasticity of steel = 29,000 ksi
E_c = modulus of elasticity of concrete, ksi
w = unit weight of concrete, lb/ft^3
A_c = cross–sectional area of concrete, in.^{2}
A_r = cross–sectional area of reinforcing bars, in.^{2}
A_s = cross–sectional area of structural steel, in.^{2}
c_1, c_2, c_3 = numerical coefficients.

For concrete filled pipe and tubing: c_1 = 1.0, c_2 = 0.85, c_3 = 0.4

For concrete–encased shapes: c_1 = 0.7, c_2 = 0.6, c_3 = 0.2

Composite columns can be designed by using the Composite Columns Tables in Part 5 of this LRFD Manual (or the numerous tables in AISC Steel Design Guide No. 6: *Load and Resistance Factor Design of W-Shapes Encased in Concrete*) for the cross sections tabulated therein, or the above equations for all cross sections.

Flexural Members

The most common case of a composite flexural member is a steel beam interacting with a concrete slab by means of stud or channel shear connectors. The slab can be a solid reinforced concrete slab, but is usually concrete on a corrugated metal deck.

The effective width of concrete slab acting compositely with a steel beam is determined by three criteria. On either side of the beam centerline, the effective width of concrete slab cannot exceed:

a. one-eighth of the beam span,
b. one-half the distance to the centerline of the adjacent beam, or
c. the distance to the edge of the slab.

The following pertains to rolled W shapes in regions of positive moment, the predominant use of composite beam design. Other cases (e.g., plate girders and negative moments) are covered in Chapter I of the LRFD Specification.

The horizontal shear force between the steel beam and concrete slab, to be transferred by the shear connectors between the points of zero and maximum positive moments, is the minimum of:

a. $0.85f'_cA_c$
(the maximum possible compressive force in concrete)
b. A_sF_y
(the maximum possible tensile force in the steel)
c. ΣQ_n
(the strength of the shear connectors)

For W shapes, the design flexural strength ϕ_bM_n, with $\phi_b = 0.85$, is based on:

a. a uniform compressive stress of $0.85f'_c$ and zero tensile strength in the conctete
b. a uniform steel stress of F_y in the tension area and compression area (if any) of the steel section, and
c. equilibrium; i.e., the sum of the tensile forces equals the sum of the compressive forces.

The above is valid for shored and unshored construction. However, in the latter case, it is also necessary to check the bare steel beam for adequacy to support the wet concrete and other construction loads (properly factored).

The number of shear connectors required between a point of maximum moment and the nearest location of zero moment is

$$n = \frac{V_h}{Q_n} \qquad \text{(2-26)}$$

where

V_h = the total horizontal shear force to be transferred, kips
= the minimum of $0.85f'_cA_c$, A_sF_y, and ΣQ_n
Q_n = the shear strength of one connector

The nominal strength of a single stud shear connector in a solid concrete slab is

$$Q_n = 0.5A_{sc}\sqrt{f'_cE_c} \le A_{sc}F_u \qquad \text{(I5-1)}$$

where

A_{sc} = cross-sectional area of a stud shear connector, in.^{2}
f'_c = specified compressive strength of concrete, ksi
F_u = minimum specified tensile strength of a stud shear connector, ksi
E_c = modulus of elasticity of concrete, ksi

Special provisions for shear connectors embedded in concrete on formed steel deck are given in Section I3.5 of the LRFD Specification. Among them are reduction factors (given by Equation I3-1 and I3-2) to be applied to the middle term of Equation I5-1 above.

The design of composite beams and the selection of shear connectors can be accomplished with the tables in Part 5 of this LRFD Manual.

The design shear strength for composite beams is determined by the shear strength of the steel web, as for noncomposite beams; see Section F above.

Combined Compression and Flexure

Composite beam-columns are covered in Section I4 of the LRFD Specification.

One of the newest and most exciting products to hit the market is the **Castelite** line of steel shapes from ***Chaparral Steel Company***. Castelite is a line of castellated beams that are scheduled to begin shipping this September. The beams behave in a manner similar to Vierendeel trusses, with stresses and deflections that exceed those calculated using simple bending theory. The primary benefit is the increased section modulus and moment of inertia, which are realized with no increase in the weight of the beam. The loss of web area has little effect on bending strength, as the beam flanges carry most of the bending load. The design of the Castelite beams does, however take into account the secondary bending stresses generated by the shear force transfer across the tee at a castellation.

For a brochure fully describing the new product line, including load designations and details, contact: Castelite Steel Products, 300 Ward Road, Midlothian, TX 76065-9651; 800/779-1211; fax: 214/779-1236 or **CIRCLE #43**.

Seismic design has been a hot topic since the steel problems were discovered after the Northridge Earthquake, and two companies have introduced systems to help designers. **MNH-SMRF Systems** is marketing a proprietary column-to-beam connection system for new and retrofit construction. The system is accomplished without direct attachment between beam flanges and the column flange and it utilizes fillet welds to complete the connection. As part of the system, MNH-SMRF provides: details of the connection assemblies; design guides and system computer software; technical and engineering information, drawings, data, and construction and operating information; general review of structural drawings, specifications, data, calculations and shop detail drawings; and coordination of testing as may be required to obtain agency acceptance and/or approval of elements or details.

An alternate approach is provided by ***Tekton*** with its **Seismic Brake**. The product is a maintenance-free, passive, dry friction damper. It can be custom fabricated for loads of a few hundred pounds to loads in excess of 890 kips. It converts energy into heat when a structure is set in motion by earthquake or wind-induced horizontal loads force a metal rod in the device back and forth between spring loaded gripping blocks. The devices can be used in both new and retrofit projects. When the Seismic Brake is incorporated into an SMRF, the bay will become an energy dissipating braced frame and the structure will be able to survive an earthquake without any permanent deformation.

For more information on the MNH-SMRF Systems, contact MNH-SMRF at 3151 Airway Ave., Suite N-1, Costa Mesa, CA 92626; 800/475-2077; fax: 714/540-0319; or **CIRCLE #114**. For more information on the Seismic Brake, contact: Tekton, 2985 W. Whitton Ave., Phoenix, AZ 85017; 602/254-8661; fax: 602/254-6621; or **CIRCLE #25**.

While OSHA's new fall protection standard does not directly apply to the erection of steel buildings, they have drawn renewed attention to the subject. One of the leader's in the field, ***Miller Equipment***, has recently introduced the **Veralite #650 Series Harnesses**. The harnesses meet all applicable OSHA and ANSI standards while allowing unrestricted movement while climbing and working. The harness comes in four models, allowing workers to customize the harness to their specific needs. Accessories include cooling backpacks and shoulder pads as well as a saddle-seat with D rings.

The photo on the cover of the May 1995 issue of *Modern Steel Construction* resulted in more than 30 phone calls from people wanting information on the safety device so prominently shown. For those who don't already know, the picture showed the **Sinco Beam Safe system**, which provides an easy-to-install tie-off point for ironworkers and other construction personnel who must work at heights before flooring or other protection is available. The system includes two stanchions, a steel safety cable, and an energy absorber on the cable. The stanchions weigh less than 35 lbs. each and are available for use on 6-in.-wide or greater beams. Also, they mount at a 19 degree angle from the base so they are out of the way of workers.

Another fall safety product is the **Gemtor energy absorber lanyard**. The lanyard is designed to be used in conjunction with a body belt or full body harness and reduces the maximum arresting force to approximately 650 lbs. The energy absorbers meet OSHA and ANSI standards and come in a choice of models, with locking snap hooks on both ends, D ring and locking snap hook, or a locking snap hook and loop configuration for

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Franklin, PA 16323-0271; 814/432-2118; fax: 814/432-2415 or **CIRCLE #29**. For more information on SINCO safety systems, contact: The Sinco Group, One SINCO Place, P.O. Box 361, East Hampton, CT 06424; 800/243-6753; fax: 203/267-5515 or **CIRCLE #62**. For more information on the Gemtor lanyard, contact: Gemtor, Inc., 1 Johnson Ave., Matawan, NJ 07747; 800/405-9048 or **CIRCLE #80**.

The fastest changing segment of the design and construction industry is undoubtedly the computer field. One extremely useful recently introduced product is **PowerPLOT Lite** from ***20/20 Software***. The program, designed to work specifically with AutoCAD, offers: plot file generation and spooling; batch plotting; accurate plots; and printer/plotter support.

Speaking of printing, several new output devices are on the market. ***Mutoh America*** has introduced the **Accujet Series** of large format inkjet plotters. The plotters offer 300 dpi resolution in both monochrome and color models. Prices range from $2,995 to $4,495. In addition, the company has introduced a new series of pencil/pen plotters.

The **SummaJet 2 Series** Large-Format Ink Jet Plotters from ***Summagraphics*** have been reduced in price and now range from $2,299 for a D-size monochrome plotter to $3,199 for an E-size color plotter.

On the input side, ***ANAtech*** has introduced a new large-scale scanner, the **Eagle SLI 3840**, with a maximum document width of 38-in. and a maximum base resolution of 400 dpi.

For information on PowerPlot, contact: 20/20 Software, 322 East Broad St., Falls Church, VA 22046; 703/534-3400; fax: 703/534-0100 or **CIRCLE #104**. For information on the Accujet Series, contact: Mutoh America, 500 West Algonquin Road, Mt. Prospect, IL 60056; 708/952-8880; fax: 708/952-8808 or **CIRCLE #105**. For information on the SummaJet 2 Series, contact: Summagraphics Corp., 8500 Cameron Road, Austin, TX 78754-3999; 512/835-0900; fax: 512/835-1916 or **CIRCLE #106**. For information on the Eagle SLI 3840, contact:

ANAtech, 10499 Bradford Road, Littleton, CO 80127; 800/533-0165; fax: 303/973-7092 or **CIRCLE #107**.

New releases are available fora number of structural engineering software products. RAM-STEEL from ***Ram Analysis*** has introduced V4.1, which now includes a DXF Import command. This powerful feature allows the designer to import .dxf format CADdrawings into the Floor Framing Program. Drawings originating with the project architect, client or created in-house can now be directly imported for analysis, design and drafting.

STAAD-III, perhaps the most used general purpose structural software for analysis, design and drafting, is now available in a Windows version. It offers: pulldown menus; multiple viewports; multiple sessions; non-preemptive multitasking; intra- and inter-software interrupts, custom icons and controls, copy and paste, object linking and embedding and dynamic data exchange compliance.

Another new Windows introduction is from ***Numera***, which recently shipped v1.2 of VisualCADD for Windows. The new release offers completely open architecture, improved performance, and customization enhancements.

Still another new Windows introduction is v1.0 of Analysis-Group from ***Integrated Engineering Software***. The software performs structural analysis for six common problems: rectangular plates; continuous beams; beams on elastic foundations; rectangular mat footings; circular tanks; and shear wall systems.

Metrosoft has recently introduced v3.0 of Robot V6, a fully integrated graphical structural analysis and design program. Emphasis in v3.0 was on including more productivity features, including: intelligent proactive prompting system that suggests the next action by displaying text at the current mouse position; and implementation of DLL technology, which allows for instantaneous switching between cursor location and message field.Other enhancements include the addition of Kan's meshing method for finite elements and support for SSDNF (Structural

Steel Detailing Neutral File) for direct interface with steel detailing programs.

Das Consulting has released **DAST-EZ-Analysis**, a simple-to-use 2D/3D Finite Element Static, P-Delta and Dynamic Analysis software program. It comes with a parametric library, which offers simple icon-based model generation; a graphical pre-processor; built-in input editor with context sensitive help; output browser; and a graphical post-processor. Users may view and print structures, deformations and contours. The $439 program can handle up to 200 joints per 2D/3D model.

While not software per se, ***Algor***'s new 24-minute video, "**Finite Element Analysis In Action**," is clearly a related product. The video is designed to help engineers improve their skills in performing computer-aided finite element analysis (FEA) and includes: a lesson in brittle materials engineering; laboratory testing of a part under high load stress; finite element analysis of a part under high load stress; finite element analysis of the part under simulated laboratory loading; a comparison of the two results; and an additional, more complex, example comparing the two results to demonstrate techniques in finite element modeling including load applications, boundary conditions, the "staged model" concept and engineering judgement.

For information on RAMSTEEL, contact: Ram Analysis, 5315 Avenida Encinas, Suite 220, Carlsbad, CA 92008; 619/431-3610; fax: 619/431-5214 or **CIRCLE #41**. For information on STAAD-III, contact: Research Engineers, 22700 Savi Ranch, Yorba Linda, CA 92687; 714/974-2500; fax: 714/974-4471 or **CIRCLE #34**. For information on VisualCADD, contact: Numera, 1501 Fourth Ave., Suite 2880, Seattle, WA 98101; 206/622-2233; fax: 206/622-5382 or **CIRCLE #112**. For information on AnalysisGroup, contact: Integrated Engineering Software, 8840 Chapman Road, Bozeman, MT 59715; 406/586-8988; fax: 406/586-9151 or **CIRCLE #87**. For information on Robot V6, contact: Metrosoft, 332 Paterson Ave., East Rutherford, NJ 07073; 201/438-4915; fax: 201/438-7058 or **CIRCLE #51**. For information on DAST-EZ, contact: DAS Consulting, 865 Turnpike St., Norht Andover, MA 01845; 800/322-1487; fax: 508/685-7230 or **CIRCLE #82**. For information on the FEA video, contact: Algor, 150 Beta Dr., Pittsburgh, PA 15238-2932; 412/967-2700; fax: 412/967-2781 or **CIRCLE #113**.

Detailers and fabricators also have a number of new software products available. A program recently introduced to the U.S. market is **SteelModeler** from ***dcCAD***. The program produces a 3D model of the structure, including standard steel sections, plates, bolts, and welds, as well as any tailor-made or special shapes. Outputs include drawings, lists and NC files.

The **Precision Steel Database** from ***Timberline*** provides fabricators with all the needed details—from drilling holes to the total weight of steel on a project—to start estimating right away. Material and labor estimates take into account: types of plates; clips; angles; and even drilling sets of holes.

A free demonstration diskette is now available from ***Steel Solutions***. The disk demonstrates the **STEEL 2000** system. Modules include estimating, mill order, inventory/purchasing, fabrication, drawing control, service center, plate burning/shearing, CNC programming & multing, and accounting.

CompuSTEEL from the ***Baresel Corp.***, a powerful steel detailing program, now employs ARRIS graphics, a full-featured, 32-bit professional application software system that runs in Windows.

E.J.E. Industries has introduced v5.0 of its **Structural Material Manager** The series of programs produces: job estimates; bills of material; length-cutting lists; and shipping lists. New is a plate-nesting report. Single-user systems start at $995.

A new inventory control system from ***P2 Programs*** utilizes a bar coding system to identify each piece of steel with a unique serial number. The **Steel Tracking System** also offers custom interfaces for AutoCAD, SteelCAD, Dbase, Design Data, E.J.E. Industries, and other major applications.

For information on SteelModeler, contact: dcCAD, 605 Royal York Road, Suite 201, Etobicoke, Ontario, M8Y 4G5 CANADA; 800/776-4840; fax: 416/253-4315 or **CIRCLE #107**. For information on the Precision Steel Database, contact: Timberline Software, 9600 SW Nimbus Ave., Beaverton, OR 97008-7163; 800/628-6583; fax: 503/526-8049 or **CIRCLE 123**. For information on STEEL 2000, contact: Steel Solutions, P.O. Box 1128, Jackson, MS 39215-1128; 601/932-2760; fax: 601/939-9359 or **CIRCLE #98**. For information on CompuSTEEL, contact: Barasel Corp., Bank of America Tower, 300 South Harbor Blvd., Suite 500, Anaheim, CA 92805; 714/776-3200; fax: 714/776-1255 or **CIRCLE #131**. For information on Structural Material Manager, contact: E.J.E. Industries, 287 Dewey Ave., Washington, PA 15301; 800/321-3955; fax: 412/228-7668 or **CIRCLE #46**. For information on the Steel Tracking System, contact: P2 Programs, P.O. Box 1000, Dripping Springs, TX 78620-1000; 800/563-6737 or **CIRCLE #126**.

Two very useful new tools are now available for both engineers and fabricators. **Jobber 4** from ***Jobber Instruments*** is a dimensional calculator. In addition to working in feet, inches, sixteenths, decimals and metric lengths, the calculator offers scientific functions and works directly with degrees-minutes-seconds. And finally, the program now solves both triangles and circles. The calculator is currently priced at $99.95 plus $5.50 s&h.

Another calculator is the **Inchmate+** from ***Digitool***. The foot/inch/fraction calculator automatically solves for rise, run, diagonal or slope of any right triangle.The calculator is currently priced at $49.95.

For more information on Jobber 4, contact: Jobber Instruments, P.O. Box 4112, Sevierville, TN 37864; 800/635-1339 or **CIRCLE #23**. For information on Inchmate+, contact: Digitool, 414 North Mill St., Aspen, CO 81611; 800/543-8930 or **CIRCLE #129**.

Another useful "tool" for fabricators and engineers is the pocket-sized **"Structural Bolting Handbook"** from the ***Steel Structures Technology Center***. The guide includes step-by-step instructions for: bolt installation using the turn-of-the-nut, calibrated wrench, twist-off bolt and direct tension indicator methods; pre-installation tests and inspection procedures for each method; dispute arbitration procedures; AASTHO/FHWA rotational capacity test procedures; bolt, nut and washer product and manufacturer identification markings; bolt, nut and washer dimensions; compatibility tables; and discussion of many critical bolting issues. Single copies are $9; 11-50 copies are $8 each; and 51+ copies cost $7 each.

For more information, contact: Steel Structures Technology Center, 40612 Village Oaks Dr., Novi MI 48375-4462; 810/344-2910; fax: 810/344-2911 or **CIRCLE #21**.

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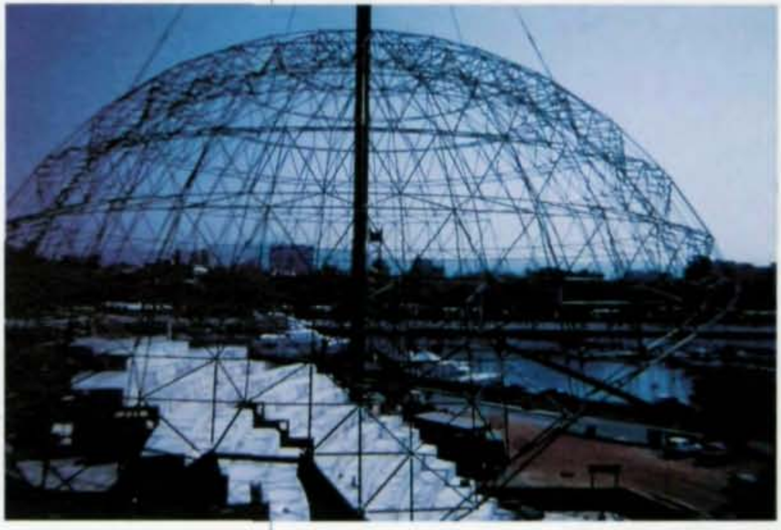

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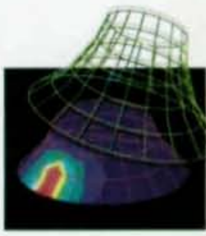

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