MODERN STEEL CONSTRUCTION

May 1992

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No. 16

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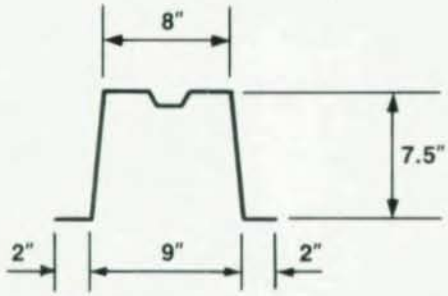
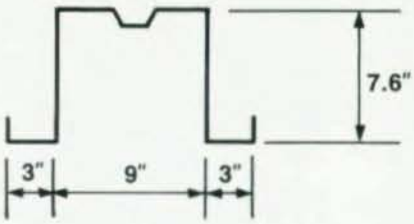
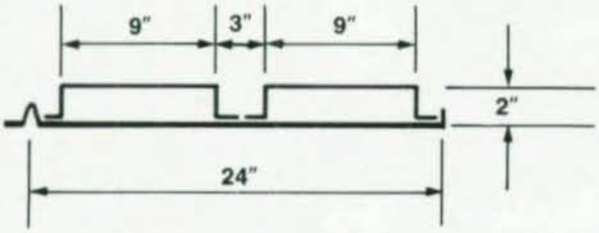


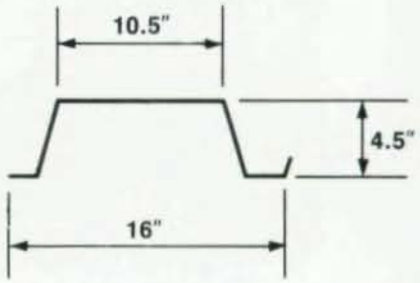
16 GAGE FORM DECK — USED TO BUILD A PIER WITH AN 18" SLAB.



12 GAGE 28′ LONG DECK WAS USED TO ROOF AN EXISTING TANK. 15" SLAB; 6" WIDE RIBS USED AS REINFORCED CONCRETE JOISTS AT 15" CENTERS.



12 GAGE TOP AND 16 GAGE BOTTOM CELLULAR DECK USED TO SPAN BETWEEN BOTTOM BEAM FLANGES IN A POWERPLANT. VERY THICK SLAB — FLAT UNDERSIDE LEFT EXPOSED.



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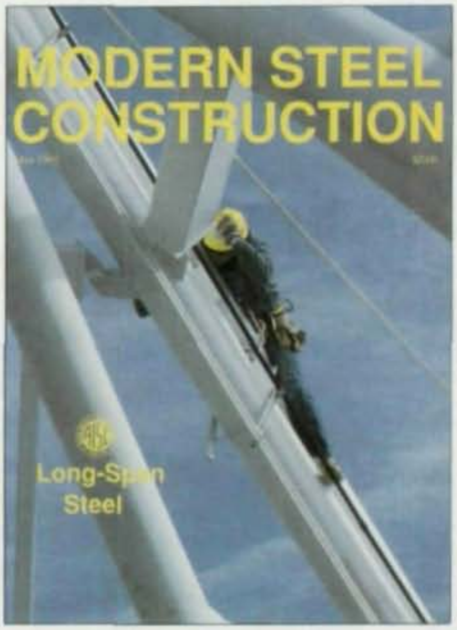
MODERN STEEL CONSTRUCTION

Volume 32, Number 5 May 1992



To simplify erection on the Rain Forest Pyramid on Galveston Island in Texas, all field connections were designed with slip critical bolts in oversized or slotted holes. To find out more about this fascinating project, turn to page 34.

FEATURES

18 AIRPORT OVERHANG
Providing the needed column-free space for an airplane maintenance facility required cantilevering long-span steel trusses 282'

29 LONG-SPAN PERFORMANCE
Steel trusses clear-spanning 287' create the wide-open space needed for a combined basketball arena/performance center

34 STEEL PYRAMID
A pyramid proved the ideal structure for a rain forest exhibit

NEWS AND DEPARTMENTS

6 EDITORIAL

9 STEEL INTERCHANGE
- When To Use Doubler Plates
- Analyzing Steel Beams And Members To Aid Erection

12 SOFTWARE REVIEWS
- Engineering Software Company
- Graphic Magic

16 STEEL CALENDAR

16 PEOPLE IN THE NEWS

17 CORRESPONDENCE

33 NATIONAL STEEL CONSTRUCTION CONFERENCE CALL FOR PAPERS—1993

40 FIRE-PROTECTION PRODUCTS

42 AD INDEX

Modern Steel Construction (Volume 32, Number 5). ISSN 0002-8445. Published monthly by the American Institute of Steel Construction, Inc. (AISC), One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Advertising office: Pattis/3M, 7161 North Cicero, Lincolnwood, IL 60646.

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

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Making Better *Designers*

A friend of mine works in the marketing department of a large financial firm in New York. I asked him if the recession was cutting into his marketing budget and he said no, just the opposite. And then he proceeded to lecture me—despite my protestations that he was preaching to the converted—about how smart companies increase their marketing and advertising in tough times to take market share from their competitors and to better position themselves for the imminent recovery.

The advertising community has been singing their song about increasing ad budgets during slow periods for more than 20 years and savvy firms are starting to listen.

But for design firms, there's a corollary tune that's at least as important. No matter how good your marketing is, you still need a good product to sell. And the only way a practicing designer will substantially improve his "product" is through continuing education—a budget area that too many firms are trimming.

In February we printed one of the best received issues in *MSC's* history—our "Special Report On How Designers Can Cut Fabrication Costs" (if you missed that issue, 16-page reprints are available free to AISC Members and for $2 per copy to non-members; for more information call 312/670-5421). And while the information in that issue may have been news to many practicing engineers, it was old hat for the thousands who attended the National Steel Construction Conference during the past few years.

Next month we'll publish copies of selected papers from this year's conference. But given our space limitations, we can barely breach the surface of available information. And, of course, there's no substitute to attending a presentation and then taking advantage of the opportunity to discuss the subject with your peers.

There's a lot of firms out there that are producing good designs today. But unless they also concentrate on producing ***good designers***, there's no guarantee they'll still be producing good designs tomorrow.

To receive more information on the National Steel Construction Conference, call (312) 670-5422. I hope to see all of you in Las Vegas next month. **SM**

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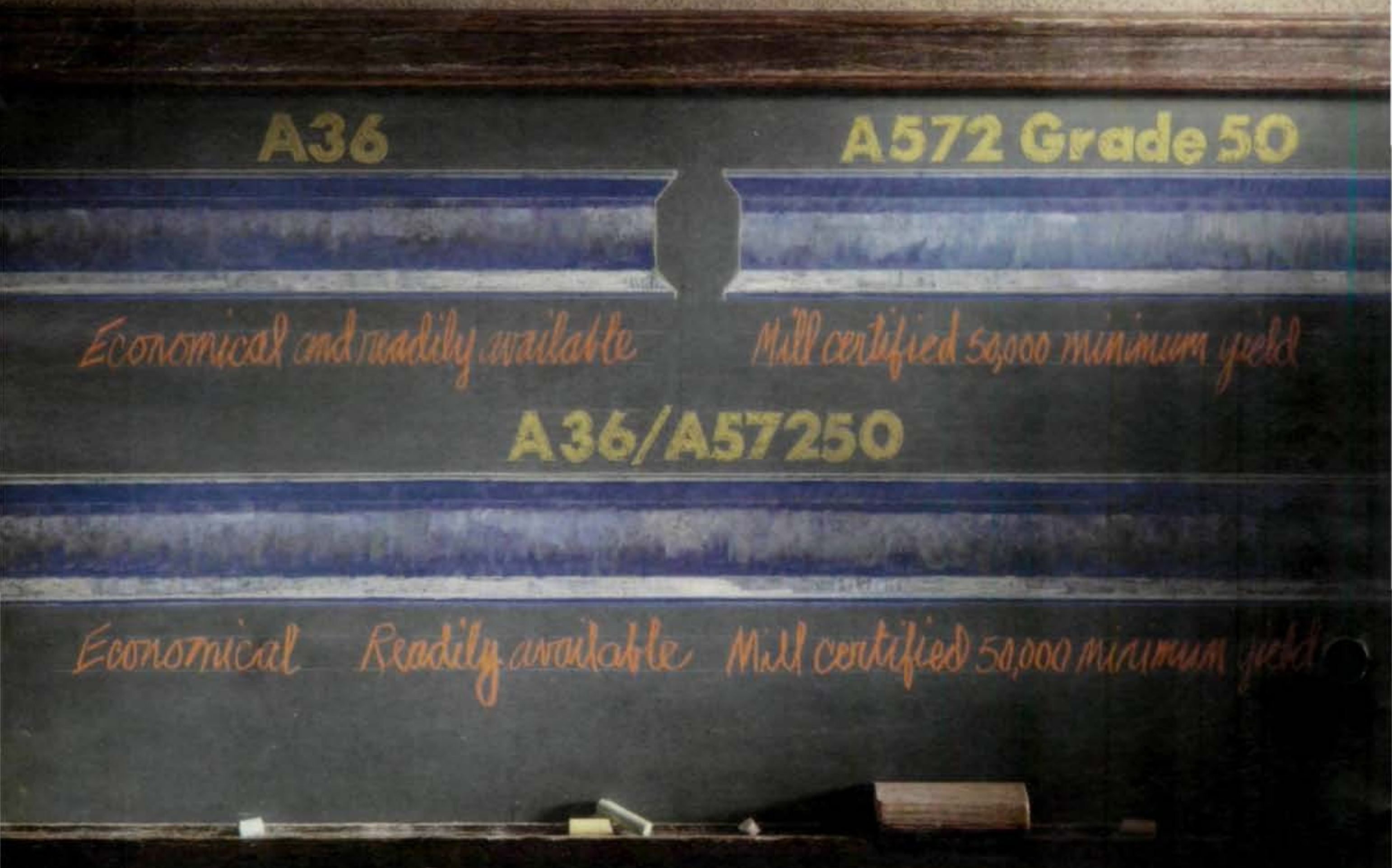

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Steel Interchange

Steel Interchange is an open forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine. If you have a question or problem that your fellow readers might help 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
1 East Wacker Dr.
Suite 3100
Chicago, IL 60601

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. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

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

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

How do you decide when to use doubler plates and when to increase the size of a column?

Unfortunately, many engineers release structural drawings without analysis and include a statement to provide a doubler plate "if needed". It is too late, at this stage, to pick a column with a thicker web. Most fabricators would be more than happy to help the designer with this decision during the design stage. Designers should ask for this advise.

Robert O. Disque, P.E.
Besier Gibble Norden
Old Saybrook, CT

The design of steel structures is clearly discussed in the AISC Manuals and publications; however, how does one design and analyze the steel beams and members used to aid in erection?

The design of steel members used in the hoisting process during construction is discussed in the *Engineering Journal* article by Dave Ricker titled, *Design and Construction of Lifting Beams*. This article appeared in the fourth quarter 1991 issue starting on page 149 and will be summarized here.

Lifting beams which are used during erection can also be called spreader beams. Some examples of lifting beams are shown in the figures printed above.

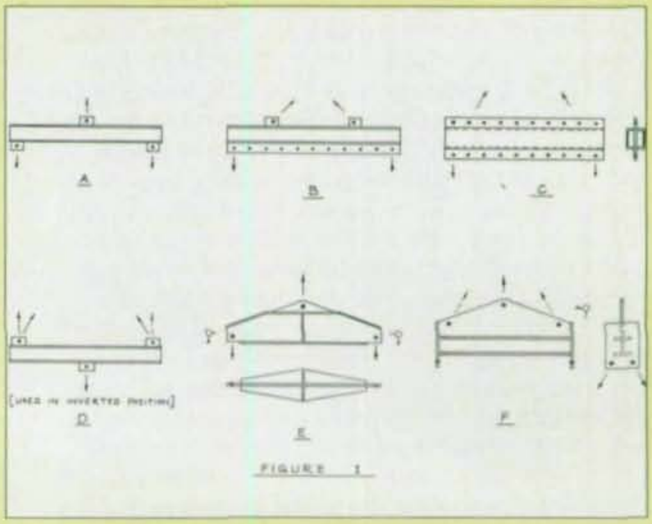


Figure A is the basic lifting beam and provides two places of attachment to the object being lifted, this avoids the possibility of overstressing if a single attachment were used. This also allows for a straight pull on the object rather than an oblique pull as would result if chokers alone were used. It is sometimes important to minimize unwanted erection stresses or to prevent reversal of stress in certain portions of the lifted object. For instance an oblique pull may cause excess compression in the top chord of the truss at a time when that chord is laterally unbraced. A lifting beam can be used as a "strong back" to provide multiple lifting points on a relatively flexible object.

Other elements commonly associated with lifting beams are hooks, shackles, chokers, and slings. Shackles are used to connect the lines to the lifting beams. Shackles come in various patterns and capacities. Chokers are often used to wrap around the object to be lifted and are usually fastened to the underside of the lifting beam by means of shackles. Slings are used to suspend the lifting beam from the main hook. Hooks are often used with shackles or oblong rings. Hooks with safety latches which prevent the shackles or lines from escaping the throat of the hook are recommended.

A lug plate with its pin hole is an important component of the lifting beam assembly. Tests have indicated that the ratio of pin diameter to hole diameter has little influence on the ultimate strength of the lug material. The diameter of the hole in the lifting lug should be at least 1/16 in greater than the largest pin (or bolt) diameter which is anticipated. However, it is not necessary to have the pin fit snugly in the hole. In fact, the pins are apt to be rough cast and not perfectly round. More often than not, the pin may be considerably smaller than the hole. Pins as small as one half the hole diameter are not rare.

If the lug plate proves to be deficient in bearing strength, washer plates can be welded around the hole to increase the thickness. If the pin is less than a snug fit the lug plate must be designed to prevent tearing failure at the plate edge. Tear-out results when the pin attempts to plow through the plate edge, often resulting in a bulge whose outer edge is in severe local tension. The dimension must be adequate to prevent tear-out but small enough so it will accommodate the shackle length.

In order to prevent the line or shackle from fouling the square corner of the lug plate, the corner may be cut on a diagonal or it may be rounded.

Since lifting beams are often used and stored outdoors, it is recommended that welds be made water tight to prevent hidden corrosion. The weld should be sized to account for any eccentricies which may result from various angles of pull. The thickness of the lug should be such that it will accommodate the "jaw" opening of the shackle. It is not necessary that the lug plate fit snugly in the jaw opening. However, a gross mismatch may cause the lifting beam to hang slightly out-of-vertical which may result in undesirable torsion stresses. A suggested rule of thumb is lug thickness should be no less than one-half the jaw opening. Bottom lugs are treated much the same as top lugs except that they are apt to be continuous plates in order to be more versatile.

Since dead weight is sometimes a factor it may be desirable to use high-strength steel. Lifting beam deflection is hardly ever a governing factor. Column sections, that is wide-flange shapes that are approximately as wide as they are deep, are popular for lifting beams because they generally have longer L and Lu lengths. The unsupported length of a lifting beam is the length between the outermost lifting holes.

In establishing the lifting capacity of a lifting beam, several factors must be considered in addition to the static weight of lifting beam, shackles, and lines must be included. In addition, the effects of impact, acceleration, deceleration, wear, deterioration, and abuse must considered. An effective way to account for these unknowns is to apply an additional factor of safety to the static load. If the normal allowable stresses are reduced by a factor of 1.8, the resulting maximum working bending stress in the lifting beam will be about a fifth of the minimum ultimate bending strength of the steel. This is in line with other components of the lifting assembly such as the shackles, lines, and hooks, which are usually load rated for ¼, ⅕, or ⅙ of their ultimate capacity. ANSI/ASME Standard B30.20 requires that lifting beams be designed using a minimum design factor of 3 based on yield strength.

Lifting beams and associated lines and equipment should be inspected before and after use.

After the beam has been assembled and welded, it is usually cleaned and given a coat (or coats) of rust-inhibitive paint. The color should be light in hue and one which contrasts sharply with the primer colors normally used by the fabricator. The lifting capacity of the beam in tons should be clearly stenciled on both sides of the beam in block numbers and letters at least 5 in. in height. If the lifting beam must be used in the upright position only, the top of the beam must be stenciled: USE THIS SIDE UP ONLY. Very often the fabricator or erector will want to stencil the company name on the lifting beam for advertising and identification purposes. Although stenciling in paint is the most common means of marking, a more positive method consists of bead welding the messages onto the beam. Welded figures will endure even if the beam is repainted at a future time.

Equally important as the strength of the lifting beam is the strength of the other components used in conjunction with the beam. Theses are lines, chokers, hooks, and shackles. Only a knowledgeable experienced rigger should be entrusted with the selection of these other items.

New Questions

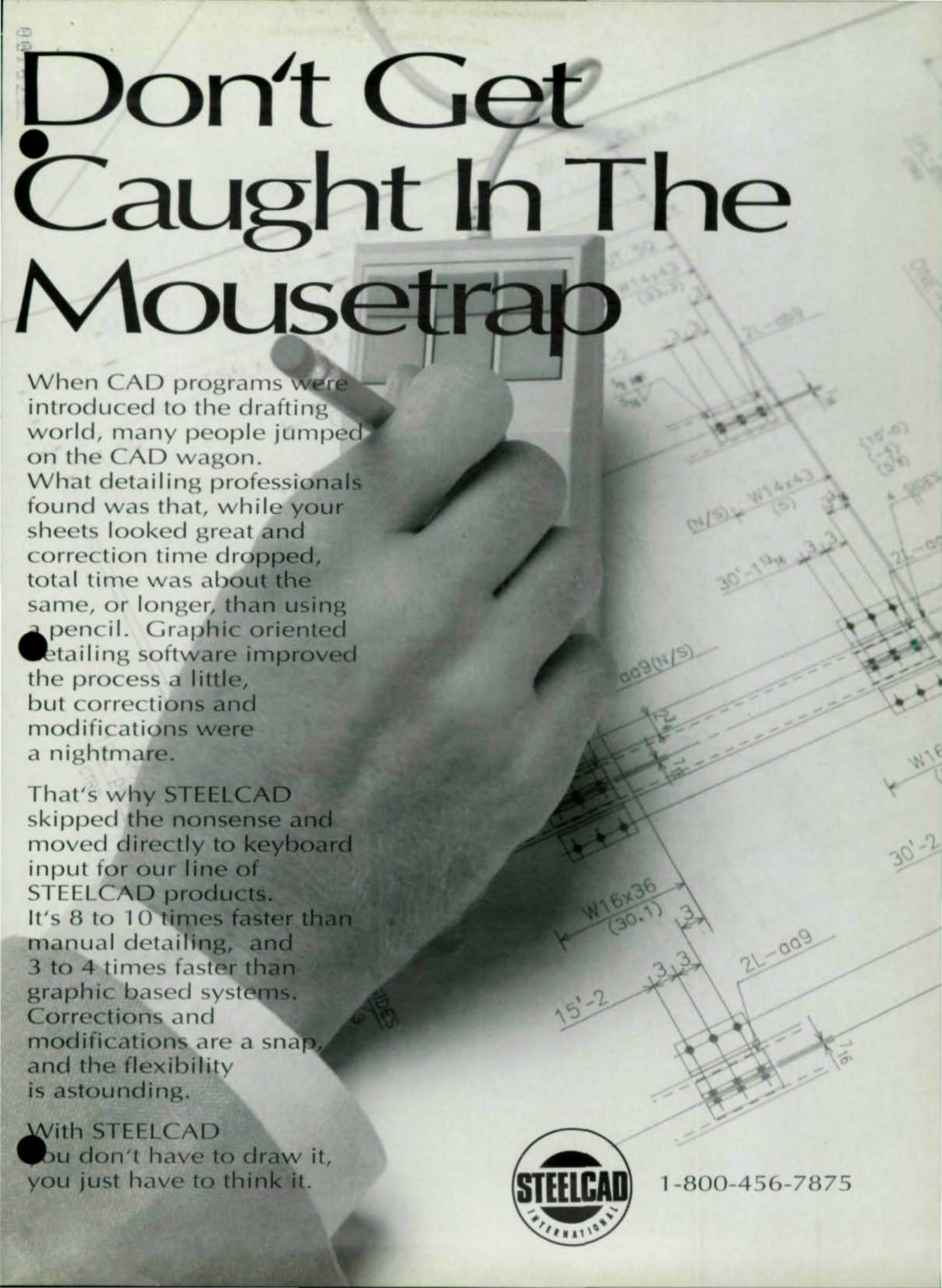
Listed below are some questions that we would like the readers to answer or discuss. If you have an answer or suggestion please send it to the Steel Interchange Editor. Questions and responses will be printed in future editions of Steel Interchange Also if you have a question or problem that readers might help solve, send these to the Steel Interchange Editor.

1. Consider eccentricity and what has to be done to accommodate it in various connections.

2. AISC says end-plates and shear tabs shouldn't be more than 6 or 7 bolts deep and clips shouldn't be thicker than ⅝". Can these rules be relaxed if the connection is only on one side of a header beam?

3. Are there concerns about bending of the tube wall in shear tab type connections? When should the shear plate be carried through the tube section?

4. When would you justify the additional cost of high bond epoxy paint and coating for an exterior steel frame exposed to weather and water? What if there is standing water at the base of the column?


Don't Get Caught In The Mousetrap
When CAD programs were introduced to the drafting world, many people jumped on the CAD wagon. What detailing professionals found was that, while your sheets looked great and correction time dropped, total time was about the same, or longer, than using a pencil. Graphic oriented detailing software improved the process a little, but corrections and modifications were a nightmare.
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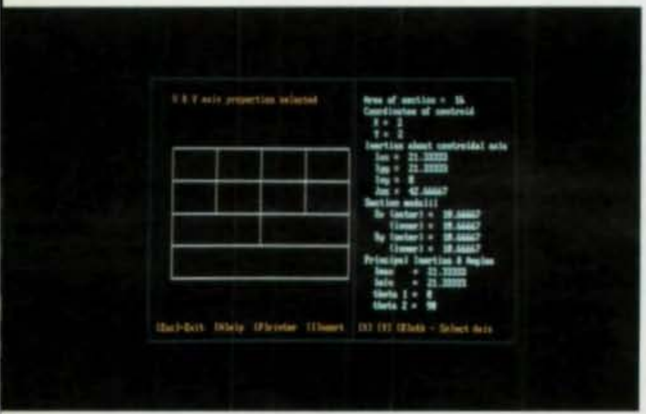
Engineering Software Company's

Advanced Integrated Beam Design Program

By Ahmad Rahimian, Ph.D., P.E.

Advanced Integrated Beam Design by Engineering Software Co. is a special purpose computer program for analysis of continuous beams. The program functions under a spread sheet format, which simplifies problem definition as well as error correction. The ease of inputting variable cross sections as well as irregular loadings is one of the strong features of this program.

The program consists of two modules. The Beam Module performs the analysis of continuous beams, while the Properties Module includes a structural data base as well as features to calculate sectional properties for irregular shapes. The first module can be purchased separately for $495, or both can be purchased together for $995.



Some computer knowledge is required to load the program for the first time. But once it is loaded, inputting data is very simple.

The package is basically an analysis program. However, Safety Factors for each section are reported on the ratio of Smax divided by the calculated stress for that section. Extreme caution should be exercised when using this feature since the value is dependent on more than just the material and an engineer must consider such factors as the design Specification, loading, boundary conditions, and stability.

The program was reviewed in five areas: system requirements; ease of loading the program onto a computer; documentation clarity; performance of stated function; and ease of use for practicing engineers. Each area was ranked from 1 to 5, using the following criteria:

5: This is a great program and I'd like to buy a copy.

4: This is a very effective, very useful program.

3: This program performs its stated goals adequately.

2: The program could be useful but needs to be improved.

1: The program is inadequate.

Because software reviews in *Modern Steel Construction* are written by different engineers at different firms, program ratings cannot be directly compared with previous reviews.

System Requirements

Rating: 5.0

This program runs on a wide variety of IBM compatible computers with minimum hardware requirements and restrictions. These hardware requirements include: PC (8086-based), XT (8088-based), AT (80286-based), PS/2 (80386-based) machines; 512K RAM; one hard disk drive; one floppy disk drive; MS-DOS or PC-DOS versions 2.0 or greater; color graphics adapter; color/composite/EGA/VGA monitor; and an IBM or Epson compatible graphics printer.

Loading

Rating: 2.0

The installation requires the user to be familiar with DOS since a directory has to be created by the user. The files on floppies are compressed and have to be expanded during installation. The installation could become confusing if the user copies all floppies to the hard disk and then tries to install them. What the manual does not clearly specify is that each floppy should be installed separately, one after another.

Documentation

Rating: 3.5

The documentation is divided into the following sections:

Section 1: Preliminaries
Section 2: Introduction
Section 3: Beam Design
Section 4: Beam Example
Section 5: Properties Module

The Preliminaries section covers the license agreement and installation process, which was discussed above. The Introduction explains the program's features, as well as other packages offered by the same company. The latter would be more appropriately included as an appendix or as a separate flyer, rather than as part of the body of the manual.

Sections 3, 4, and 5 are explained adequately using screen printout illustrations. For Section 3, "Beam Analysis" may be a more appropriate title since the program basically obtains the member forces, nodal displacements and reactions. Section 4 includes several examples with Input and Output. Section 5 explains the Properties Module, which by itself has two parts: MOMENTS (Properties Calculator) and QUICK-SHAPE (Materials Library). The Properties Calculator assists in the calculation of sectional properties for nine common geometrical sections as well as user defined irregular sections comprised of rectangular pieces. The Quick Shape submodule contains a data base for AISC sectional properties. An optional data base for wood sections may be obtained from the software supplier.

Program Functioning

Rating: 3.0

The program is very easy to work with because of its spread sheet format. The Properties Module also could be very handy. The Input sheet is very concise, which is a positive point in error corrections and documentation. Also, the program execution is as fast as advertised.

However, there are areas in which the program could be improved:

Units. Program allows the use of either any consistent set of units or the U.S. Customary system. However, the input sheet does not indicate any unit flags. The only way of knowing under what units the system is operating is through a parameter (Fmt) shown at the lower right hand corner of the screen. Ignoring this coded parameter could have serious consequences. The program should flag the units in each appropriate column.

Reactions. Features should be added for spring reactions as well as displaced reactions. These features are not currently included.

Shear deformation. The program should include, or at least give the user the option of including, the shear deformation in the analysis.

Loading sign. The program assumes the positive sign is upward. This means that for the majority of gravity loading cases the user has to input a negative sign in front of the loading value, which requires extra work. Also, partial omission of this sign could be serious. It is better to assume the downward load as positive, which simplifies input and lessens the chance of error. Alternatively, the user could choose to ignore this sign but the moment diagrams will be reversed in respect to common practice.

Loading case. The program allows only one loading case. Features for additional loading case would be helpful.

Design features. The package does not currently offer any design features. A more complete package would integrate design and analysis to reduce the amount of design effort required.

Safety factor. The potential problem with this safety factor was discussed above. Attempts should be made either to clarify the intent better or to incorporate it separately with a design package.

While not a true design package, the program does adequately perform continuous beam analysis

Conclusions

Overall Rating: 3.0

The program performed the goals stated in its manual adequately, even though the title of the program may lead a user to expect a design package. The drawbacks outlined above do not diminish the program's ability to perform the designated task. However, these shortcomings mean that the program is designed for a limited task on a special class of structures: To analyze continuous beams only.

Ahmad Rahimian, Ph.D., P.E., is a vice president with The Office of Irwin G. Cantor, P.C., a nationally respected consulting engineering firm headquartered in New York City.

Graphic Magic's
Multiframe 2D & 3D

By S. B. Haley, P.E., and J. P. Morgan Jr., P.E.

Macintosh-based Multiframe 2D, Multiframe 3D, and Section Maker by Graphic Magic Inc., are object-oriented structural analysis programs that use a graphical user interface to quickly input and analyze structural problems. Multiframe 2D (version 2.01) and Multiframe 3D (version 1.51) analyze two dimensional and three dimensional structures. Both programs use the stiffness matrix method of analysis for solving simultaneous equations to determine forces and deflections of simple or complex structures. The programs perform "a first order, linear elastic analysis to determine the forces and deflections."

Input and output for these programs is handled by moving through a series of "windows" and pointing and clicking a mouse. These programs allow a structural model to be created by drawing the structure and selecting members from standard libraries. The program's standard library includes properties for sections found in the AISC *Manual of Steel Construction*. Custom sections or other materials may also be added to the existing library or to a newly created library.

Each program allows a variety of restraints and loading conditions. Member releases options are rigid/rigid, pin/rigid, rigid/pin, and pin/pin. Choices for support restraints include free, pinned, fixed, X- roller, Y-roller, X-roller fixed, Y-roller fixed, and theta fixed. Spring support restraints are also available as vertical, horizontal, and rotational springs. Load types include global and local joint and member loads. Member loads include point loads, point moments, distributed, partially distributed, trapezoidal, thermal, and selfweight loads. All of these options are applied by choosing graphical icons from the appropriate window and assigning them to the corresponding member or joint.

Section Maker (version 1.51) is a graphical program tool that determines the elastic section properties for members and materials that are not included in its standard library. These sections may be simple with only one shape and material, or may be complex sections composed of multiple shapes and materials. The section properties computed by Section Maker may be stored in a "library" that may be used by the Multiframe programs.

System Requirements
Rating: 5.0

These three programs are written for use on any Apple Macintosh computer system. The Macintosh computer must have a minimum of 1 MB RAM and 1.6 megabytes of hard disk space available. The programs will operate with or without a math co-processor. The copies received for the review came on 3-1/2" disks. The review was conducted using an Apple Macintosh IIx computer with 8 MB RAM, 80 MB hard disk, a math co-processor, and System Software 7.0. Analysis and graphics printouts were made using an Apple Laser Writer II printer with Postscript capabilities.

Loading
Rating: 5.0

Installation instructions provided are clear and concise. Anyone who is familiar with the Macintosh computer will find the software easy to install. Those engineers who are not familiar with the ease of an Apple computer will learn quickly how object-oriented computer structuring can simplify this procedure.

Documentation
Rating: 3.5

Each program comes with an instruction manual in an 8-1/2" x 11" spiral-bound format. The documentation is organized, clear, and professional in appearance. Additional example problems and documentation about the programs theory and the implementation of the theory would make the documentation even better.

The documentation for the Multiframe 2D and Multiframe 3D programs are set up in the similar formats.

"Learning Multiframe2D" (3D) leads the user through a series of exercises that show the basics of Multiframe2D (3D). The exercise is a detailed example for a simple structure. The exercise procedure teaches the user how to create a structure, select members and their properties, apply and combine loads, analyze, view diagrams and tables, and print the results.

"Using Multiframe2D" (3D) elaborates on the basic step by step instructions learned in Chapter 1 by explaining additional instructions, options, and techniques that may be needed to create and analyze a structure.

"Multiframe2D Reference" (3D) lists the organizational structure of the program(s). It graphically shows each of the eight "windows" used by the program and provides a summary for each command within the "windows."

"Multiframe2D Analysis" (3D) describes the numerical method of analysis used by the program(s) to analyze structures. It also lists the capabilities and limitations of the program(s).

The documentation for the Section Maker program is divided into the following chapters: Learning Section Maker; Using Section Maker; Section Maker Reference; and Sections Calculation.

Program Functioning

Rating: 2.5

According to Graphic Magic Inc., all three programs are "a structural analysis and design system for the Apple Macintosh range of computers." It appears that the three programs have analysis capabilities only. The programs give the user the ability to prepare their own design calculations by making use of a "calculation sheet" feature. However, the "calculation sheet" feature has no support for IF statements, looping, or Boolean operations, which limits its usefulness.

A set of example problems were devised to test each program's performance and its ability to meet the program's stated function. The example problems were set up so that each would test most aspects of the program's input, loading, analysis, plotting, and printing capabilities.

A series of multi-story portal frames were used to test Multiframe2D and Multiframe3D. Each test example included a variety of member types, restraints, loading conditions, and load combinations. The test for Section Maker included example problems which were comprised of built-up sections using combinations of standard sections and plates. The results of the Multiframe two and three dimensional sample problems were compared with the results of a well known DOS-based problem solver. The results of the Section Maker sample problems were compared with the results of a HTB proprietary program and calculations done by hand. The comparison demonstrated sufficient accuracy in all three programs.

Conclusions

Rating: 2.5

Initially we devised verification problems for both 2D and 3D modules that were somewhat large compared to the thirty (2D) degrees of freedom and ninety (3D) degrees of freedom of the test problems used. The first thing that came to our attention while creating the 2D test problem was that we could easily produce the basic structure, but the assignment of member properties, end releases, and loadings were cumbersome. The programs allow the user to identify and assign the member properties, end releases, and loadings to the members in four ways: one at a time, sloping members, all horizontal members, or all vertical members. This constraint makes the input or alteration of a large structure unmanageable. We suggest that the random "tagging" of members in a group and the subsequent assignments be allowed to enrich the application of this software.

The user is also restricted by the program's compulsory node numbering. The node and member numbering are something that should be left to the discretion of the engineer for generation purposes and convenience in numbering. The renumbering of the nodes for band-width optimization could be performed internally in each program.

It should be noted that the three software programs do not perform a design or a check of the steel sections. They are strictly analysis programs except for the interactive "calculation sheets" which are ineffective and must be created by the user. This greatly limits their usefulness in most design offices. However, Graphics Magic Inc. has implemented features for the use of third party postprocessors, spreadsheets, and AISC code checking by software developed by DayStar Software of Kansas City, MO.

The speed, agility, and graphical capabilities of Macintosh windows based software are tremendous. The member end conditions, supports, and many other necessary functions are assigned visually as if one was sketching a model of a structure by hand. The ability to see the structure forming on the screen virtually eliminates common errors associated with batch or numerical input software. These errors include, among others, no connectivity of members to nodes, incorrect member end releases, and erroneous support designation.

We believe that this object-oriented user interface is the wave of the future for structural analysis problem solvers and much more practical than the CAD integrated software which has been developed at this time. Any software that reduces the possibility of user error is headed in the right direction.

This software review has been prepared by S. Blake Haley P.E. and J. Patrick Morgan Jr. P.E. Both are structural engineers with HTB, Inc., Oklahoma City, Oklahoma.

We believe that this object-oriented user interface is the wave of the future for structural analysis problem solvers and much more practical than the CAD integrated software which has been developed at this time

Ratings:

5: This is a great program and I'd like to buy a copy.

4: This is a very effective, very useful program.

3: The program performs its stated goals adequately.

2: The program could be useful but needs to be improved.

1: The program is inadequate.

May 5. **Tubular Sections in Building Construction** (co-sponsored by AISC and VCSSFA) breakfast meeting in Raleigh, NC Will include design criteria, Type 2 Connections, tube-to-tube connections, design guides, practical recommendations and application examples.

May 5. **Structural Computer Expo** (sponsored by Structural Engineers Association of Northern California) in San Francisco. Contact: Mark Middlebrook at (510) 547-0602.

May 6. **Tubular Sections in Building Construction** (co-sponsored by AISC and VCSSFA) breakfast meeting in Wilmington, NC (see May 5 listing).

May 7. **Tubular Sections in Building Construction** (co-sponsored by AISC and VCSSFA) breakfast meeting in Charleston, SC (see May 5 listing).

May 11-12. **Central Fabricators Meeting**, Clarion Hotel, St. Louis. General topic: estimating and sales. Contact: LaVerne Duckrow, 7227 W. 127th St., Palos Hills, IL 60463 (708) 361-2338.

May 12-13. **Welding Structural Design** two-day seminar, San Francisco (See January 27-28 listing).

May 18-22. **Structural Steel Design** continuing education course, Madison, WI. Topics include recent changes in specifications and LRFD. Contact: Department of Engineering Professional Development, University of Wisconsin—Madison, 43 North Lake St., Madison, WI 53706 (608) 262-2061.

May 18-21. **AASHTO Subcommittee Meeting on Bridges & Structures**, Portland, ME. Annual meeting of the 50 state bridge engineers who comprise the membership of the AASHTO Subcommittee. Contact: Clellon L. Loveall, Asst. Executive Director, Bureau of Planning and Development, Tenn. DOT, James K. Polk Building, Suite 700, 505 Deaderick St., Nashville, TN 37219 (615) 741-2831.

May 18-21. **International Symposium on Earthquake Disaster Prevention**, Mexico City. Contact: National Center for Disaster Prevention, Av. Delfin Madrigal 665, Pedregal de Sto. Domingo, MEXICO D.F. 04360 (52-5) 606-9942.

June 3-5. **6th Annual National Steel Construction Conference**, Las Vegas. More than 45 seminars and meetings covering every aspect of steel design and construction, including: codes and specifications; computerized design; research; project and shop management; inspection and safety; and fabrication and erection procedures. Trade show features more than 100 exhibitors. Contact: David G. Wiley, AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001 (312) 670-5422.

June 8-11. **Seismic Design of Bridges**, Washington, DC. Course includes seismic design, structural dynamics, seismic loading, design concepts, seismic response analysis, bearings and columns, and retrofitting. Contact: Cliff Hopkins at (202) 994-8521.

June 8-11. **A/E/C Systems computer show**, Dallas. Contact: (800) 451-1196.

People In The News

AISC-member Stupp Bros. Bridge & Iron Co. has donated $1,000 to the AISC Memorial Fund in the memory of Cornelius F. P. Stueck, a former assistant vice president who worked at the firm for 40 years.

Metric Debate

Dear Editor:

On page 14 of the March Issue ["Metric Gains Momentum"], you say that "it is more convenient to express stress in newtons per square millimeter," and that "Moments are expressed in terms of N-mm." This is erroneous. ASTM Standard E 380 is the authoritative guide for metric (SI) expressions. ASTM E 380 does not permit a composite unit (such as mm) in the denominator. The proper expression is "Pa" for stress and "N-m" for moment. If the resulting number is not between 1 and 1000, prefixes (such as mPa, kPa or MPa) must be used.

Sincerely,
Werner H. Gumpertz
Simpson Gumpertz & Heger, Inc.
Arlington, MA

MSC Response:

As stated in the article, because section properties are expressed in millimeters, AISC opts to follow the International Standards Organization (ISO) *Draft International Standard for Steel Design*, as well as the draft of *Eurocode 3, Design of Steel Structures, Part 1—General Rules and Rules for Buildings*, both of which express stress in newtons per square millimeter and moments in Newtons per millimeter. Incidentally, the British Standard for the *Structural Use of Steelwork in Building* also uses N-mm² and N-mm.

Truss Bridges

Dear Editor:

The recent article by Messrs. Olsson and Bandel ("Upgrading Steel Truss Bridges", March 1992) had some interesting ideas, although the lack of clear drawings and the brevity of the article made it difficult to understand some of the proposed details.

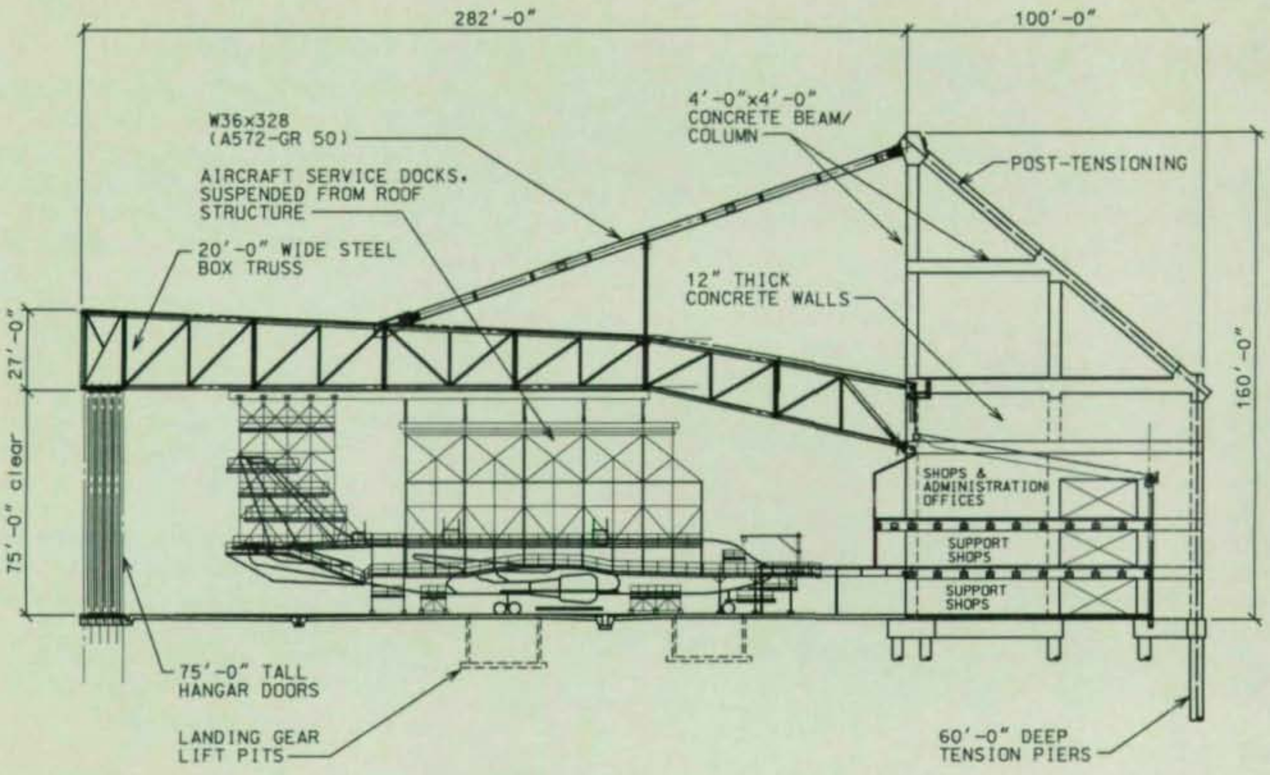
For instance, in the case of using clamped connections to the existing steel, how do the authors propose to control corrosion between the existing steel and the clamp? As shown in Sketch 3 of the article, the top clamp plate bears evenly against the top of the existing angles; however, in reality, it will probably bear against the toe of the angle, as the clamp bolts are tightened. This creates a crevice for dirt and moisture to accumulate and start corrosion. If the sketch is to scale, these corrosive products will be approximately 3" to 4" from the bolt, which will cause large prying forces on the bolts. Whether the bolts will fail or the plate curls will depend on the thickness of the clamping plate and the pitch of the bolts.

Sincerely,
Ralph D. Watts, P.E.
North Island Engineering Ltd.
Comox, B.C., Canada

Nils Olsson responds:

Figure 3 shows how the bottom flange of the new cantilevers are connected to the existing transverse beams. In order to minimize the connection to the existing girder, the compression flange plate at the bottom is only connected at midspan to the existing transverse girder and only for unbalanced live loading.

For all symmetrical loading, the new bottom flange is not connected to the existing girder, but the existing girder is guiding the new flange plate to prevent vertical or horizontal buckling. The clamps, therefore, connect only the side lugs, the rivet head spacer plate, a filler plate and a clamp plate; there is no force exerted from the clamp plate to the existing floor beam angles. It is natural that all elements prior to installation shall be properly painted.



Hangar Typical Cross Section

AMERICAN AIRLINES

Alliance Maintenance Base

L. A. FUESS PARTNERS ENGINEERS
Dallas, Texas

AIRPORT OVERHANG

Providing the needed column-free space for an airplane maintenance facility required cantilevering long-span steel trusses 282'

By Jon C. Herrin, P.E.

While it may never be seen by most passengers, American Airline's latest construction project may be its most important.

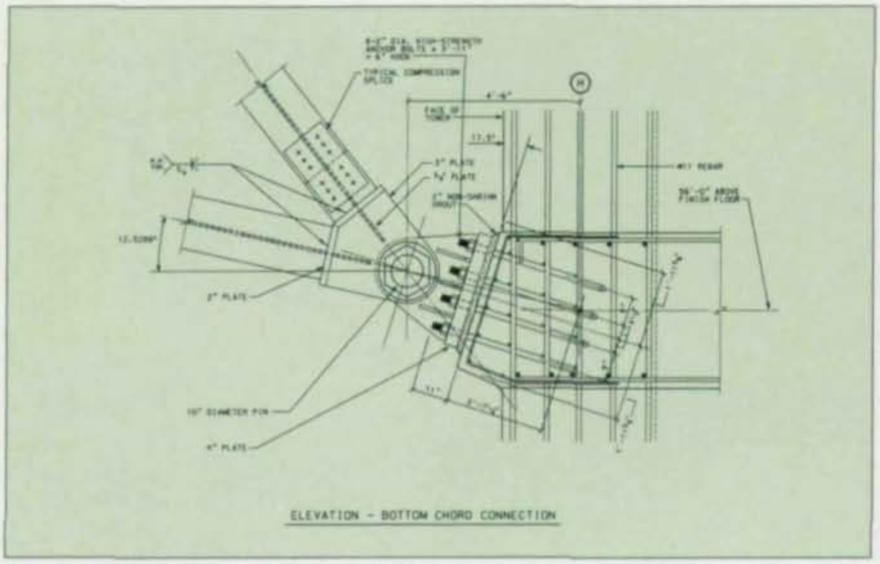
The 780,000-sq.-ft. hangar/shop building is the centerpiece of American's new maintenance and engineering facility at the new Alliance Airport (AFW) in Fort Worth and is crucial in keeping the airline's fleet in the air. American's 175-acre campus consists of the hangar, eight other major buildings, and a variety of smaller structures totaling 1.7 million sq. ft. Total cost for the base is $750 million.

The facility functions as American's heavy "C" check base for its fleet of Boeing 757, 767, and Airbus aircraft. The Federal Aviation Administration (FAA) mandatory heavy "C" check is performed after every 5,000 flight hours, about every 12 to 18 months for commercial transport. The "C" check is basically a complete overhaul of the aircraft, with over 15,000 mechanic man-hours spent on each aircraft. Each aircraft is essentially stripped down to the airframe, with items such as engines, flaps, rudder, landing gear, seats, etc., removed in the hangar and inspected, repaired or rebuilt as required in shops located throughout the base. A normal "C" check takes about 14 days.

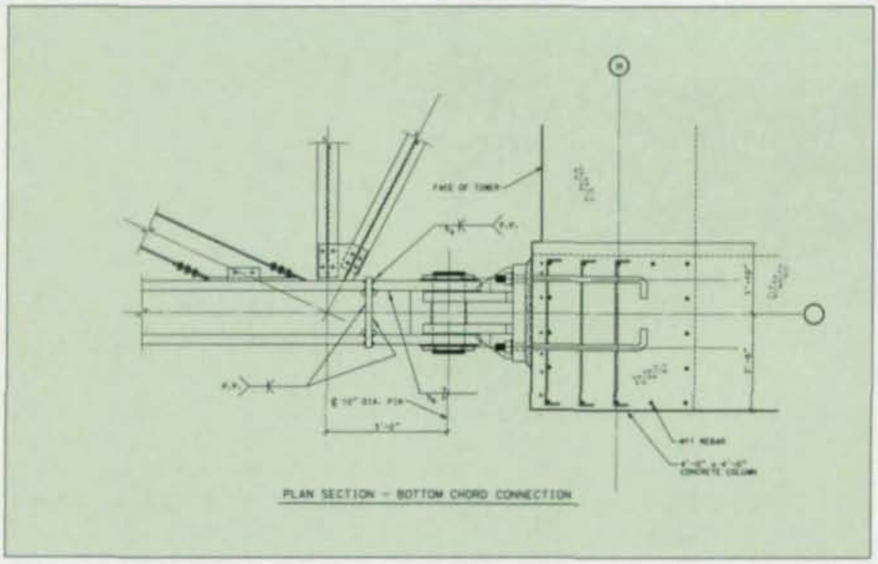
Design Requirements

The hangar was required to accommodate seven wide body aircraft simultaneously, arranged wing tip-to-wing tip, with sufficient working clearance around each aircraft. These requirements dictated a high bay floor plate 1,275' long-by-282' deep, with a clear height inside for the vertical stabilizer and bridge cranes of 75'.

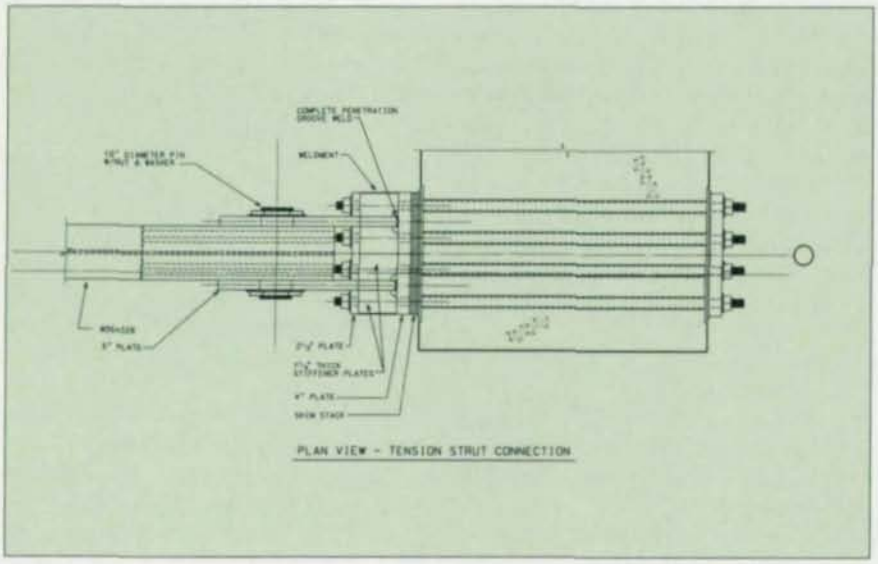
Fifty years ago, the DC-3—with an overall length of 65' and a wingspan of 94'—was the workhorse of the American airline industry. Today, it's the DC-10 and MD-11, each 182' long, 59' tall and with a



ELEVATION - BOTTOM CHORD CONNECTION



PLAN SECTION - BOTTOM CHORD CONNECTION



PLAN VIEW - TENSION STRUT CONNECTION

wingspan of 165'. Fifty years from now, who knows?

No Columns Along 1,275' Opening

Providing a column-free clear opening along the hangar doors would alleviate concerns that fixed objects such as columns along the door opening would hinder future operations. However, accomplishing this meant that no columns would be allowed along one of the 1,275' sides. The challenging design parameters were met by an innovative combination of 282'-long cantilevered steel trusses supported by reinforced and post-tensioned concrete towers along the back wall of the high bay space.

The high bay roof structure is composed of seven bays of essentially identical framing: 165' span trusses, with depths varying from 16' to 24', space 44' on center, supporting long span open web bar joists. These bays of framing are supported on wide flange columns at each end of the 1,275' long building, and by six identical cantilevered box trusses along the interior of the building. The box trusses are suspended from six concrete towers measuring 25'x100' in plan, and rising to a height of more than 160' above the finish floor.

In addition to the customary design loads for the roof structure (structure weight, roofing and insulation, catwalks, plumbing and fire protection, and code required roof live load), it was required to support a 10-ton bridge crane in each bay plus the weight of hanging work docks serving the fuselage and horizontal and vertical stabilizers. The cranes and docks along impose a design load of 540,000 lbs. per bay onto the roof structure.

Project architect was Corgan Associates Architects, Dallas, and structural engineer and prime consultant was L.A. Fuess Partners Engineers, Dallas. General contractor was HCB/Walker Joint Venture Dallas/Ft. Worth.

Structural Details

From a purely structural standpoint, the lightest, and therefore most economical truss, has maximum depth at point of maximum moment. It was imperative, then, to obtain the maximum depth possible at the face of the concrete towers.

The top of the concrete towers was set at the maximum height allowed by FAA standards with regard to radar and instrument landing guidelines. It was possible to slope the bottom chord of the truss down over the nose of the aircraft and the second floor mezzanine, so that the bottom chord framed into the towers at the fourth floor level, providing a depth of 100′ from the point of support measured from the top of the tension strut to the bottom chord of the box section. At that point, the depth between the top and bottom chords is 20′, and it varies over its length from 16′ deep to 27′ deep.

The top chord of the box section is sloped from front to back at a minimum of ½" per foot to provide the necessary roof drainage, establishing the box depth and also setting the depth of the 165′ span trusses in the orthogonal direction.

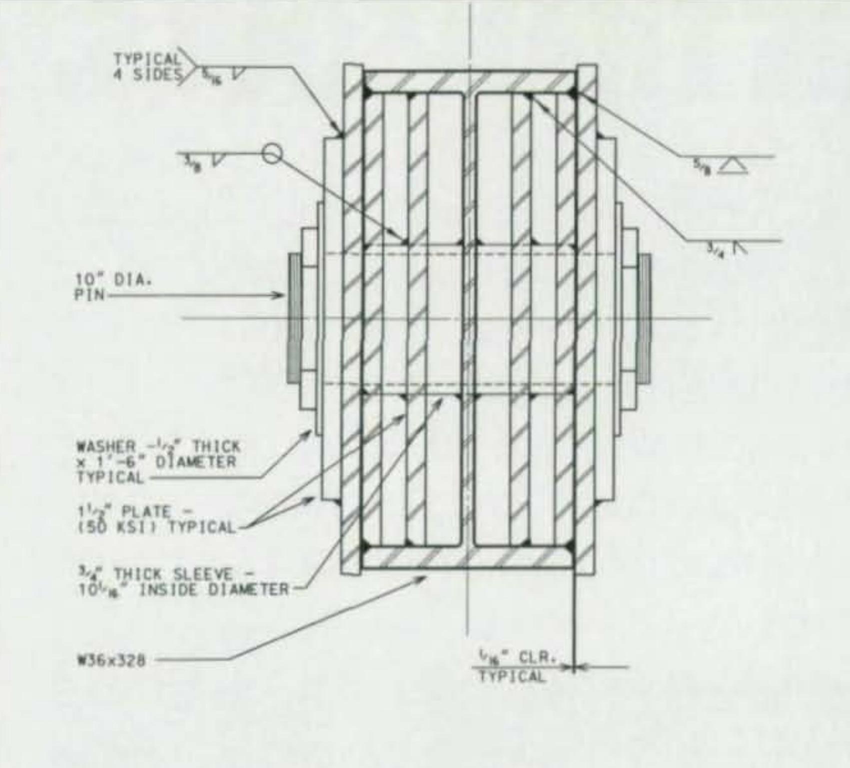
All trusses on the project are composed of 14" wide flange sections turned with webs horizontal for connection simplicity. Virtually all 7,770 tons of structural steel on the project, including all truss members, is ASTM A572 Grade 50. The largest individual truss member is a W14x455 located on the box truss bottom chord adjacent to the tower. The box truss tension struts are W36x328.

Tension Struts

Of utmost importance to the design and constructability of the truss was the connection detail at the top of the tension struts. The design force in each tension strut is 2,400 kips, which must be transmitted to the concrete tower at a joint in which three concrete members intersect. The joint contains a total of 76 #11 reinforcing bars converging from three directions, plus eight 3¾" diameter corrugated ducts in the diagonal concrete member housing a total of 96 0.6", 270 ksi post-tensioning strands,

WE HELPED CALVARY CHURC

Calvary Church in Charlotte, North Carolina seats 6,200 people.



SECTION @ 10" PIN

plus various other ties and confinement reinforcing.

With careful design and dimensioning of the reinforcing steel and post-tensioning, it was possible to provide for 20 2½"-diameter anchor bolts to support the 2,400 kip tension force. The 12'-long, ASTM A193-B7 bolts bear on a 4"-thick steel bearing plate on the back side of the concrete member.

The design load is transmitted from the W36x328 tension strut to the bolts through a 4½ ton weldment composed primarily of 3"- and 4"-thick plates. All plates in the weldment are fine grained, silicon killed, 50 ksi steel to allow for the required complete penetration groove welds. All welding on the assembly was performed in the shop. A 10"-diameter, 50 ksi steel pin provides the connection between the wide flange strut and weldment.

Correct location of the anchor bolts was assured through the use of a shop welded steel pipe sleeve assembly that was positioned in the forms and cast within the concrete beam at the peak of the tower. The plates on each end of the embedded assembly were match drilled with the bearing plate and weldment to assure proper fit-up in the field. In addition, varying thicknesses of finger shim plates were used between the weldment and the face of concrete to allow for erection tolerances and elevation adjustment of the truss.

The compression connection to the tower at the bottom chord is accomplished through a similar weldment and 10" pin. The face of the concrete at this location is cast such that the resultant truss force is orthogonal to the concrete. After final adjustment of the truss, the base plate was grouted with high-strength, non-shrink grout.

The cantilevered truss was analyzed and designed with a three dimensional computer model using STAAD-III from Research Engineers and Intergraph's Micasplus.

The inside of American Airline's new maintenance and engineering facility is designed without columns to accommodate even the largest aircraft. The roof structure can support 540,000 lbs. per bay.

Great care was taken during the design and erection of the steel members. The erector, Derr Construction Co., lifted the box section in three parts and provided three sets of shoring towers to temporarily support the box sections prior to adding the tension struts.

The 3D analysis was required to accurately model the affect of unbalanced loading on the box section, such as full dock and crane load on one bay and none on the adjacent bay. Calculated deflection at the tip of the cantilever due to all loads is 17"; 7" on this was cambered out during fabrication and erection. Erector was AISC-member Derr Construction Company.

150,000 Bolts

The project required 800,000 drilled bolt holes and 150,000 high-strength bolts. All trusses were field bolted using A325 or A490 bolts of various sizes, in bearing connections. Both halves of each cantilever truss, and all 165′ span trusses were required to be completely assembled in the shop to assure proper fit-up in the field. Trusses were shipped knocked-down and re-assembled in the field prior to lifting.

Each cantilevered truss was fully erected and adjusted to the proper tip elevation prior to beginning erection of the simple trusses in the adjacent bays.

In addition to the final cantilevered configuration, the truss was designed to be supported during erection at a point directly beneath the intersection of the box section and the tension struts. Derr elected to lift the box section in three sections and provided three sets of shoring towers to temporarily support the box sections prior to adding the tension struts.

After all connections were complete, two sets of shoring towers were lowered and the truss was adjusted by jacking at the location described previously. After the necessary shim stack thickness at the upper tension strut connection was achieved, providing the proper tip elevation, the anchor bolts were tightened and the jacking towers removed.

A series of horizontal X-braces at the bottom chord level provides the diaphragm required for stability and lateral forces. Composed of field bolted double angles, these elements transfer lateral forces to the concrete towers and vertical steel X-braces located in the end walls.

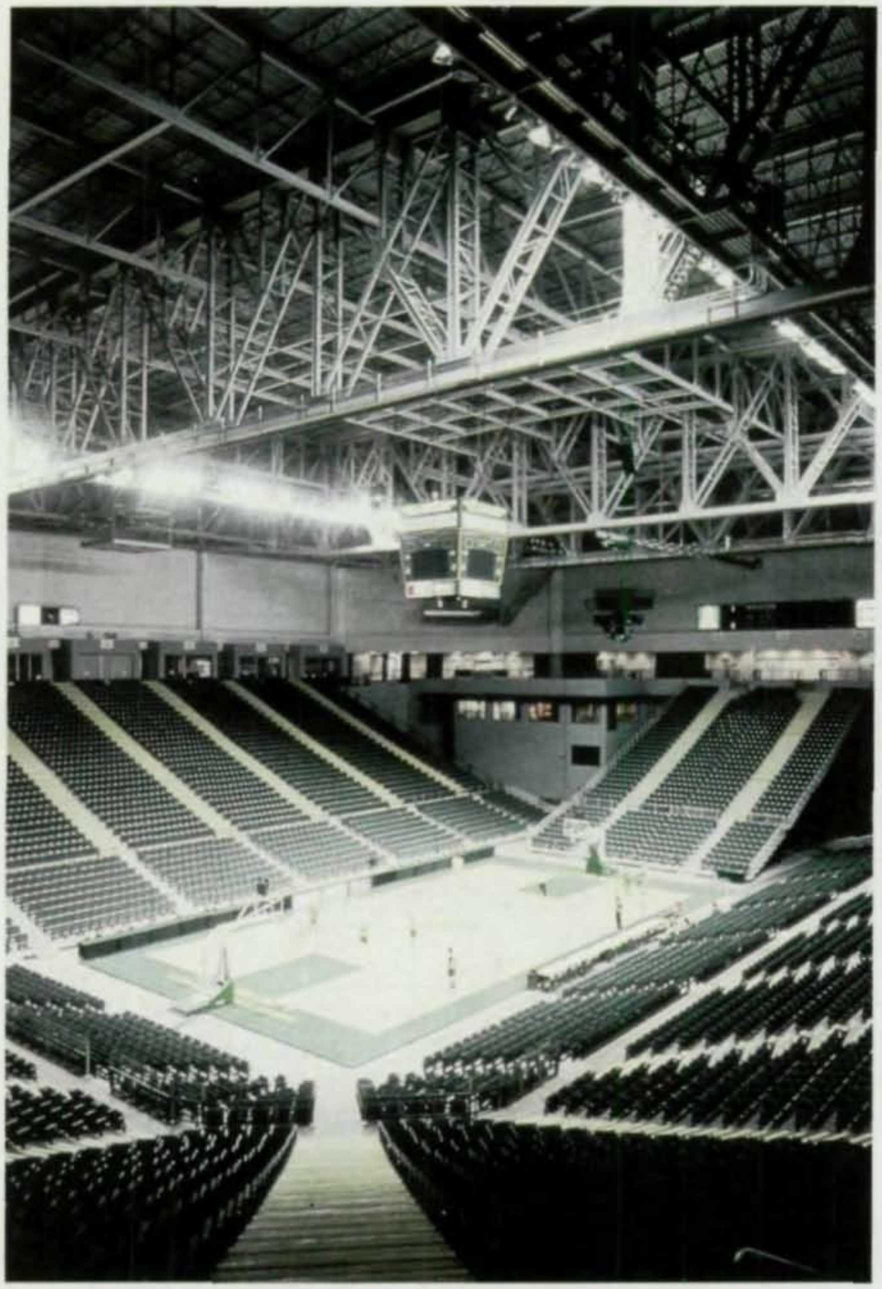
The shop and office area along the back of the hangar building between concrete towers contains 300,000 sq. ft. of composite steel floors. All framing is with ASTM A572 Grade 50 steel. Design live load for these floors is a hefty 250 psf.

The hangar doors for this project consist of eight independently operated sections, each 75′ tall and 159′ wide. They are electrically operated, motor driven, and run along crane rails embedded into the floor slab. The telescoping roller guides at the top of each door leaf allow for the vertical deflection of the roof structure and transmit lateral forces to the bottom chord diaphragm framing.

The roofing of the building is a standing seam metal roofing over rigid insulation and a base steel deck. Specially designed light gauge metal Z purlins nest into the base deck flutes and are screwed directly into the bar joist top chord. The standing seam panels are attached to the top flange of the Z purlins as in a "normal" metal roofing application. The base steel deck, which is a standard 22 gauge roof deck, provided a working platform for the roofing contractor and neat, durable finish ceiling. The standing seam panels were field rolled to lengths required in order to eliminate end laps in the panels. Maximum sheet length is 200′.

Design of the hangar facility began in August 1989, with construction commencing in March 1990. American Airlines began servicing aircraft in the facility in October 1991.

Jon C. Herrin, P.E., is a principal/vice president with L.A. Fuess Partners Inc., a structural engineering firm headquartered in Dallas. ■



Long-Span Performance

Steel trusses clear-spanning 287' create the wide-open space needed for a combined basketball arena/performance center

By Donald W. Hoffmann, P.E.

Although the original design of the Ervin J. Nutter Center—which allowed the architecture to reflect the building's structural design—proved too expensive, the engineers still managed to maintain the desired appearance by incorporating a more conventional—and more economical—steel frame.

The original framing concept for the new athletic and performance center at Wright State University near Dayton, OH, featured a space truss system known as a *Takanaka Truss*. The shape of this system has its top chords in a direction of 45 degrees to its bottom chords and its first diagonal slopes inward and upward to the top chord creating a mansard-type roof shape. The geometry of the Takanaka Truss established the perimeter mansard roof profile and dictated the main column locations. In addition, the entire design of the seating and aisle layout (10,500 seats for basketball; 13,000 for performances) was based on this geometry.

But what started as a wonderful blending of architecture and engineering proved too expensive; when the roof structure was bid, the cost exceeded budget by $1 million. Fortunately, a redesign using conventional linear trusses and other conventional steel framing allowed the project to come in on budget.

The redesign used four main trusses varying in depth up to 30' and clear spanning 287' across the arena. Sub-trusses spanning 62' and 105' frame between the main trusses. Metal deck and bar joists frame between the sub-trusses.

The center, which opened late in 1990, includes physical education and athletic facilities. Auxiliary gymnasiums and an indoor running track provide recreational facilities for the student body. KZF Incorporated, headquartered in Cincinnati, was the prime architect and engineer. Construction manager was Shook Building Group, Dayton.

Structural Challenges

Since the original Takanaka system established the roof profile and column locations, KZF engineers had a structural challenge to adapt a conventional system to fit the mansard sloped roof design. In order to maintain the architectural design—a truncated pyramid with clipped corners—the structural engineers designed a perimeter space truss around the edges of the roof to support both gravity and lateral loads with spans of as much as 172'. The perimeter space truss was designed to fit the architectural profile and provided the needed three-dimensional stiffness to stabilize the main roof trusses.

In addition to the roof truss dead load, the design roof live load consisted of snow load and an additional 20 psf for miscellaneous loadings—primarily the lighting, event rigging, sound and catwalk systems. The lateral loads used were based upon a wind speed of 90 mph, exposure "C" and Zone 2 seismic loadings.

Spherical bearings were used to accommodate the truss end rotations and to uniformly distribute the truss reactions. The lateral forces from wind and earthquake are distributed to two columns on the north wall and two columns on the east wall. At these four locations, guided slide bearings transfer the lateral thrust to concrete columns in one direction while allowing the bearing to slide in the other.

The four remaining truss reactions are supported by unguided or free-floating slide bearings, which theoretically transfer no lateral load to the column tops, except for the friction of the slide bearing. The temperature effects on a structure of this size required provisions to accommodate 4" of movement by the slide bearing.

Jumbo Sections

For the truss design, the top and bottom chords used W40 jumbo wide flange sections turned horizontally. The weight ranged from 167 lbs to 531 lbs per linear foot.

Dual gusset plates were connected to the outside face of the flanges to essentially form a through truss, which was used as part of the catwalk system. This

Top photo courtesy of Wright State University; others by Greg Girard.

The mansard-style roof shape reflects the original structural design of the center. When that system proved too expensive, the engineers switched to a more conventional steel system but managed to still keep the aesthetically desirable roof shape. Photo by Greg Girard

chord orientation allowed for two reasonably light diagonal web truss members connecting the top to bottom chords. An additional benefit was a very stiff truss in the plane of the truss, a major benefit during the erection of the truss. Prime steel fabricator on the project was AISC-member Carter Steel & Fabricating Co. and steel erector was AISC-member John Beasley Co.

Pre-Assembled In The Shop

All of the major truss components were shop assembled to avoid the problem of member fit-up when dealing with such large members. A combination of pre-punching and shop drilling of the bolt holes as the components were assembled insured a good fit. The trusses were then partially disassembled, as needed, for shipping.

The main trusses were assembled in thirds on the ground. Each 40- to 60-ton section was then lifted into position. The outer portions were suspended from a guyed "gin pole" and the center section lifted in place. The sub-trusses, which also act as bracing for the main truss, were then framed into the main trusses. These sub-trusses were not pre-assembled in the shop, but were assembled on the ground and lifted into position.

The perimeter edge space trusses proved to be the most time-consuming design portion of the roof system. Assisted by a computer, analysis of the design was fairly straight-forward, though still time consuming. Analysis was performed using Structural Analysis Inc.' (SAI) "Space" and "Plane" programs.

Detailing the joints presented a unique challenge. A joint with the axis of all members framing to a single concurrent point was physically impossible to construct. Therefore, a few of the members had to be offset from their theoretical alignment and the eccentric forces resolved with the addition of joint stiffeners. More than 400 fab-

The Ervin J. Nutter Center at Wright State University is a combined sports and entertainment center. It seats 10,500 for basketball games and 13,000 for concerts and also includes auxiliary gymnasiums and a running track. Photo by Gordon Morioka.

rication drawings were needed to detail the complex roof system.

Lateral Loads

The transfer of lateral loads from the roof system to the supporting columns and foundations presented the challenge of dealing with 250 kips of lateral load applied to the top of a 6′-diameter cantilevered column 50′ above the concourse level. Two stability columns were located on the east end of the arena and extend an additional 35′ to the arena floor. The stability for these columns was provided by a diaphragm link to a nearby concrete stair/elevator tower.

However, this bracing arrangement produced unexpected design problems when the effect of a long cantilever and a short back span were analyzed. The "PRYBAR" effect magnified the 250 kip horizontal load at the column top to approximately 800 kips just above the concourse level.

To provide the needed stability a pair of W36x260 beams were encased within the 6′ diameter concrete arena columns. Also, a horizontal steel transfer truss cast within a concrete diaphragm slab was used to transfer the loads to the stair/elevator "shear" towers. These towers provided the east/west stability not only for the arena roof but also for the adjacent academic wing and auxiliary gym.

The auxiliary gym is 120′ x 240′ with a 33′-10" clear height. Long span joist girders span the 120′, spaced at 20′ centers. The depth of the joist girder varied from 7′ at the low end to 9′ at the high end. Around the perimeter of the auxiliary gym, a 9′-wide banked running track was suspended from the roof girders.

The academic wing is a 3-story structure with approximately 35,000 sq. ft. per floor. It was framed with conventional composite beam and column construction.

Donald Hoffmann, P.E., is a senior project structural engineer with KZF, Incorporated, a multi-discipline design and consulting firm in Cincinnati.

CALL FOR PAPERS

1993
National Steel Construction Conference
Orlando Convention Center, Orlando, FL
March 17-19, 1993

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Home Phone ( ) ____________ *Preferred mailing address ( )Business ( )Home

Co-Author(s):

1. Name ____
(First) (Middle Initial) (Last) (Professional Suffix-Degree)

2. Name ____
(First) (Middle Initial) (Last) (Professional Suffix-Degree)

3. Name ____
(First) (Middle Initial) (Last) (Professional Suffix-Degree)

Invitation/Call For Papers

The 1993 National Steel Construction Conference will be held at the Orlando Convention Center, March 17-19, 1993. Participants include structural engineers, fabricators, erectors, educators, and researchers. Potential authors may submit abstracts of papers on design, fabrication and erection of steel structures for buildings and bridges. Topics of interest include:

Practical application of research;

Advances in steel bridge design and construction;

Composite members and frames;

Buildings designed by LRFD;

Heavy framing connections;

Steel-framed high-rise residential buildings;

Partially restrained connections and frames;

Economical fabrication and erection practice;

Quality assurance and control;

Case studies of unique projects;

Computer-aided design and detailing;

Case studies of unique projects;

Computer-aided design and detailing;

Material considerations;

Fire Protection;

Coatings and material preparation;

Structural systems.

Guidelines for Abstract Proposals

Abstracts for papers must be submitted before June 15, 1992. They should be approximately 250 words in length, and submitted on a separate sheet of 8 1/2" x 11" white paper attached to this form.

Authors will be informed of the Organizing Committee's decisions by September 1, 1992. Successful authors must submit their final manuscripts for publication in the 1993 Conference Proceedings by December 15.

Preparation of Paper

Final manuscripts for publication in the official 1993 Conference Proceedings are expected to be approximately 20 pages in length. Copy (including photographs) must be camera-ready. Complete instructions will be forwarded to authors upon acceptance of Abstract Proposals.

Poster Session

Papers not accepted for presentation at the Conference may, at the author's expense, be presented at the Conference Poster Session. Guidelines for the Poster Session will be provided upon request.

Return your abstract with this submission form before June 15, 1992 to:
American Institute of Steel Construction, Inc., One East Wacker Drive, Suite 3100
Chicago, IL 60601-2001 Attention: 1993 NSCC Abstracts
Phone 312/670-5400 Fax: 312/670-5403

Steel Pyramid

A pyramid proved the ideal structure for a rain forest exhibit

By Ali Haris, S.E., Ph.D., and Ian Robertson, P.E., Ph.D.

The latest addition to an education/conference center on Galveston Island along Texas' Gulf Coast is a lush rain forest housed in a steel-and-glass pyramid.

The 142-acre Moody Gardens, a non-profit complex of educational, physical treatment and entertainment attractions, is in its sixth year of a 20-year master plan and already boasts a convention/conference center, an animal contact facility used for both physical therapy and children's education, an equestrian arena used for physical therapy, plus the only white sand beach along the Texas coast.

The new rain forest exhibit, which is currently under construction, is expected to be the complex's most spectacular project to date. It will include displays of a wide variety of tropical flora, exotic fish in fresh water tanks and ponds, a Mayan ruin, a Japanese tea house, a series of waterfalls cascading from a man-made mountain, and a 2D/3D IMAX theater in a companion visitor's center. The entire one-acre exhibit will be housed in a 100'-tall pyramidal glass-and-steel enclosure.

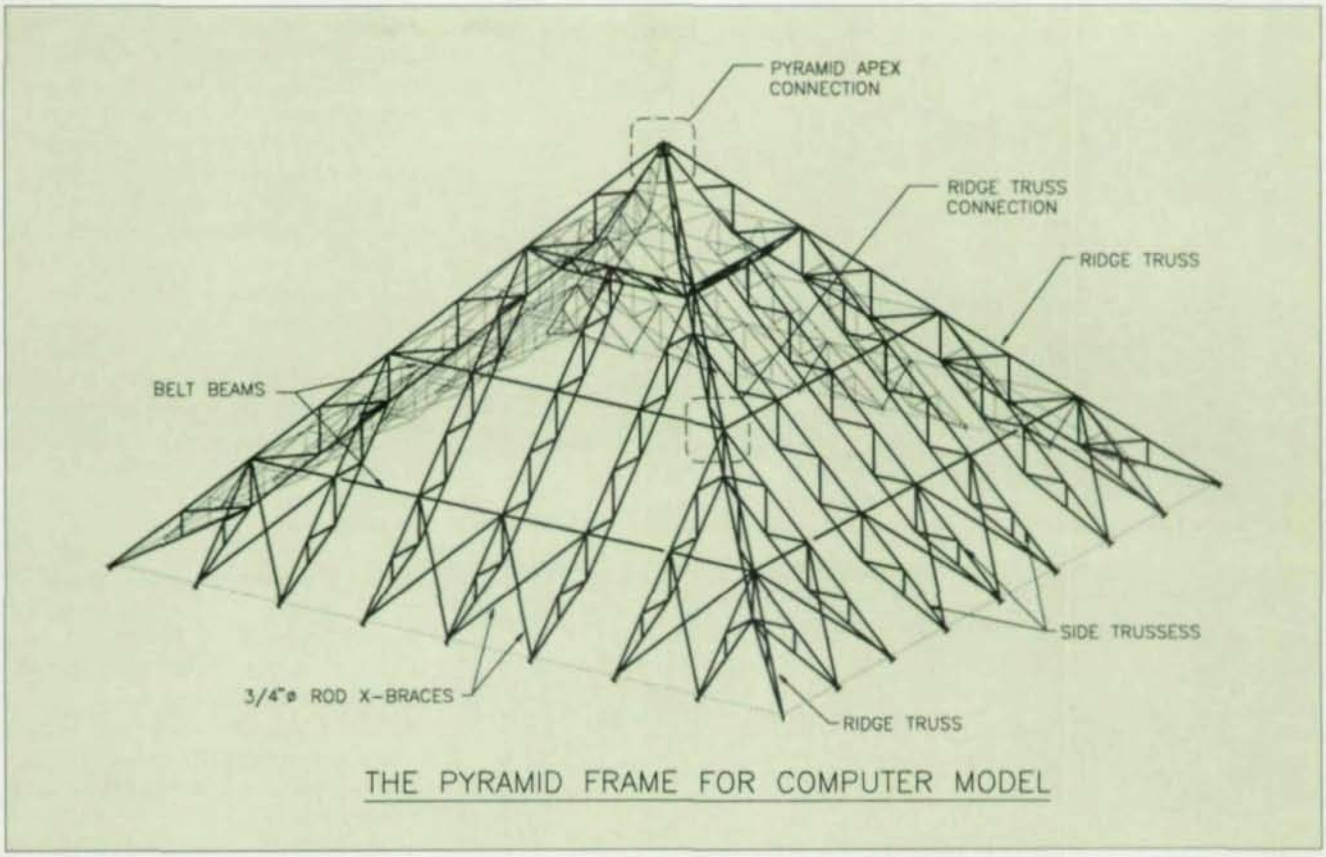
The pyramid shape was selected for its superior structural economy and inherent stiffness under the 120 mph hurricane winds common along the Texas coast. In addition, the pyramid provides sufficient height near the center to accommodate large tropical trees.

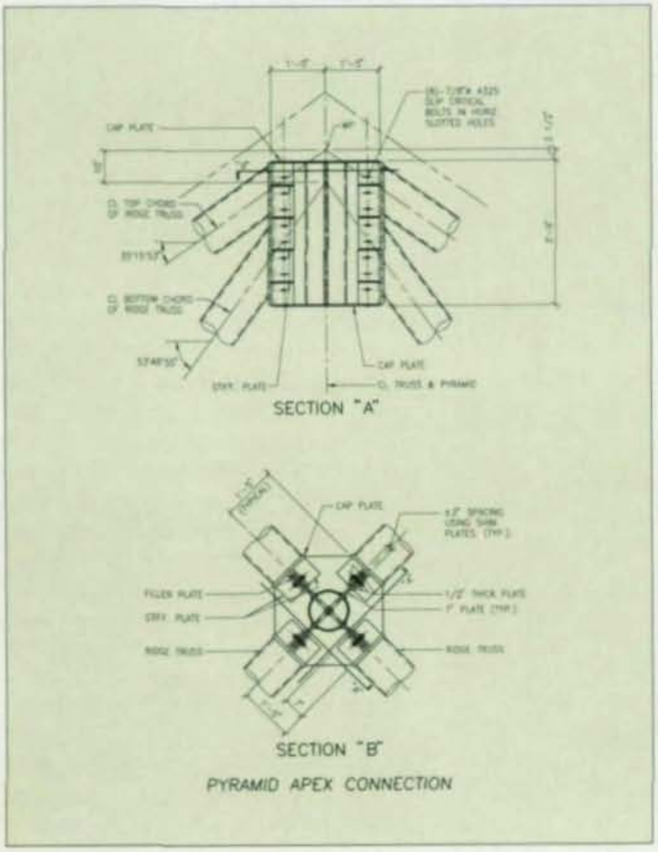
Structural engineer Walter P. Moore and Associates, Inc., and architect Morris * Architects, both of Houston, collaborated to effectively blend the efficiency of the pyramidal structure with the stringent functional criteria of the unique exhibit. As the design developed, a third team member, the Houston office of construction manager Gilbane Building Co., provided analyses to confirm the economy and constructability of the structural system.

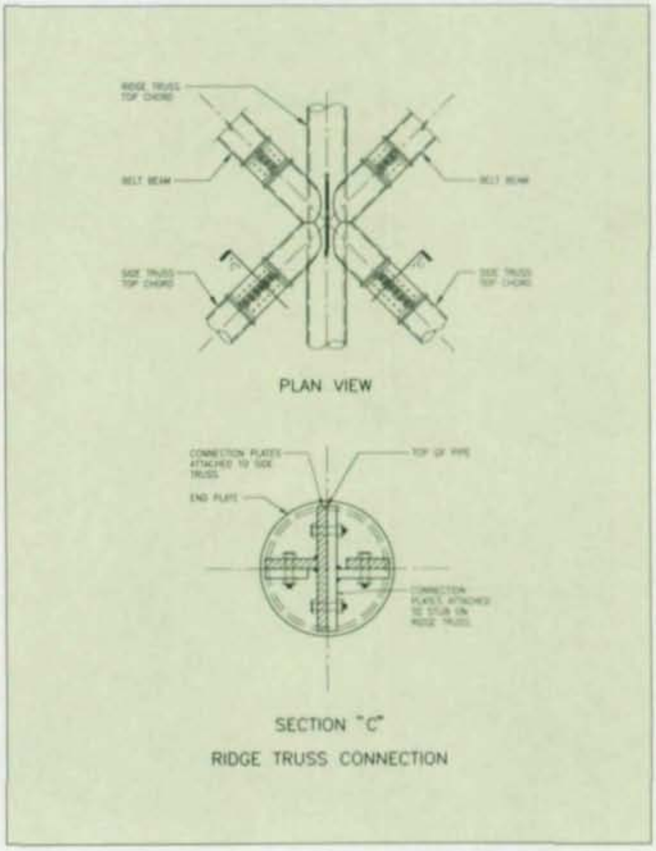
Analysis And Design

The structural frame consists of steel trusses with welded pipe members and steel tube purlins supporting sloping glass cladding. Pipe sections were used for truss elements and tube sections for purlins to minimize the effective size of all members, thereby allowing the maximum amount of sunlight into the rain forest exhibit. Likewise, welded connections minimized the size of gusset plates, which also reduced shadowing.

To assure a comprehensive analysis, the structure was analyzed

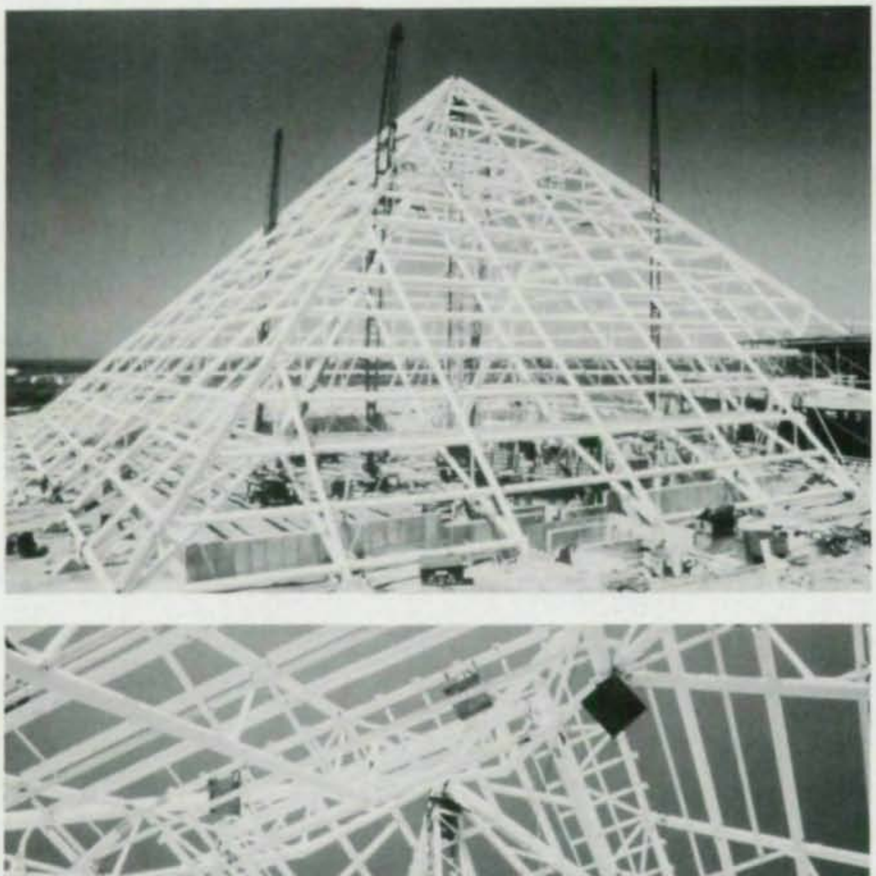






Opposite photo courtesy of Jack Kraft

A center erection tower and four red gin poles supported the ridge trusses during erection. Additionally, the center erection tower supported the apex connection as each quadrant fo the pyramid was erected. These supports were removed after all bolted connections were torqued. Pictured on the opposite page is the side truss-to-ridge truss top chord connection. Pipe stubs projecting from ridge trusses simplified connection of side trusses and belt beams. The slip-critical high-strength bolts were torqued prior to final cleaning and painting with a special corrosion resistant white coating. The simplicity of the truss connections helped to speed the erection process. The majority of the 310-ton structure was erected in just 45 days.

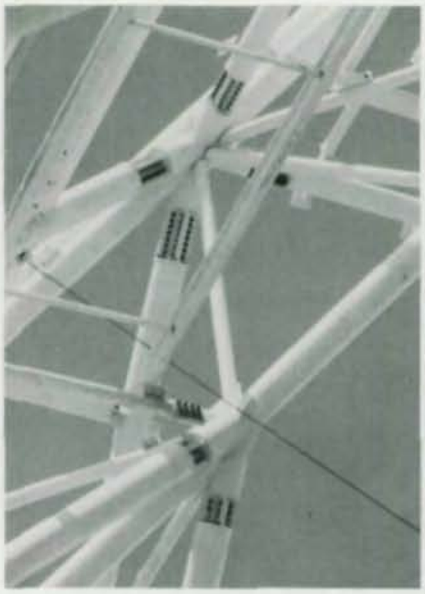


with a three-dimensional computer model using the SAP90 program from Computers & Structures, Inc. The following basic load conditions were considered:

- Dead loads including the weight of the steel structure;
- Live loads on the sloping surfaces;
- Wind loads along each orthogonal axis and along the principle diagonal axes;
- A thermal differential of 0 to 140 degree F;
- Lateral bracing forces in the plane of the top chord of the trusses (which are equal to the forces required to provide second order lateral bracing of the chords).

Wind loads from three sources were considered: the *1985 Standard Building Code* with 140 mph basic wind speed; results of a boundary layer wind tunnel test with 120 mph basic wind speed; and *Building Code for Windstorm Resistant Construction* by the Texas Catastrophe Property Insurance Association. Interestingly, the wind design criteria required by the insurance carrier was the most stringent and hence controlled the majority of the design.

The basic loading conditions generated 35 possible load combinations for analysis. Since the SAP90 post-processor provides only a design check, the engineers developed a customized post-pro-



cessor to retrieve output files from the SAP90 runs and rapidly perform the load combinations. In addition, the post-processor program accomplished three important tasks:

- For each element, it produced a member design to satisfy each load combination;
- For any specific structural steel size specified by the designers, it produced a code design check;
- To develop forces for connection and foundation designs, it produced sets of axial loads, moments, and shears corresponding to load combinations that produced the maximum (or worst) combinations of axial load and moment.

By harnessing the power of the customized post-processor, the designers accurately analyzed the entire structure and designed every element for the 35 load combinations. The result is an efficient and economical structure with a total weight of 310 tons (or 15.5 psf based on the area of the 200′ square base). The post-processor also allowed the designers to utilize a range of readily available pipe and tube sizes.

Design Results

The major supporting elements of the superstructure are four primary ridge trusses, each 173′ long and 10′-2" deep. The trusses meet at the apex of the pyramid as shown in Figure 3. The trusses support—and are braced by—26 intersecting side trusses that vary in length from 35′ to 106′. The 8′-10" deep side trusses are located at alternate chord panel points along the primary trusses.

Pipe elements ranging from standard 10"-diameter pipes to extra-strong 12"-diameter pipes were specified for the truss top and bottom chords, while 5" diameter pipes in strengths of standard, extra-strong and double extra-strong were used for the diagonal web members. Tube purlins, typically 10" square, span between side truss top chords. The purlin sizes were increased to 12" square to take bracing forces at each of the circumferential bracing belts.

Structural steel fabricator on the project was AISC-member Richmond Steel, while the tubes were supplied by AISC-member Welded Tube Co. of America and the pipes were supplied by Saginaw Pipe Co.

The pyramidal structural system performs efficiently, but with a few peculiarities and special design challenges. With the recent design experience of a major pyramid-shaped roof structure ("The Great American Pyramid, Memphis, TN," see June 1991 *Modern Steel Construction*) as background, the design team was able to anticipate and economically resolve these challenges, the most critical of

Shown above is the start of the glazing mullion installation at the base of the pyramid.

which are discussed below.

Connection Design

To simplify erection, all field connections were designed with slip-critical bolts in oversized or slotted holes. The typical connection detail at the joint where the side trusses intersect the ridge trusses is shown in Figures 2 and 4.

To streamline mating of the pieces, a pipe stub with circular end plate and two projecting perpendicular gusset plates was shop-welded to the top chord of the ridge truss chord at each joint. The horizontal gusset plate was split in a dovetail to accept a similar set of interlocking gussets on the side truss. The top and bottom connections on each truss were slipped into position simultaneously.

The connections were designed to resist large simultaneous axial loads, bi-axial bending, and shear.

Thermal Behavior

The key structural elements in the pyramid system are the four primary ridge trusses, which are predominantly loaded in axial compression. Each of the four ridge trusses is braced at alternate panel points by smaller side trusses and horizontal tube members acting as belts around the structure (see Figure 1). The horizontal belt members are subject to significant axial loads, both in tension and compression.

Near the base of the pyramid, where the belt members are longest and subject to the highest end constraint, the stresses due to temperature extremes are significant. To resolve this concern, the first line of belts above the ground were detailed with releases at one end of each member to allow axial slip movement. To compensate for the loss of lateral stiffness that these slip connections created, a series of ¾" rod X-braces was added in the plane of the sloping wall (see Figure 1).

Settlement Sensitivity

Any pyramidal structure is inherently sensitive to differential settlement of the supports. Even relatively modest settlement of one truss support may reduce that truss' axial load significantly and increase the loads at the adjacent trusses. The resulting redistribution of forces can overstress other elements. In response, a very stiff system of deep pile foundations was utilized on the Rain Forest Pyramid to limit the maximum expected differential settlements to less than 0.25".

Field connections between the trusses, as well as for purlin-to-truss connections, were detailed

Pipes and tubes were used to minimize the size of the steel members and therefore maximize the amount of light streaming through the glass walls of the Rain Forest Pyramid at Moody Gardens on Galveston Island, TX.

with high-strength A325 bolts. The purlin-to-truss connections in particular were detailed to minimize joint size, which both simplified erection and reduces sunlight shading.

Construction Issues

Differing support locations and lateral bracing of individual members during erection make adequate erection analysis important for any long-span roof structure. Because the pyramid is so efficient—and hence individual member sizes are relatively small—erection concerns are particularly important. As a result, the designers fully analyzed the structure for each phase of construction with construction loads and temporary erection towers in place. In addition, the erection scheme was fully developed and described on the contract drawings.

The steel erector, Peterson Brothers Steel Erection, Co., followed the specified erection scheme and sequence very closely. A guyed erection tower was placed at the center of the structure to support the top of the ridge trusses. Four secondary gin poles were placed near the mid-span of each of the ridge trusses. Since the ridge trusses were designed as predominantly axially-loaded members, a temporary support near mid-span was necessary to prevent flexural overstresses and reduce deflections due to self-weight prior to completion of all erection and the resultant efficient pyramid behavior.

Several pre-erection meetings were held between the structural engineer, fabricator and construction manager to work out all aspects of steel fabrication, detailing and erection. As a result of the careful planning and detailing of this structure, the erection progressed very smoothly, requiring only 1½ months to install all trusses and the majority of the purlins. Completion of the slip critical bolted connections continued for another month before the erection towers were removed, and deflection at the apex of the pyramid was even less than the ½" expected.

After erection, the structural members and connections were cleaned and painted white with a high quality exterior coating system from DeVoe Coating Co. to prevent corrosion and improve light reflection.

The project is scheduled for completion in Spring, 1993. It successfully blends the economy of structural steel with the efficiency of the pyramid shape to create a graceful structure. Welded steel pipe trusses and tube purlins satisfy all of the architectural and structural requirements, including resisting hurricane winds and minimizing shading.

Ali A.K. Haris, S.E., Ph.D., is a vice president and Ian Robertson, P.E., Ph.D., is an associate with Walter P. Moore and Associates, Inc., a leading structural and civil engineering firm headquartered in Houston. ■

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For more information, contact: Isolatek International, 41 Furnace St., Stanhope, NJ 07874 (201) 347-1200; FAX (201) 347-9170.

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For more information, contact: Mitch Braun, Marketing Services Manager, Carboline Co., 350 Hanley Industrial Court, St. Louis, MO 63144 (314) 644-1100; FAX (314) 644-4617.

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For more information, contact: Lisa Seidel, Sales & Marketing Coordinator, Fire Protection Systems, Textron Specialty Materials, 2 Industrial Ave., Lowell, MA 01851 (508) 452-8961; (508) 454-5619.

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