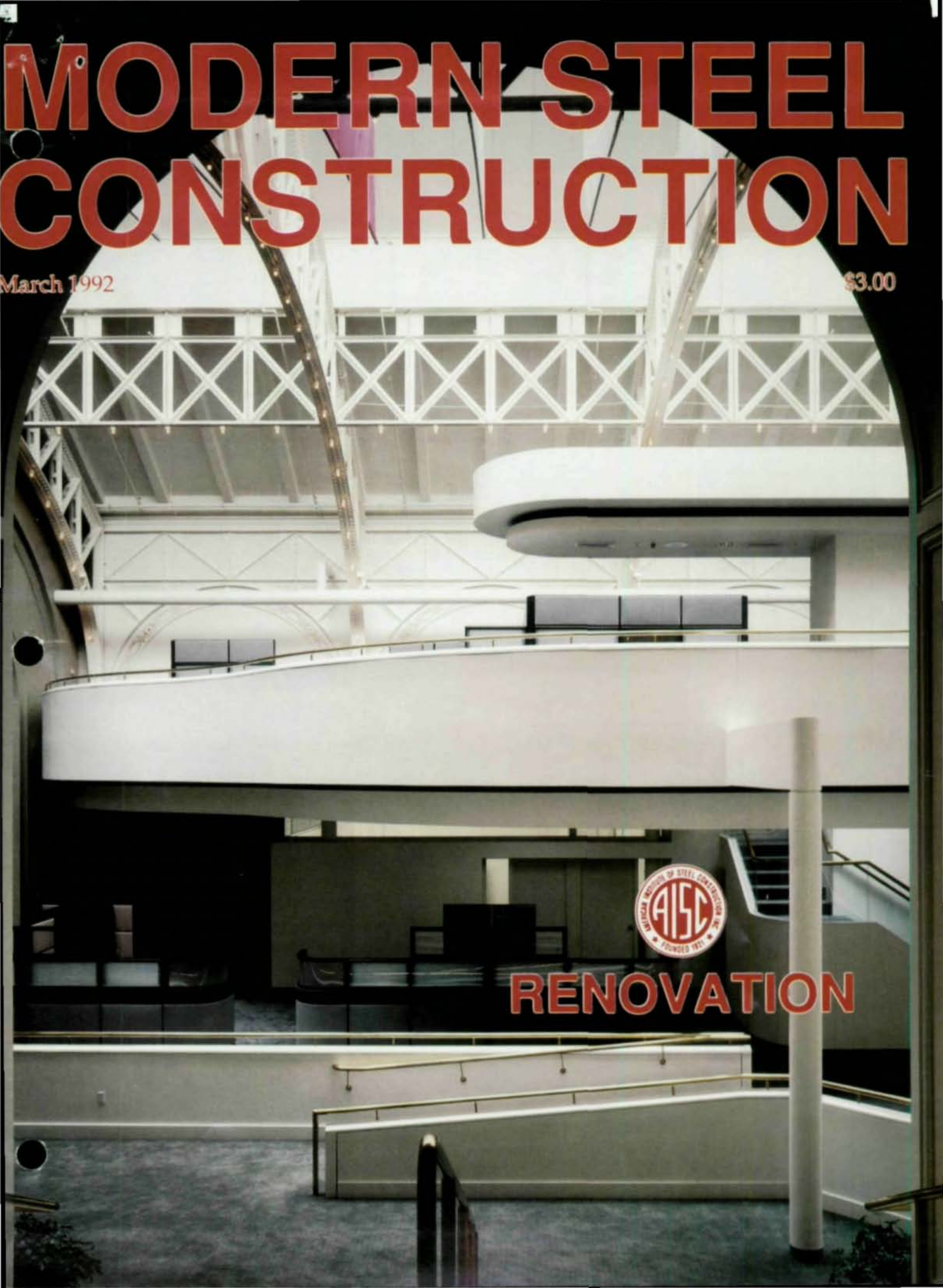

MODERN STEEL CONSTRUCTION
March 1992
$3.00
RENOVATION

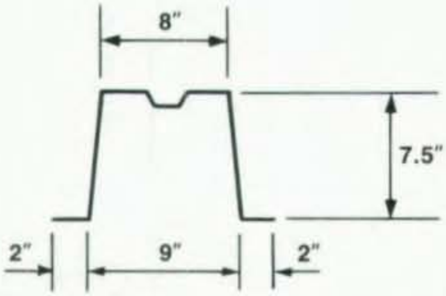
UNITED STEEL DECK, INC.

DECK DESIGN DATA SHEET

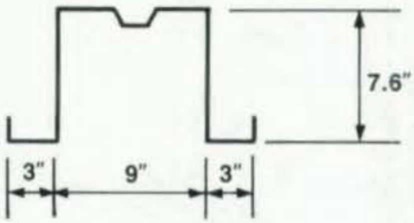
No. 16

CUSTOM DECKS

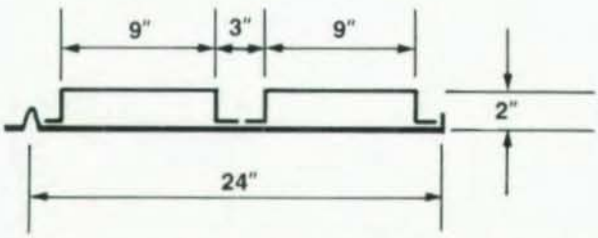
THESE SECTIONS ARE ALL DIFFERENT. NOT ONE IS A "STANDARD" DECK SHAPE. BUT, THEY DO HAVE ONE THING IN COMMON — THEY WERE ALL MADE AT **UNITED STEEL DECK, INC.** TO SERVE A NEED FOR A PROJECT THAT REQUIRED A CUSTOM DESIGN. CALL IF YOU HAVE AN APPLICATION THAT REQUIRES CUSTOM BENDING. YOU MIGHT BE PLEASANTLY SURPRISED AT THE SAVINGS A SPECIFICALLY ENGINEERED PRODUCT CAN PROVIDE.



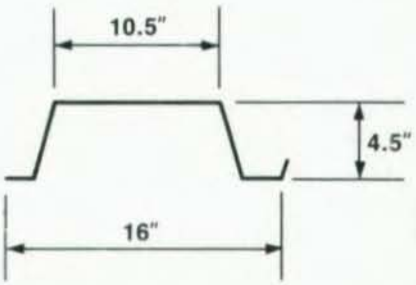
16 GAGE FORM DECK — USED TO BUILD A PIER WITH AN 18" SLAB.



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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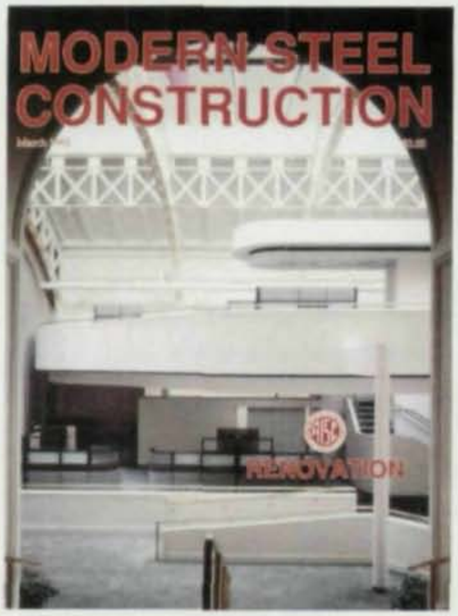
MODERN STEEL CONSTRUCTION

Volume 32, Number 3 March 1992



New mezzanines added nearly 25% additional space for the New York State Education Department. The gracefully curved steel-framed structures cantilever from outriggers extending from the existing steel columns. For more information on this project, turn to page 24. Photo by Jeff Goldberg/ESTO Photographics, Mararoneck, NY.

FEATURES

NEWS AND DEPARTMENTS

Modern Steel Construction (Volume 32, Number 3). Published monthly by the American Institute of Steel Construction, Inc. (AISC), One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

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Idea Exchange

This issue inaugurates a new monthly column titled: "Steel Interchange." *Its purpose is to serve as a dialogue between* structural engineers, fabricators, erectors, academics, and others in the steel construction industry.

Each month we'll print one or more questions with answers from a variety of sources, such as practicing professionals, university professors and AISC staff. We'll also end each column with a list of questions that will be answered in future columns.

And that's where the column's title comes in: We want to create an interchange of ideas about steel design and construction.

If you have any thoughts concerning any of the questions that appear at the end of the column, jot them down (actually, we'd prefer if you typed them double-spaced) and send them to: Steel Interchange, Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001. And if you have any comments on answers appearing in the column, send them along too.

Also, we're looking for questions to pose. If there areas of steel construction about which you have questions, send them to the address listed above.

Of course *MSC* will still run a "Correspondence" column. If you have any comments on anything appearing in the magazine or on subjects of interest to the steel design and construction community, please send them.

Steel Renovation

Flexibility has always been touted as one of the advantages of steel and part of flexibility is the ready adaptability of steel to change, even years after the building is completed. In this issue we examine four examples of steel renovation.

On the glamorous side is the addition of a mezzanine to a historic structure in Upstate New York (page 24). The addition was made possible by the combination of the engineers design for extending some existing steel columns and the lightweight of the new steel construction that didn't overload the load bearing capacity of the existing members. In a similar vein is the addition of skyboxes and a skylobby, as well as a major expansion of an existing theater in the Madison Square Garden Complex (page 30).

A much smaller project was the cutting and reinforcing of an perimeter opening at the ninth level of a steel-framed building in Indianapolis (page 36) to accommodate a passageway to two newly constructed buildings.

And on page 16 we present a proposal by three practicing engineers for how the nation's inventory of existing truss bridges can be economically upgraded.

More than 60% of the construction dollars spent in the U.S. are for renovation. And steel's flexibility makes it a prime component for every structural renovation project. **SM**

Steel Interchange

This is the start of a new monthly column to discuss questions regarding structural steel design, fabrication and erection. *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, 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. 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.

How can one get the out-of-date design specifications and properties and dimensions of structural steel shapes that are not currently being produced?

When AISC prepares a new edition of the *Manual of Steel Construction* we survey the producers of structural shapes. The *Manual of Steel Construction* includes the properties and dimensions of the structural steel shapes currently rolled. The structural steel shapes are defined in the ASTM Specification A6/A6M, *Standard Specification for Rolled Steel Plates, Shapes, Sheet Piling, And Bars For Structural Use*. Surveying the producers and using the steel sections in the latest ASTM Specification A6 insures that the AISC Manual provides up-to-date information.

However, as the infrastructure ages and our buildings and bridges need renovation or retrofitting, they often have to be evaluated and, if necessary, strengthened to meet the current needs. And many of these structures were built with steel shapes and grades that are not produced today. The AISC book *Iron and Steel Beams 1873 - 1952* (AISC Publication No. M003) aims at helping engineers and architects to solve the problems that this question raises.

This book includes all of the properties and dimensions required for design of shapes that were produced in the U.S. between 1873 and 1952. In addition to providing design properties of the shapes, the book also contains a section that summarizes the history of the materials standards that were used. The data includes the tensile and yield strength requirements for the steels that were commonly used for bridges and buildings.

Iron and Steel Beams 1873 - 1952 does not contain any of the structural steel design specifications that were in effect throughout this period. Part of the reason for this is the lack of standardization prior to the organization of AISC in 1921. A great many different specifications were in use in the early 20th century: some of these had been developed by various municipalities or cities; others had been prepared by steel or construction companies. There are even instances where designers developed individual, unique design standards for major structures. However, appropriate working stress recommendations that were utilized at the time are shown in this book.

There is consequently no need to find, much less purchase a specification that is out of print. You must, though, take into account the properties of the actual steel that was used, including the very important chemical and metallurgical characteristics, as well as the production method itself. For example, if the structure in question is a bridge that was originally built in 1918, the steel is most likely ASTM A7. This material had a tensile strength between 55 and 75 ksi, and a specified minimum yield stress of 30 ksi. In addition, a laboratory evaluation of a coupon specimen from the steel is desirable, if possible. The loading and design criteria of the present-day building code can then be used along with the identified material properties to assess the adequacy of the structure.

However, it is also essential to consider the chemical composition of the steel; it is not uncommon to find that some of the older materials had relatively large amounts of agents such as sulphur and phosphorus. This composition may result in a relatively high carbon equivalent, which could make welding difficult.

(*Recent AISC* Engineering Journal's *have included several articles on reinforcing existing structures that are of great use to engineers working on renovations.*)

Steel Interchange

The Research Council on Structural Connections' *Specification for Structural Joints Using ASTM A325 or A490 Bolts* states that reuse of non-galvanized A325 bolts is permitted if approved by the Engineer responsible. When should the Engineer approve reuse of A325 bolts?

Research has shown that non-galvanized A325 bolts can be reused in some applications. In order to make an appropriate choice the engineer should have some background knowledge of the research on bolted joints.

The AISC document *Quality Criteria and Inspection Standards* (AISC publication no. S323) has the following recommendation: "A325 Bolts (except if galvanized) shall be considered satisfactory for reuse, regardless of previous use, if the nuts can be placed on the threads and run down the full length of the thread by hand." (Chapter 2, Section III. E.) This is a good, simple rule based on prevention of plastic deformation of the bolt that an engineer can follow when reusing bolts.

The *Guide to Design Criteria for Bolted and Riveted Joints* (AISC publication no. P633) written by Kulak, Fisher, and Struik also includes a section on reuse of high-strength bolts. This book recommends that A325 bolts can be reused once or twice, provided that proper control on the number of reuses can be established. They state that A325 bolts have adequate nut rotation capacity as long as there is some lubricant on the bolt. This lubricant can be the original lubrication or oil, grease, wax or a lubricant that is added later.

There has only been limited testing on repetitive tightening of bolts but some good information can be obtained from the results. A detailed reference on this testing is a recent article in the AISC *Engineering Journal*: Bowman, Mark D. and Miguel Betancourt, "Reuse of A325 and A490 High-Strength Bolts," *Engineering Journal*, AISC, Vol. 28, No. 3, 3rd Quarter 1991, pp. 110-118. This paper reviews the work that has been completed and presents their own research program.

The conclusions that are reached in this paper are as follows:

On the basis of the limited number of tests conducted in this study using ⅞"-diameter A325, A490, and galvanized A325 bolts, the following conclusions regarding bolt behavior can be stated:

1. Bolts lubricated with either wax or grease perform much better, or at worst only equal to, that of similar bolts in the "as-received" condition. Thread lubrication resulted in improvements in the ultimate load, elongation, and rotational capacity of the structural fasteners tested, especially for the galvanized bolts.
2. The load-elongation characteristics of the bolts loaded in repetitive torque do not differ significantly from that of similar bolts in continuous torque. For most bolt types observed, there was a similar pattern of torque-tension behavior between the two loading methods.
3. The performance of 2½"-long bolts was found to be superior to that of 5½"-long bolts when the bolts were repetitively tightened until the bolts failed. The shorter bolts sustained an average of nine complete cycles prior to failure for all bolt types tested, whereas the longer bolts averaged four complete cycles prior to failure. A difference of one or two tightenings was observed between the black A325 and the galvanized A325 high-strength bolts.
4. Thread lubrication was observed to increase the number of cycles to failure of the repetitively tightened test bolts by one to three cycles. Moreover, thread lubrication was found to be more effective in improving the repetitive torque behavior of the galvanized bolts than of the black bolts.

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. The question 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. What procedures should be followed when assessing steel that has been exposed to a fire?

2. How has the recent allowance of snug-tight high-strength bolting for certain types of shear/bearing connections affected your projects?

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

4. What is a good "wind" connection for the top of a column?

AISC's ***1992 National Steel Construction Conference*** is the only "all-steel" conference and trade show produced in the U.S. The 1992 Conference will be held June 3-5 at the Las Vegas Hilton Hotel. Don't miss the opportunity to experience Las Vegas and attend this outstanding event! To request information, simply fill out and mail this postage paid card.

Please send me, without obligation, information about AISC's 1992 National Steel Construction Conference.

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NSCC Scheduled For June 3-5

More than 45 seminars and meetings are scheduled for this year's National Steel Construction Conference in Las Vegas from June 3-5. Also, more than 100 exhibitors will showcase products for the design, fabrication and construction community.

Sessions focus on the specific needs of structural steel fabricators, engineers, architects, contractors, owners, public officials, erectors, detailers, researchers and educators. Topics include: codes and specifications; computerized design; research; project and shop management; inspection and safety; and fabrication and erection procedures. Workshop sessions get down to basics: the nuts-and-bolts details of designing, fabricating and erecting steel structures.

Contact: David G. Wiley, AISC, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001 (312) 670-5422.

March 6. **CONXPRT and Moment Connections** in Denver with Thomas M. Murray, Prof., VA. Tech. Contact: Jim Anders (214) 369-0664.

March 11. **Texas Structural Steel Institute** in Houston. Contact: Ken Teel, Am. Steel & Aluminum (214) 264-1533.

March 11-12. **SSPC tutorial on lead paint removal** in Minneapolis. Contact: Rose Mary Surgent, SSPC, 4400 Fifth Ave., Pittsburgh, PA 15213-2683 (412) 268-2980.

March 12. **Northeast Steel Bridge Forum**, Boston. Contact: Camille Rubeiz, Steel Bridge Forum, c/o AISI, 1101 17th St., N.W., Suite 1300, Washington, DC 20036 (202) 452-7190.

March 18. **Earthquake Design** (breakfast meeting), St. Louis. Sponsored by AISC Regional Advisory Missouri/Southern Illinois Committee. Contact: Phil Stupp, Stupp Bros. Bridge & Iron Co., 3800 Webber Road, St. Louis, MO 63125 (314) 638-5000.

March 18. **Construction Economics in the Southwest**, Little Rock Chapter of ASCE dinner meeting, Little Rock, AR. Contact: Joe Hilliard (501) 372-2900.

March 24. **Tubular Sections in Building Construction** (co-sponsored by AISC and VCSSFA) breakfast meeting in Greenville, SC. Will include design criteria, Type 2 Connections, tube-to-tube connections, design guides, practical recommendations and application examples.

March 24-26. **American Welding Society Show and Exposition**, Chicago. Contact: AWS, 550 N.W. LeJeune Road, P.O. Box 351040, Miami, FL 33135 (800) 443-9353.

March 25. **Tubular Sections in Building Construction** (co-sponsored by AISC and VCSSFA) breakfast meeting in Columbia, SC (see March 24 listing).

March 26. **Tubular Sections in Building Construction** (co-sponsored by AISC and VCSSFA) breakfast meeting in Charlotte, NC (see March 24 listing).

March 27. **Tubular Sections in Building Construction** (co-sponsored by AISC and VCSSFA) breakfast meeting in Greensboro, NC (see March 24 listing).

March 31. **Bolting Update** (co-sponsored by AISC and SASF) breakfast meeting in Jacksonville, FL. 45 minute description of changes since the issuance of the 1985 High-Strength Bolt Spec. Also includes a review of installation methods for high-strength A325 and A490 bolts.

April 1. **Structural Vibrations**, Lehigh University, Bethlehem, PA. Full-day course includes presentations on vibrations in bridges as well as Thomas Murray's T.R. Higgins Lecture on floor vibrations. Contact: Indra Ghosh, BASE Engineering Inc., 1044 N. Quebec St., Allentown, PA 18013 (215) 437-0978.

April 1. **Bolting Update** (co-sponsored by AISC and SASF) breakfast meeting in Orlando (see March 31 listing).

April 2. **Bolting Update** (co-sponsored by AISC and SASF) breakfast meeting in Tampa (see March 31 listing).

April 6-7. **Welding Structural Design** two-day seminar, Detroit. Contact: AWS, 550 N.w. LeJeune Road, P.O. Box 351040, Miami, FL 33135 (800) 443-9353.

April 6-8. **Structural Stability Research Council** (SSRC) Annual Meeting, Pittsburgh. Sessions will revolve around earthquake stability problems in Eastern North America. Contact: Lesleigh G. Federinic, SSRC, Fritz Engineering Laboratory #13, Lehigh University, Bethlehem, PA 18015 (215) 758-3522 or Prof. Reidar Bjorhovde (412) 624-9876.

April 8. **Tubular Sections in Building Construction** (co-sponsored by AISC and VCSSFA) breakfast meeting in Norfolk, VA (see March 24 listing).

April 9. **Tubular Sections in Building Construction** (co-sponsored by AISC and VCSSFA) breakfast meeting in Richmond, VA (see March 24 listing).

April 9. **Southeast Steel Bridge Forum**, New Orleans. Contact: Camille Rubeiz, Steel Bridge Forum, c/o AISI, 1101 17th St., N.W., Suite 1300, Washington, DC 20036 (202) 452-7190.

April 12-16. **ASCE Structures Congress**, San Antonio, TX. Four plenary sessions and more than 70 technical sessions. Contact: American Society of Civil Engineers, 345 East 47th St., New York, NY 10017 (800) 548-ASCE; Prof. James R. Morgan (409) 845-4394.

People In The News

Peter Sanderson has been named the new president of AISC-member American Bridge Co., Pittsburgh. Sanderson, a professional engineer in Canada, formerly was vice president and general manager of the Heavy Civil Construction Division, PCL Civil Contractors and PCL Civil Constructors (PCL Group of Companies in Edmonton, Canada).

Carl Pete Watson, past president of AISC-member Louisville Bridge & Iron, Inc., passed away late last year.

Steel News

Metric Gains Momentum

The move towards metrication within the construction industry is expected to gain ground this year when several federal agencies institute metric pilot programs. Each federal agency has been instructed to begin pilot programs during the next two years as the first step in a five-year plan to convert the design of all new federal facilities to metric.

And if all goes according to plan, the move by the federal government towards metric for construction will provide an impetus for the private sector to also convert.

Initially, federal agencies are expected to use a "soft" metric system before eventually moving to a "hard" metric system. Soft metric is simply repesenting U.S. Customary Units in its exact metric equivalent. Hard metric is when the physical size has been changed to produce a nice round number for ease of discussion and to make remembering the size easier. For example, with soft metric, a 4' x 8' wall panel would be 1219 mm x 2438 mm; in hard metric, it would be 1200 mm x 2400 mm. For manufacturers wanting to sell products overseas, the hard metric size is far more useful, while for manufacturers matching an existing U.S. product line, soft metric is important.

> The General Services Administration leads the pack and has already selected several pilot projects

The General Services Administration leads the pack and has already selected several pilot projects, including the Southeast Federal Center in Washington, DC. Also, all new major projects in the Philadelphia region will be designed in metric.

To facilitate the use of fabricated structural steel, AISC has prepared drafts of two documents based on the SI (System International) units of measurement: *Metric Properties of Structural Shapes with Dimensions According to ASTM A6M* and a metric conversion of the *1986 LRFD Specification for Structural Steel Buildings*. For more information, call AISC at (312) 670-2400 ext. 433.

Also, AISC is offering a metric software package: *AISC Metric Database for Structural Shapes*. The $60 software package contains properties and dimensions of structural shapes corresponding to data published in Part 1 of the 1st Edition, AISC *LRFD Manual of Steel Construction* as well as the 9th Edition, AISC *Manual of Steel Construction*. Included is a data base in ASCII format, an explanation of the variables specified in each of the data fields, and a listing of a BASIC read/write program and sample search routine. For information, call AISC Software at (312) 670-5434.

A new AISC software package contains properties and dimensions of structural shapes for both ASD and LRFD

Other federal agencies involved in metric design include:

- Federal Highway Administration. FHWA will work closely with AASHTO in implementing metric through the state grant programs. The agency will begin publishing documents in metric and procuring projects in soft metric in October 1992, procuring projects in hard metric in October 1994, and complete metric implementation in October 1996.
- Forest Service. Document conversion process began in January and FHWA conversion timetables will be followed for the metrication of roads, bridges and recreation areas.
- Department of Agriculture. Document conversion process began in January and one $400,000 pilot project is now in design.
- Department of State, Foreign Buildings Office. Officially "went metric" last October, and all new work is being designed in metric.
- National Science Foundation. Since January 1991, all research and education proposals have been required to use metric units. Metric modifications of NSF's *Grant Policy Manual* and *Grant General Conditions* have recently been published.
- Tennessee Valley Authority. Much of TVA's equipment is already metric and suppliers are now being notified of procedures for implementation of metric by October. Also, pilot projects using pre-engineered buildings are beginning.
- Public Health Service. Six small pilot projects are in the planning stage.
- Coast Guard. A station was designed in metric in 1989 and its design is being exported. Com-

plete metrication of the Coast Guard is expected by October 1997.

- NASA. It's expected that the agency will set a goal of January 1994 for the design of all construction in metric.
- Secretary of Defense, Washington Services. Currently setting up pilot projects.
- Corps of Engineers. Currently selecting metric pilot projects.

Metric Conversions

Although there are seven metric base units in the SI system, only four are currently using in AISC publications. The base units being used by AISC and their symbols are listed in Table 1. Note that proper upper and lower case is important in the metric system. For example, a lower case "m" denotes "meter", while an upper case "M" denotes "mega".

Of the numerous decimal prefixes included in the SI system, only three are used by AISC and these are listed in Table 2.

Although specified in SI, the pascal is not universally adopted as the unit of stress. Because section properties are expressed in millimeters, it is more convenient to express stress in newtons per square millimeter (1 N/mm^2 = 1 MPa). This is the practice followed in recent international structural design standards, including 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*. It should be noted that the joule, as the unit of energy, is used to express energy absorption requirements for impact tests. Moments are expressed in terms of N-mm.

The derived metric units for force, stress and energy are given in Table 3.

Table 4 contains the conversion factors to relate traditional U.S. units of measurement to the corresponding SI units. Note that fractions resulting from metric conversion should be rounded to whole millimeters (Table 5).

Bolt diameters are taken directly

Table 1: Metric Base Units

Quantity	Unit	Symbol
length	meter	m
mass	kilogram	kg
time	second	s
temperature	celsius	C

Table 2: SI System Decimal Prefixes

Prefix	Symbol	Order of Magnitude	Expression
mega	M	10^6	1,000,000 (one million)
kilo	k	10^3	1,000 (one thousand)
milli	m	10^{-3}	0.001 (one thousandth)

Table 3: Derived Metric Units

Quantity	Name	Symbol	Expression
force	newton	N	$N = kg - m/s^2$
stress	pascal	Pa	$Pa = N/m^2$
energy	joule	J	$J = N - m$

Table 4: Conversion Of Traditional U.S. Units To Metric

Multiply:	By:	To Obtain:
inch (in)	25.4	millimeters (mm)
foot (ft)	305	millimeters (mm)
pound-mass (lb)	0.454	kilogram (kg)
pound-force (lbf)	4.448	newton (N)
ksi	6.895	N/mm^2
ft-lbf	1.356	joule (J)

Table 5: Common Fractions And Their Metric Equivalent

Fraction (in)	Exact Conversion (mm)	Rounded Conversion (mm)
1/16	1.6	2
1/8	3.175	3
3/16	4.7625	5
1/4	6.35	6
5/16	7.9375	8
3/8	9.525	10
7/16	11.1125	11
1/2	12.7	13
5/8	15.875	16
3/4	19.05	19
7/8	22.225	22
1	25.4	25

from the ASTM Specifications A325M and A490M rather than converting the diameters of bolts dimensioned in inches. Table 6 contains the bolt designations and their traditional U.S. and metric sizes.

The yield strengths of structural steels covered in the metric LRFD Specification are taken from the metric ASTM Specifications. It should be noted that the yield points are slightly different from the traditional values, and these are shown in Table 7.

More complete information is available in the *Metric Guide for Federal Construction, First Edition*. For more information call the National Institute of Building Sciences at (202) 289-7800.

Table 6: Metric Bolt Designations

Designation	Diameter, mm	Diameter, in
M16	16	0.63
M20	20	0.79
M24	24	0.94
M27	27	1.06
M30	30	1.18
M36	36	1.42

Table 7: Bolt Yield Points

ASTM Designation	Yield Stress, N/mm^2	Yield Stress, ksi
A36M	250	36.26
A572M Gr. 345, A588M	345	50.04
A852M	485	70.34
A514M	690	100.07

New Software For Connections

Designing strong axis, beam-to-column moment connections is simplified with a new software program marketed by AISC.

CONXPRT Module II—Moment Connections, a follow-up to Module I—Simple Connections, provides suggested designs for four types of moment connections to a column flange: direct welded; flange welded web bolted; flange-plate and web bolted; and flange-plate welded. Both the flange and web parts of the connection are designed in accordance with the 1989 Allowable Stress Design AISC Specification for Structural Steel Buildings.

Connections on the column side, including column tops, can be designed using this software. One or two-sided joints can be specified, with or without elevation offsets, and the required sizes of any column web stiffener plate, doubler plate, and weld are determined.

All CONXPRT Module II designs satisfy AISC specification, clearance and traditional fabricator requirements. Default tables allow the user to customize designs for individual shop practice or for a particular job. A full report is available for each connection. When a design cannot be completed, a detailed explanation, including references, is provided.

Module II costs $400, and includes a one to three user copy of DESIGN ADVISOR, an expert system for the structural engineer, while Module I costs $300. For further information, call AISC Software at (312) 670-5434.

Upgrading Steel Truss Bridges

Using innovative techniques to rehabilitate and upgrade the existing inventory of steel truss bridges could save billions of dollars compared to replacement costs

By Nils Olsson, P.E., and Hannskarl Bandel, Dr. Ing., P.E.

Among the existing inventory of 550,000 bridges in the U.S. are 28,000 major, steel truss bridges. The National Steel Bridge Inventory Service (NBIS) currently lists 173 structurally deficient and another 141 functionally obsolete structures of this type with spans in excess of 300′ all built prior to 1972.

An educated guess is that the replacement cost of each major span would cost at least $30 million—or a total of $9.4 billion for all 314 bridges. Juxtapose these 314 bridges described with the total number of 28,000 major steel truss bridges and the figure becomes astronomical. But billions of dollars could be saved by encouraging designers to develop innovative solutions for extending the life and capacity of existing steel bridges instead of relying on replacement.

Demonstration Project

A study is now underway to upgrade, rather than replace, the 40-year-old Mathews Bridge (Arlington Bridge) in Jacksonville, FL.

Currently operating at nearly 60,000 vehicles per day, this four-lane steel cantilever thru truss bridge over the St. Johns River is a prime candidate for replacement. The 7,375′-long bridge has high level approaches to the 810′ main span providing a minimum vertical clearance of 149.5′ above the navigable channel and returning to normal grade over a spoil island and connecting with a causeway leading to the riverbank.

Further impacting this overburdened urban structure are new traffic patterns created by the recently opened six-lane Dames Point Bridge to the near northeast and the new six-lane Acosta Bridge, currently under construction, just three miles to the west. These factors, among others, are dictating an eight-lane replacement program for the Mathews Bridge.

Estimates were for a ten-year program costing as much as $200 million. But in searching for a less costly solution, the Florida DOT commissioned a study by DRC Consultants (New York/Tampa) to explore the possibility of increasing the existing four-lane traffic capacity to eight lanes, maintain traffic on the existing alignment, and utilize the existing substructure and foundations.

Innovative Solutions

To double the main span capacity, the engineers proposed an external arch and the utilization of a new four-way lightweight open grid steel deck system.

The new arch would rise above the 810′ main span truss and return to bear on the tops of reinforced kingposts at each main pier. With

Figure 1

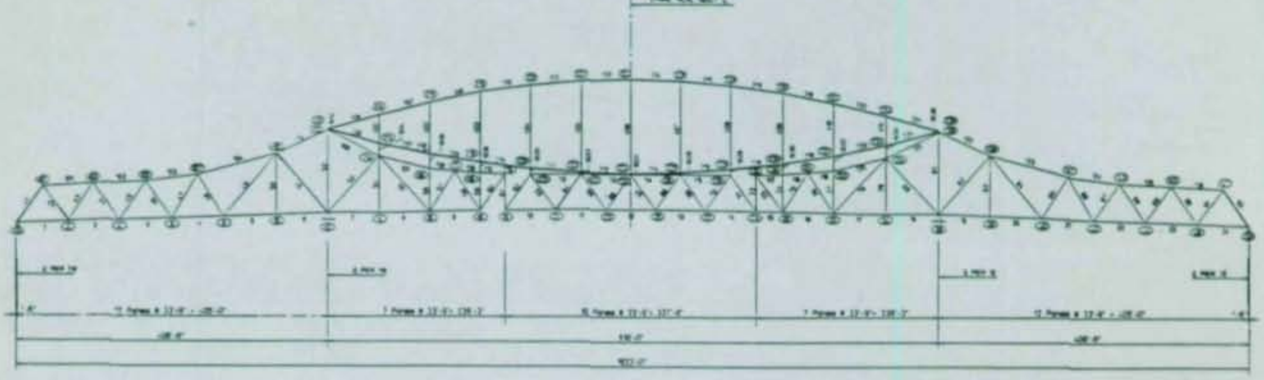
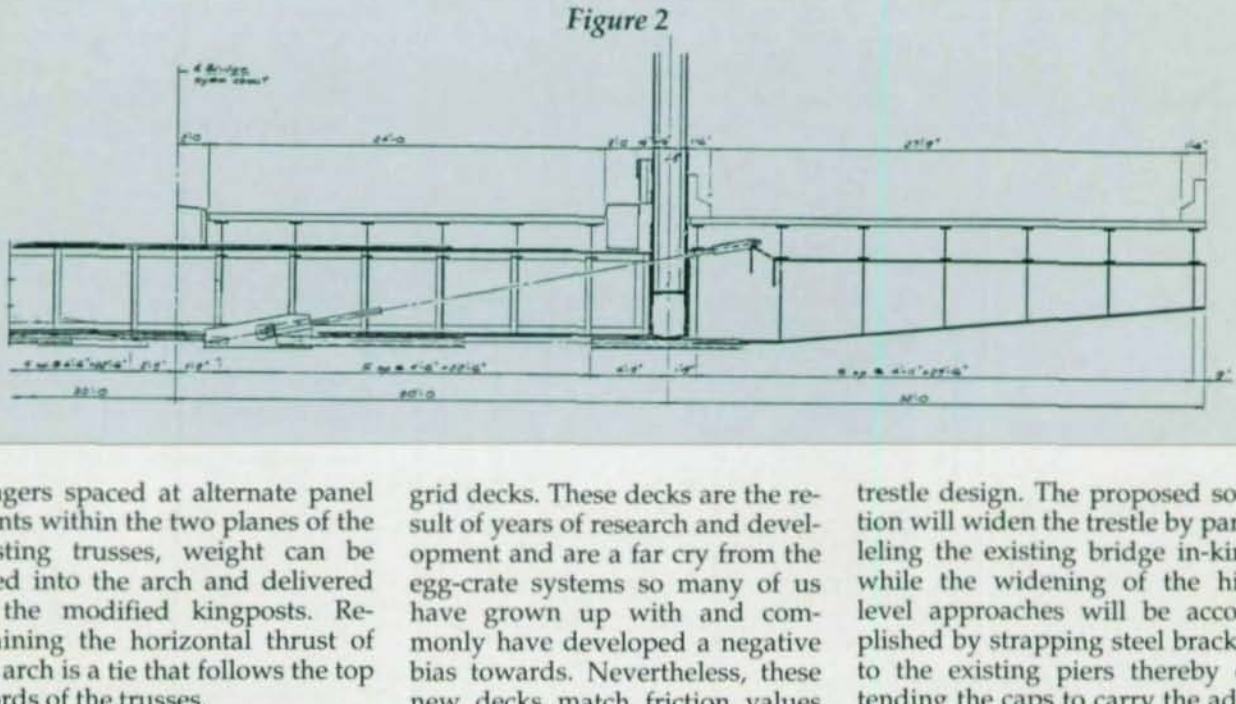


Figure 2



hangers spaced at alternate panel points within the two planes of the existing trusses, weight can be lifted into the arch and delivered to the modified kingposts. Restraining the horizontal thrust of the arch is a tie that follows the top chords of the trusses.

The existing 810′ main span deck is the "traditional egg-crate" open grid deck. The side spans have 8"-thick, normal weight, noncomposite concrete decks that when replaced with all new four-way open grid steel deck will impose a deadload of only one-fifth that of the original deck. The new outboard main span roadways will either be suspended or cantilevered from the existing deck girders.

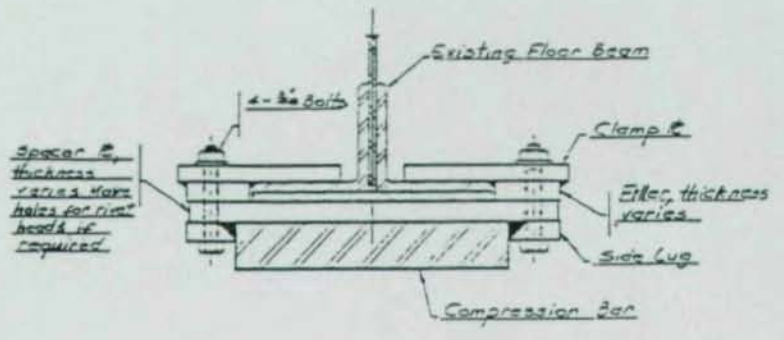
The plan is to replace all concrete roadway on the trusses with new, four-way lightweight open grid decks. These decks are the result of years of research and development and are a far cry from the egg-crate systems so many of us have grown up with and commonly have developed a negative bias towards. Nevertheless, these new decks match friction values for both concrete and asphalt while eliminating dangerous planing, have minimized tracking characteristics, and noise levels are comparable to solid decks. In addition, fatigue life has been extended manifold. All of these factors result in a superior roadway system compared with a traditional deck.

Proposed Construction

The existing 7,375′-long structure is composed of three basic construction types: 2,600′ of thru-trusses, 3,700′ of steel girders, and the balance of length utilizing a trestle design. The proposed solution will widen the trestle by paralleling the existing bridge in-kind, while the widening of the high level approaches will be accomplished by strapping steel brackets to the existing piers thereby extending the caps to carry the additional steel-framed roadway. The existing piers and foundations have available reserve capacity to carry the concrete decks and added steel framing without additional foundation modification.

Because the existing heavy concrete deck will be replaced with the lightweight four-way open grid system, the existing steel trusses can support the two embracing cantilevers carrying the added four lanes of roadway. However, the 810′ main span already has the lightweight open grid deck, and therefore other

Figure 3



measures are required to support the additional lanes in this span.

The proposed construction will enhance the load carrying capacity of the main span by the superposition of arches from kingpost-to-kingpost (Figure 1). The attachment of cantilevers to the truss floor beams present two cases: one for the replacement of the concrete decks and the other for keeping the lightweight steel deck.

Figure 2 shows the attachment of these members at the main span. To avoid the existing gusset plate connections between truss diagonals, verticals and bottom chords, the new flange plates of the floor beams—which carry the cantilever moments—are extended to the center of the existing floor beams. These center connections are only active for non-symmetric, transverse, traffic loads that create, in this case, a positive and negative moment in the near and far floor beam halves.

The stepped floor beam flange plates are able to take these additional loads if the symmetric movement of the existing floor beams is reduced by the addition of uniform post-tensioning of the bottom flange. This arrangement requires a minimum attachment to the riveted floor beams and does not affect any existing floor beam connections.

All connections to the existing bridge try to avoid the removal of rivets by providing clamped connections (Figure 3) and using existing rivet holes for bolts. The four-way, lightweight open grid decks bear on the stringer beams of the outboard roadways and on the existing stringers between the existing trusses.

The cantilever beam connections at the truss floor beams, which originally carried concrete deck slabs, have excessive depth when the light steel deck replaces the concrete. Consequently, the deeper beams' excess moment capacity obviates the necessity of post-tensioning in these areas. The arches are completely independent of the trusses, resting on separate sliding bearings. The compression arch is a box section. Vertical hangers are suspended from the arch and attached to the truss at alternate panel points (Figure 4). These hangers are adjustable permitting the controlled transfer of loads acting on the existing trusses thru the hangers to the arch. The horizontal ties consist of four 5"-diameter strands having a catenary that closely follows the top chords of the existing major main span trusses.

Shown earlier in this article is the computer model of the combined system, superimposing the arch upon the cantilever main span truss. The transfer of load from the truss to the arch is limited

Figure 4

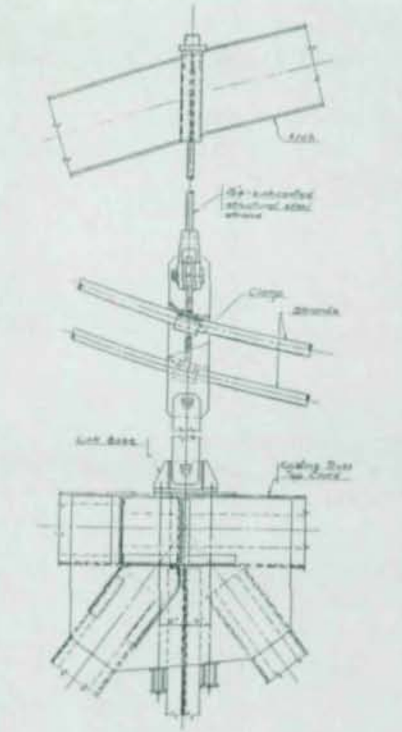
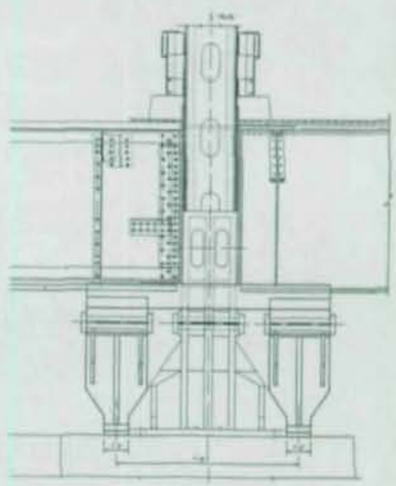


Figure 5



because the lifting introduces a new vertical moment into the side spans, which increases the stresses in these areas. However, by simultaneously activating the arch, new forces are created at the main pier truss elements that are reinforced, but additionally, necessitate the increase in capacity of the main bearings, which have been working very close to their original design limit (Figure 5). The vertical elements that now carry the cantilever dead and live load are reinforced.

Estimates are that this design will cost $67 million, rather than the $200 million estimated for replacing the bridge. If a similar savings is applied to the other 213 truss bridges needing replacement, the savings would exceed $2.3 billion. And that doesn't even consider the other 28,000 existing steel truss bridges.

Needed Innovations

And steel truss bridges represent only a fraction of the total inventory of 550,000 bridges. Savings are there for the taking if available intellectual resources are unleased and encouraged by the government. But first, the design community must be freed from the need to practice business as usual: the engineering-construction dichotomy; claims litigation; adversarial client relations; and the typical owner's attitude of "first show me one that has already been done."

Designers must be encouraged to develop new methods of saving money on infrastructure repair—and clients cannot turn down these innovations solely on the basis of their newness.

The steel truss bridge solution discussed above is being entertained by an enlightened state DOT. If the FHWA will buy into the concept, our economy will quickly realize an improvement in infrastructure costs.

Nils D. Olsson, P.E., and Hannskarl Bandel, Dr. Ing., P.E., are with DRC Consultants, Inc., in Tampa, Fl, and in New York City, respectively. ■

AN AIRTIGHT CASE FOR LOWERING COSTS

The new 27-story county courthouse building in Columbus, Ohio.

Historic Expansion

The conversion of a former museum to office space required the addition of mezzanines to increase the occupiable space and improve acoustics

During the conversion from museum to office space, a mezzanine was inserted to provide an additional 20,000 sq. ft. of office space. Photo above by Jeff Goldberg/ESTO Photographics, Mamaroneck, NY; right photo courtesy of Eihnorn Yaffee Prescott, P.C.

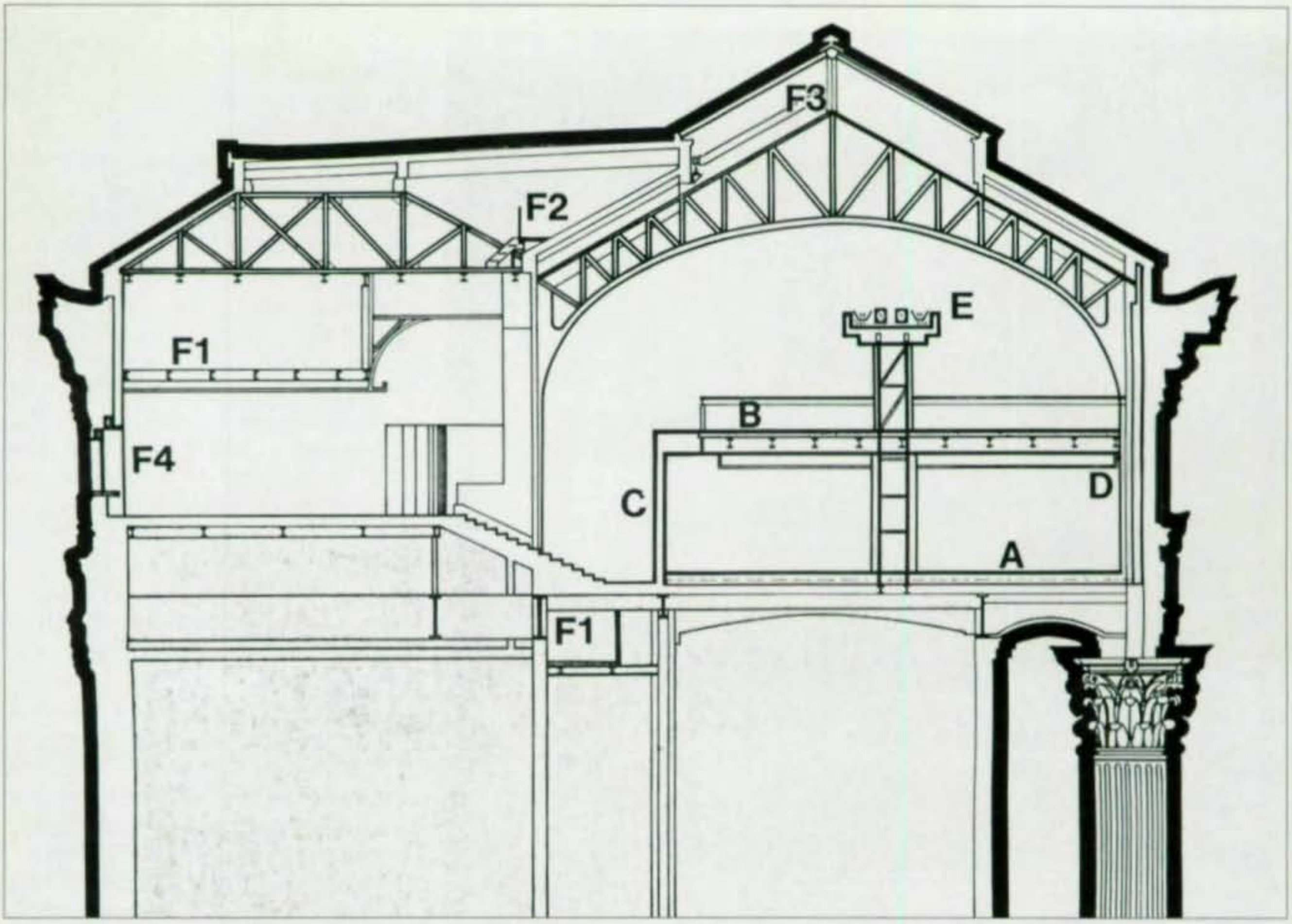
A voluminous, vacant museum might seem an odd choice for modern offices, but the successful conversion of the structure's fifth floor into the New York State Education Department's new headquarters proved otherwise.

The landmark Albany, NY, building, which for most of its life housed the State Library on its second floor and the State Museum on its fifth floor, as well as State education offices on its other floors, dates back to 1908. The five-story, steel-framed, marble-clad structure is distinguished by a 520'-long, load-bearing Corinthian colonnade—reportedly the world's largest.

In the mid-1970s, the Library and Museum were relocated to the a new Cultural Education Center, leaving much of the building empty. More than a decade later, it was decided to transform the imposing space into additional offices for the State Education Department.

Now complete is the renovation of the fifth floor—which was completed while the rest of the building was still occupied.

The existing building construction is a steel frame with concrete floor slabs. Interior wall and ceiling finishes are plaster with ornamental trim—some of which was treated to look like stone. The east and west wings are similar in character, featuring plaster walls of blind arches with exposed steel trusses and a continuous 560'-long skylight. The north wing is more elaborate, with highly decorated columns and ribs supporting plaster vaults, surmounted by skylit



A: The existing structure did not have the capacity to support the new mezzanines. Reinforcing the existing structure was prohibitively expensive. The solution was to remove heavy, non-structural concrete floor topping and replace it with a light weight, raised floor system. This saved the mezzanine design concept and cut costs by $1.1 million.

B: The large cantilevered floors of the curved mezzanines posed another problem—the tendency to vibrate. The traditional solution of thicker floors was eliminated due to weight limitations. Putting the computer to work, the structural engineer "dynamically tuned" the mezzanines by adding weight only in critical locations.

C: Because the mezzanines extend over areas without columns in the floor below, support was provided by a series of short outriggers extending from the mezzanines to the building's columns below. These slender unbraced outriggers evolved from a purely structural funtion into a significant architectural feature.

D: Attaching new steel construction to existing steel columns, while maintaining cantilevered mezzanine framing and respecting the historic nature of the monumental structure, was solved by using a combination connection consisting of high-strength bolts and welding compatible with both the old and new steel.

E: The necessity to conceal mechanical and electrical services was a challenge since there was insufficient depth between the mezzanine framing and ceiling for the traditional approach. Instead, overhead troughs were designed using light-gage steel trusses cantilevered at the ends, keeping weight to a minimum.

F: Other challenges included:

1. Shallow light weight mechanical floors in tight areas with lmited structural capacity;

2. An intricate and extensive catwalk system threaded throughout the congested attic;

3. Inconspicuous, functional sliding scaffolding to service the reclaimed skylight;

4. Historically sensitive lintels for windows.

The new mezzanines are cantilevered off of tube columns that were attached to the structure's original steel columns. Extending from the steel tubes are short outriggers. Photo above by Jeff Goldberg/ESTO Photographics, Mamaroneck, NY; right photo courtesy of Eihnorn Yaffee Prescott, P.C.

oculi.

But after many years of neglect, the space was seriously in need of repair. "Decorative plaster walls, moldings, and trim had badly deteriorated due to years of neglect and moisture penetration and were further damaged during the removal of the extensive museum displays," explained Steven Einhorn, AIA, principal with Einhorn Yaffee Prescott, Albany, the project's renovation architects and M/E/P engineers. "Sections of the continuous skylight in the east/west galleries had been removed, painted or roofed over to reduce the destructive effects of sunlight on the museum displays, to lower the heat load, and to eliminate moisture problems resulting from the deteriorated condition of the skylight."

The renovation plan developed jointly by the New York State Office of General Services and the State Education Department included not only restoring the space to its former glory, but also installing modern utilities and expanding the size of the space by nearly 25%.

"The decision to insert mezzanines into each of the three exhibit wings made it possible to meet the client's space program needs, increasing usable space from 75,000 sq. ft. to 95,000 sq. ft.," Einhorn said. Great care was needed, however, to ensure that the new mezzanines allowed adequate natural light to penetrate to the spaces below as well as had a minimal impact on the original architectural space. The resulting design for the mezzanine is gracefully curved and was derived from the numerous curvilinear forms and arches existing throughout the building. While contemporary in form, the mezzanines work to complement and respond to the building's original architectural features. The mezzanines also act to break up the scale of the cavernous space and improve acoustics.

Structural Considerations

While the mezzanines were needed to meet the client's space needs, they presented an engineering problem since the existing structure did not have adequate capacity to support the new mezzanines.

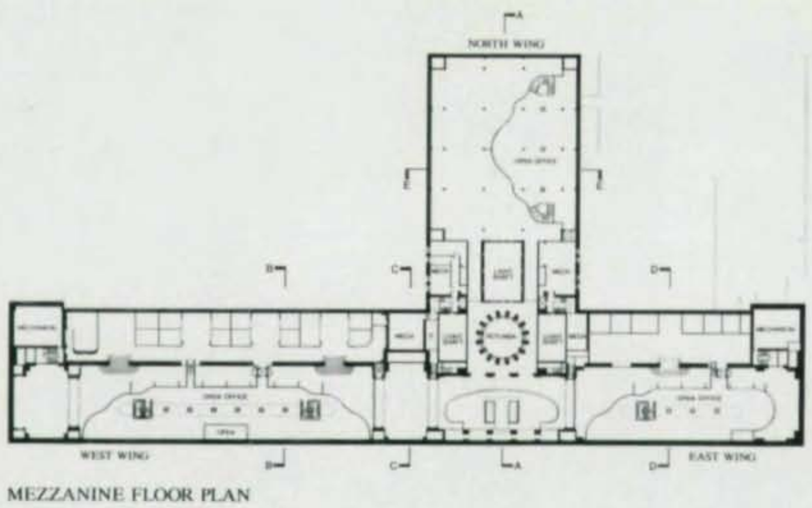
"Reinforcing the existing structure was prohibitively expensive," explained Tom Ryan, P.E., president of Ryan-Biggs Associates, P.C., Troy, NY. However, after exploring several options, the structural engineers developed a workable, practical solution. "By removing portions of the nonstructural 5" concrete floor topping equivalent to the mezzanine loads, the mezzanines could be added." A new, 1'-high lightweight raised floor was installed to replace the concrete topping, and it also served to carry mechanical and electrical systems. General contractor on the project was Sweet Associates, Inc., Schenectady, NY.

Because the mezzanines extend over column-free areas, it was necessary to devise a means of support. Even if a column line from the floor below was extended up to the fifth floor, it still wouldn't be in position to support the mezzanines. The solution was a series of short W18 x 50 outriggers extending from the mezzanines to the newly extended columns. For architectural considerations, the size of the new columns needed to be minimized, so 4" x 4" tubes were used.

"Getting the aproximate 300 tons of steel to the fifth floor posed an interesting challenge," explained James A. Stori, P.E., president of AISC-member STS Steel, Inc., Schenectady, the project's fabricator. "Some small items came up the material hoist. However, the bulk of the steel (though beams about 40' long, weighing two tons) was hoisted over the roof and down through an opening in the existing skylight." Erector was Brownell Steel, Schecetady, NY.

The connections of the new columns in the east and west wings to the existing columns was accomplished with a combination connection employing both bolts and welding. In the north wing, the outriggers were attached directly to existing columns in the same fashion. "Extensive coupon testing was done to ensure adequate weld strengths with the existing steel,"

In the north wing, the mezzanine was cantilevered directly from existing columns (above and top). The mechanical trough (left) is supported on a box column formed with tubes in each corner connected with web members to form a truss assembly. Mechanical and electrical services run up the center of the column. Photos courtesy of Ryan-Biggs Associates.



MEZZANINE FLOOR PLAN

Ryan stated.

Since weight was such a crucial consideration, the mezzanines were framed in steel with a composite steel deck with lightweight concrete topping. The mezzanine framing consists of W18 x 50 girders and W12 x 26 beams.

"The large cantilevered floors of the curved mezzanines posed another problem, the tendency to vibrate," he added. The traditional solution of thicker floors couldn't be employed due to the weight restrictions. Instead, working with a computer analysis, the mezzanines were "dynamically tuned" through the addition of weight in critical locations, primarily at the ends of the cantilevers.

The location and necessity to conceal ductwork and other mechanical/electrical services for the mezzanine space posed another problem. "There was insufficient depth between the mezzanine framing and ceiling for the traditional approach," Ryan said. "The architectural design called for mechanical troughs to rise from the mezzanine floor to house air distribution and lighting systems. However, the additional weight seemed to be a problem. The solution was to design the troughs using light gage steel trusses cantilevered at the ends, thus keeping the additional weight to a minimum." Supporting the troughs, which have a substantial cantilever, are new columns extending up from the existing floor structure. The 3' x 3' box columns are formed with 4" x 4" tubes in each corner connected with web members to form a trusswork. "The tubes provide better sectional properties and allowed ductwork to be run up the center of the columns," Ryan explained.

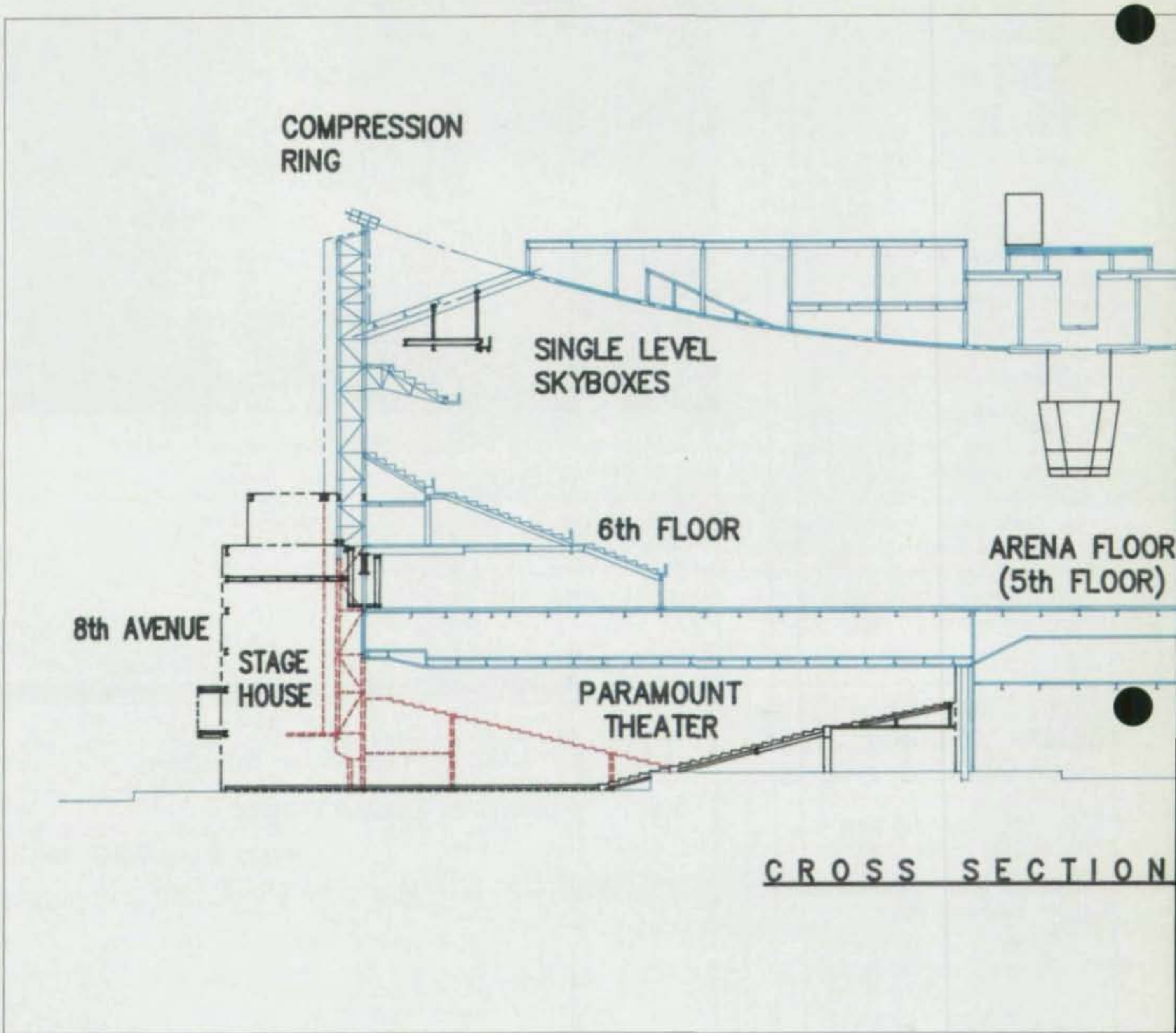
Natural light is admitted to the largely windowless space through rehabilitated skylights, as well as new windows constructed in the north, non-principal elevation. In the east and west wings, several of the blind arches are opened to facilitate access and views to adjacent spaces, while in the north wing, the original windows in the west and north walls were enlarged and the skylights above the vaulted ceilings reactivated.

Financial Success

Based on the cost of leases in privately owned space, it is estimated that the $16.5 million cost of the renovation, which includes a new central mechanical and electrical plant and vertical distribution system for the entire building, will be amortized in 10 years. And the project met the State's desire to retain the historic character of the building, as well as increase its usefulness.

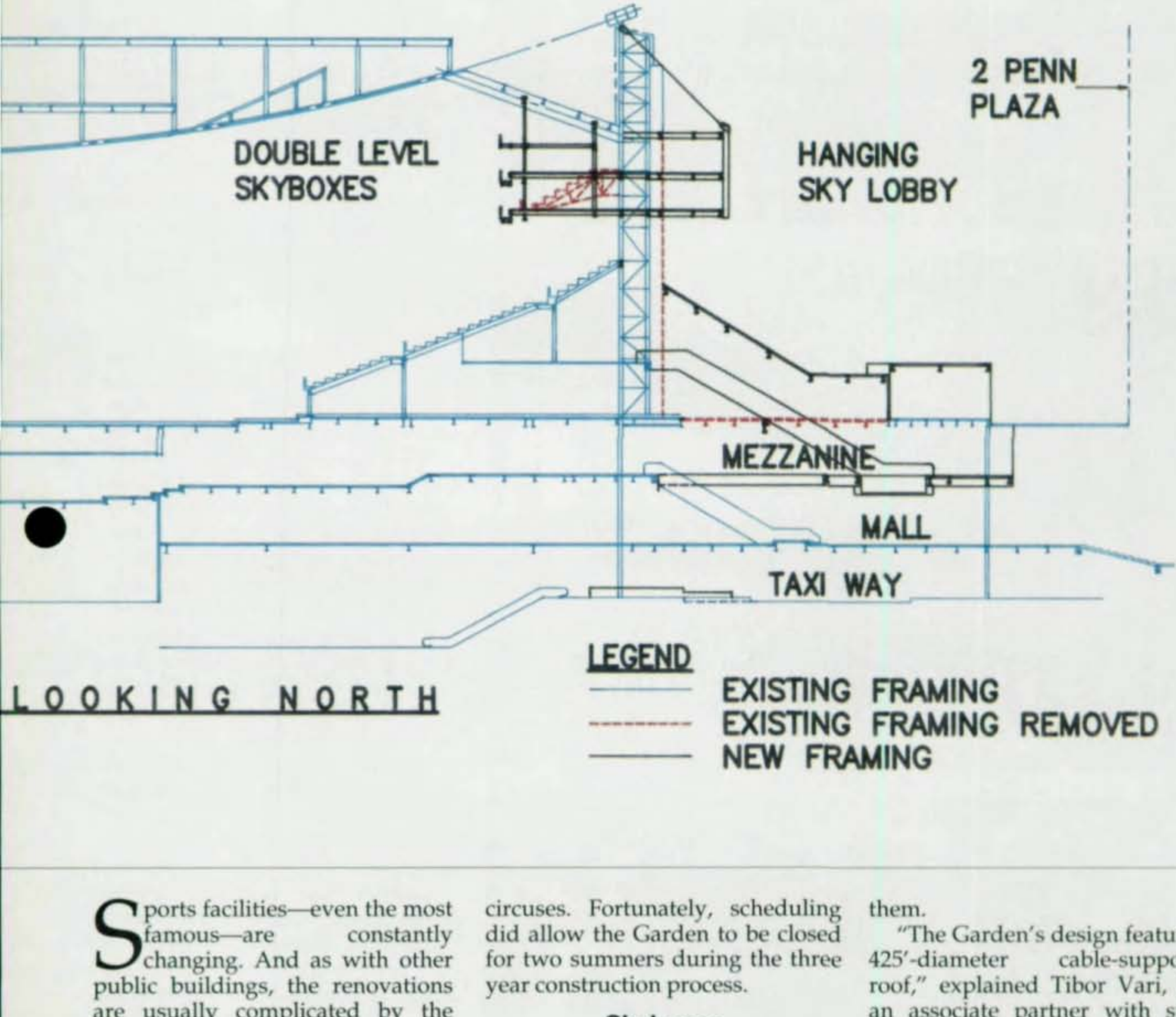
Based on the success of the project, the State is now renovating two other major areas within the building. ■

The exterior of the State Education Building in Albany, NY, features a 520'-long, load-bearing Corinthian colonnade—reputedly the world's largest.



Updating A Sports Institution

The $200 million renovation of Madison Square Garden had to be completed without disrupting the scheduled events



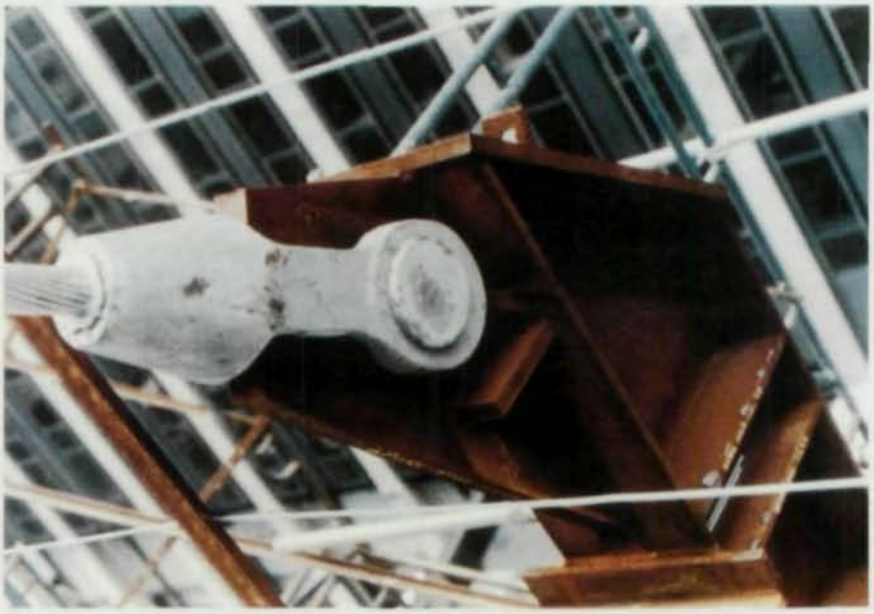
Sports facilities—even the most famous—are constantly changing. And as with other public buildings, the renovations are usually complicated by the need or desire to keep the facility operational during construction.

Madison Square Garden's recently completed $200 million renovation included adding 88 new skyboxes and a new skylobby, constructing a mezzanine over a taxi plaza, and enlarging the old Felt Forum (now called the Paramount Theater)—all without interrupting the Knicks, Rangers, or a variety of special events such as concerts and circuses. Fortunately, scheduling did allow the Garden to be closed for two summers during the three year construction process.

Skyboxes

The toughest part of the renovation was the demolition of 29 existing skyboxes and their replacement with 88 new ones. Support columns, obviously, couldn't be used since they would interfere with the sightlines from other seats. And innovative construction techniques were needed that nothing would interfere with performances or customers views of them.

"The Garden's design features a 425'-diameter cable-supported roof," explained Tibor Vari, P.E., an associate partner with structural engineer Severud Associates, New York City, which also was the original design engineering firm for the structure. "Since this was a new technology at the time of construction, it was designed with a higher safety factor on the cables than is required by today's codes. Current code allows a more realistic safety factor, so a reserve capacity existed in the cables and their supports. In addition, the de-



sign live load of the mechanical room was 75 psf, while the actual load was less than 50 psf, so that gave us an additional load," he added.

"In order to provide column-free space for the new skyboxes, they were hung from the existing cable structure for the roof and suspended mechanical rooms, which were located above the roof cables," Vari said. The hangers were connected either to the roof girders, or, in some cases, to new cross beams that were installed between the roof girders.

The roof girders were reinforced with plates welded to the existing girders to take the additional load. Steel fabricator and erector on the project was AISC-member Canron Construction Corp. (Eastern Division).

Just as complicated as the design of the new skyboxes was their installation. Because work supports couldn't be left in the seating area during performances, the construction team designed a suspended work platform. The platform was suspended at four points from the roof structure, and as many as 17 of these platforms could be joined together as needed. During working hours, the platforms could be lowered into position, and during performances they could be hoisted up to the ceiling and out of view. When construction was complete on one group of skyboxes, the platform was lowered to the floor, wheeled to another location, and hoisted up into place.

Construction managers on the project were HRH Construction Corp. and Herbert Construction Co., both headquartered in Manhattan. Project architect was Ellerbe Becket Sports, Kansas City.

With the addition of the sky-

Pictured at left (from top to bottom): An inside view of Madison Square Garden as the two-level skyboxes are being installed; a perimeter view of the installation of the skylobby; and the cable connection at the skylobby roof.

boxes, the Garden required a new skylobby for arriving suite spectators. "There was no available space inside the existing Garden, so new space was created outside the building, " Vari said. And as with the skyboxes themselves, the lobby was hung from the existing structure.

The garden is clad with precast concrete panels, and the removal of these panels from the effected area created sufficient capacity in the structure's columns to carry the new loads. "Since the new lobbies were too wide to cantilever directly from the existing structure, a cable system was devised, hanging the outside columns and using the roof beams as compression struts," Vari explained.

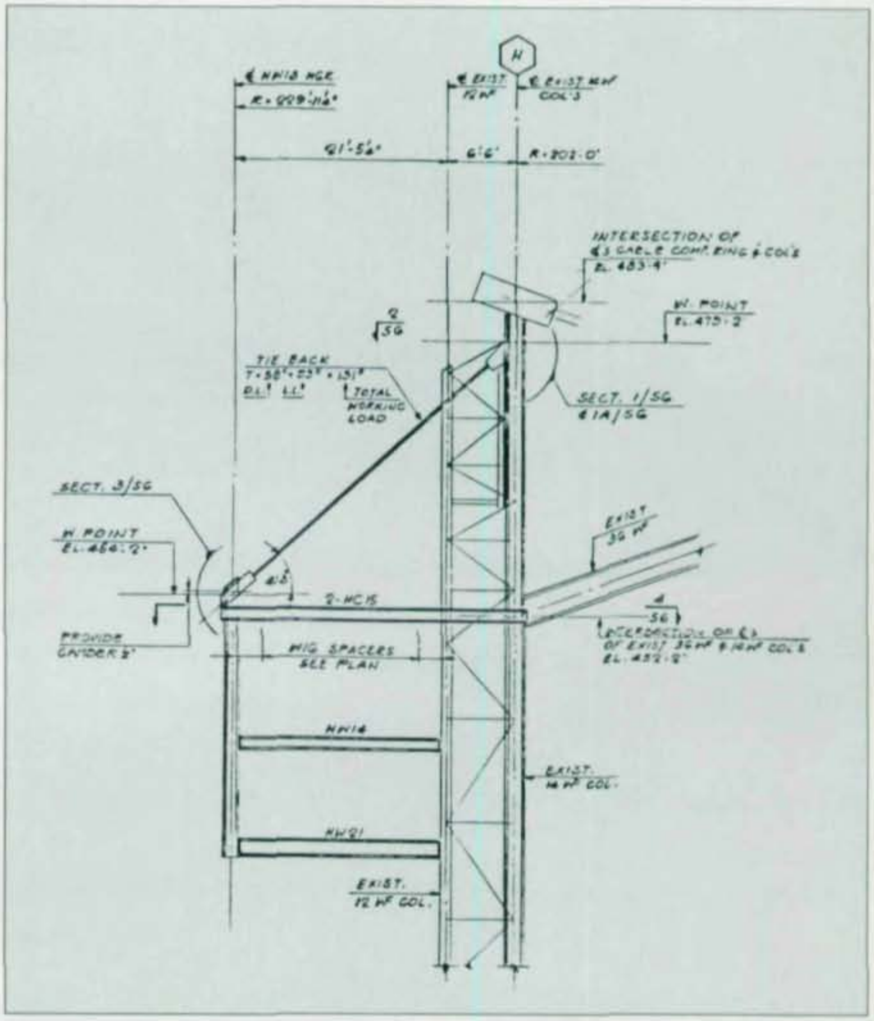
Since the existing structure already employed a cable support system, this solution had the added benefit of matching the existing construction in both style and system.

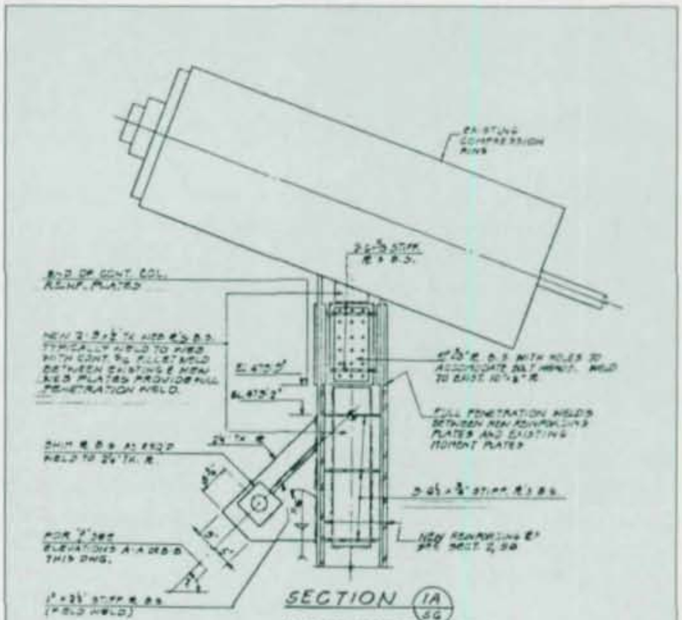
Mezzanine

The new mezzanine was built above a taxi pickup area between the Garden and the adjacent 2 Penn Plaza building. Because new columns in the taxi area would have hampered Amtrak's operations beneath the taxi area, as well as interfering with taxi traffic, the new mezzanine was hung from the existing roof girders, which were reinforced with plates to take the additional load.

Connecting the mezzanine with the skylobbies are two new elevator banks of three elevators each, with the elevator banks serving architecturally to define the ends of the skylobbies. "New columns supporting these elevators land at the taxi roadway level, which is the roof of the computer nerve center of the East Coast corridor of Amtrak," Vari said.

"Running these columns through this area would have required major reinforcement and transfer construction as well as a major disruption of Amtrak's operations. To prevent this, a "pile cap" was built at the taxi street level using three existing Penn Sta-





Shown above and at left are construction details showing the support mechanism for the skylobby. The detail at left shows the existing column splice.

In order to create a full-size stage for the Paramount Theater, an addition was added on the Eighth Ave. side (top). Because the creation of the new stage required the removal of two columns, large trusses were required to pick up the building load. However, the only available location for these trusses was between the fifth and sixth floors (above).

tion columns as the "piles."" These columns date back to the original Penn Station that was demolished in the early 1960s and therefore had plenty of reserve capacity. The cap—a 2'-thick concrete mat—could support the four new elevator columns and serve as the bottom of the elevator pit.

Paramount Theater

The old Felt Forum was located under the Garden floor, on the western portion of the building, with seating stepping down concentrically from the outside towards a low stage at the center of the garden.

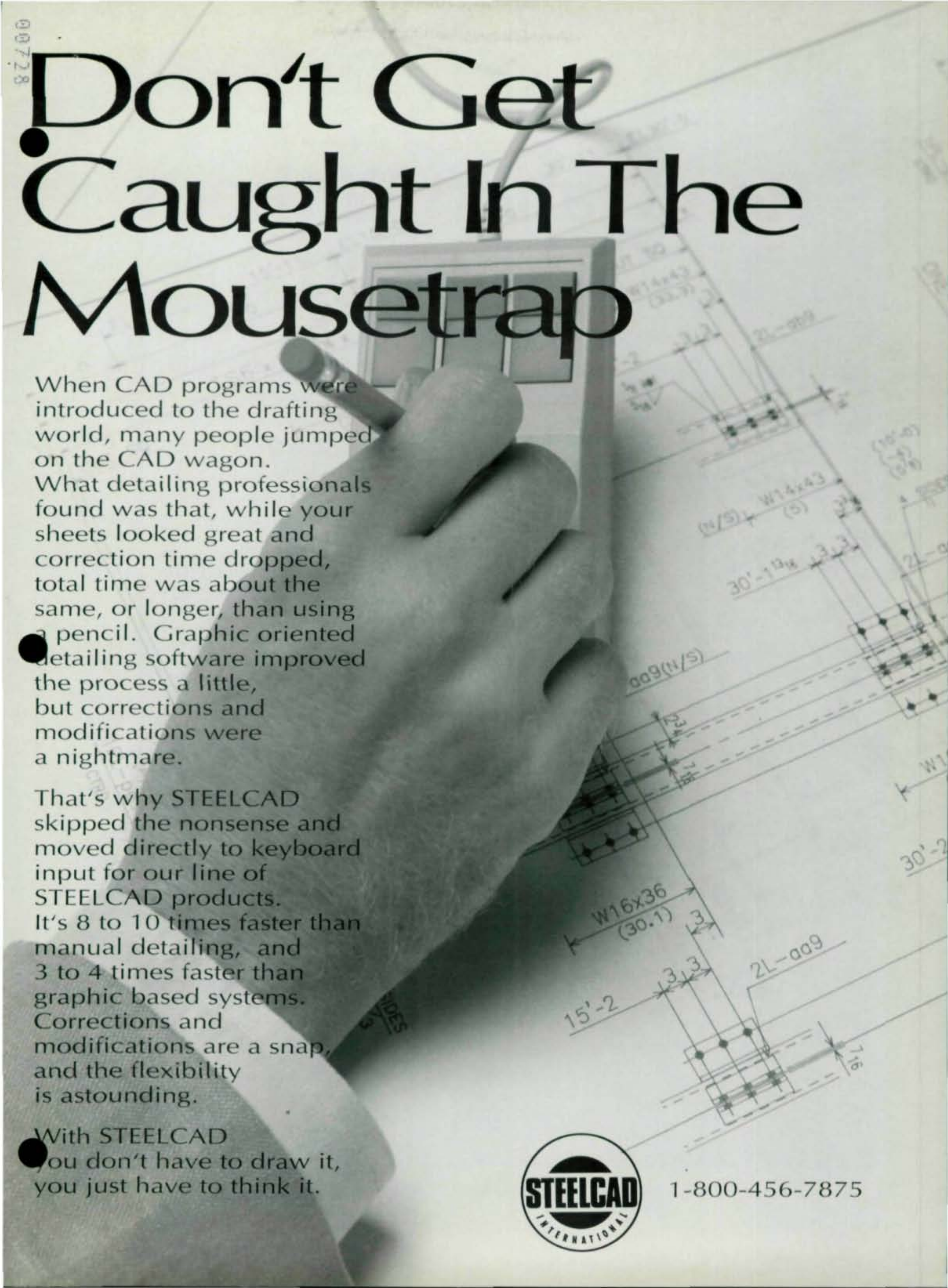
"Because the Garden floor was above the stage, it was impossible to create a full-size stage with fly-space, support space and other necessities for a legitimate theater in the original location," said Fred Severud, P.E., of Severud Associates. The only logical location for the new stage was outside the Garden, on the Eighth Ave. side. However, this required turning the seating 180 degrees. But to make an opening big enough for a 68'-wide stage required the removal of two of the 48 existing main perimeter columns supporting the roof and all of the levels above the street.

"Two trusses were built to carry the 2,600 kips load each column supported," Severud said. One of the 83'-long trusses is 14' deep, while the other is 11' deep. They were installed inside the building perimeter, straddling the columns to be removed. "The adjacent columns, which carry the loads of the trusses, were reinforced with welded plates."

Further complicating the project was that the only location available for the trusses was between the fifth and sixth floor, some 15' above the head of the proscenium. As a result, the portions of the existing columns that remained below the trusses remain as the hangers to support the story-high plate girders that carry the main Garden floor. "The trusses were built and shipped in their final form and attached to the adjacent columns at each end," Severud added. The 1,500 kips of dead load from the columns were jacked onto the trusses before the hangers were cut.

"The placing of the fly-space on the Eighth Avenue side had the additional benefit of providing much needed space on both sides of the new Stage House for offices and support spaces for the stage," Severud said. Columns for these additional spaces were placed on top of existing Penn Station columns in the sidewalk area west of the existing Garden. "This minimized the amount of work below street level in Penn Station."

The entire existing sloping seating frames and concrete steps were demolished and new seating was built using the existing Penn Station columns and girders for support. A new floating concrete floor was placed on vibration isolators, lifting the new floor above the old floor, to attenuate the noise produced by the subway below the new stage. ■


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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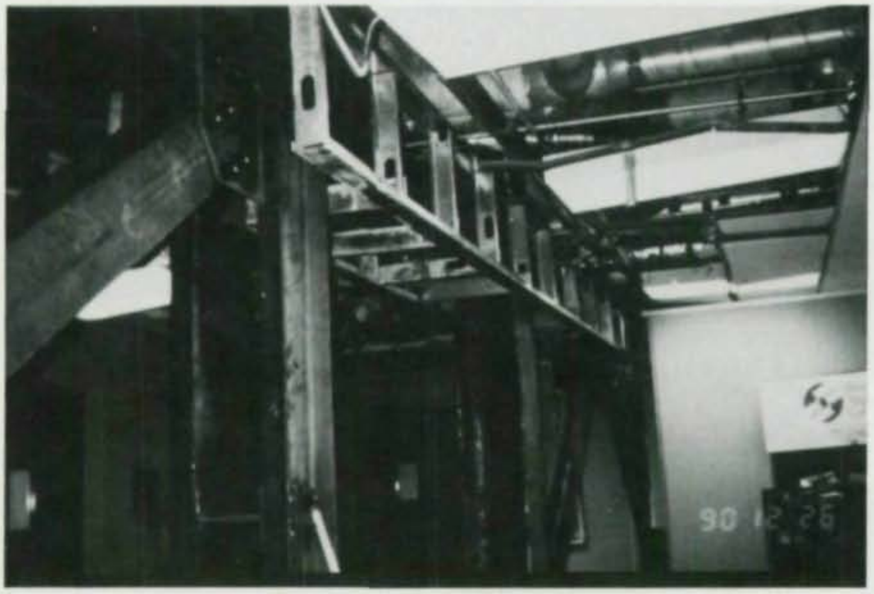
Renovation Without Disruption

A connection between the ninth floor of an existing building and two new buildings needed to be building with minimal worker disturbance



Shown at top are workmen bringing steel into the existing building through a ninth floor window. Bottom photo shows the bulkhead framing. Photos courtesy of Eli Lilly.

When Eli Lilly And Company outgrew its 13-story headquarters building in Indianapolis, the management opted to build two new additions rather than relocate. The additions took the form of two 10-story buildings, structurally independent, though abutting either side of the existing structure.

The existing building was a steel-framed building built in two stages. The first three floors, which were built in the 1950s, are simple-framed, while a ten story addition in the 1960s is moment framed. The end walls of the existing building are clad with a solid sheathing of limestone, which while not designed as a structural element essentially acts as a shear wall.

New Passageway

To facilitate pedestrian traffic between the old building and the new structures, the owner required that a passageway be added at the ninth level in addition to connections at other levels of the buildings. While the renovation project's design engineers, Fink Roberts & Petrie, Indianapolis, determined that an opening could be punched into the end walls of the existing building, they also concluded that vertical bracing was necessary to make up for the reduced shear capacity of the walls and increased lateral deflection of the structure. In addition, bracing was needed to reach the owner's desire for a Seismic Zone 2 compliant structure. This included adding diagonal

Figure One: Original Welded Design

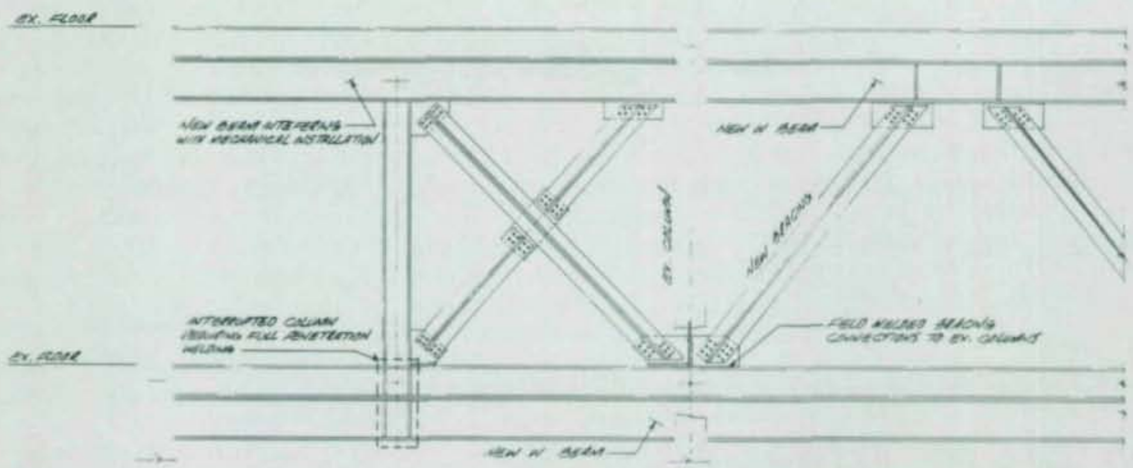
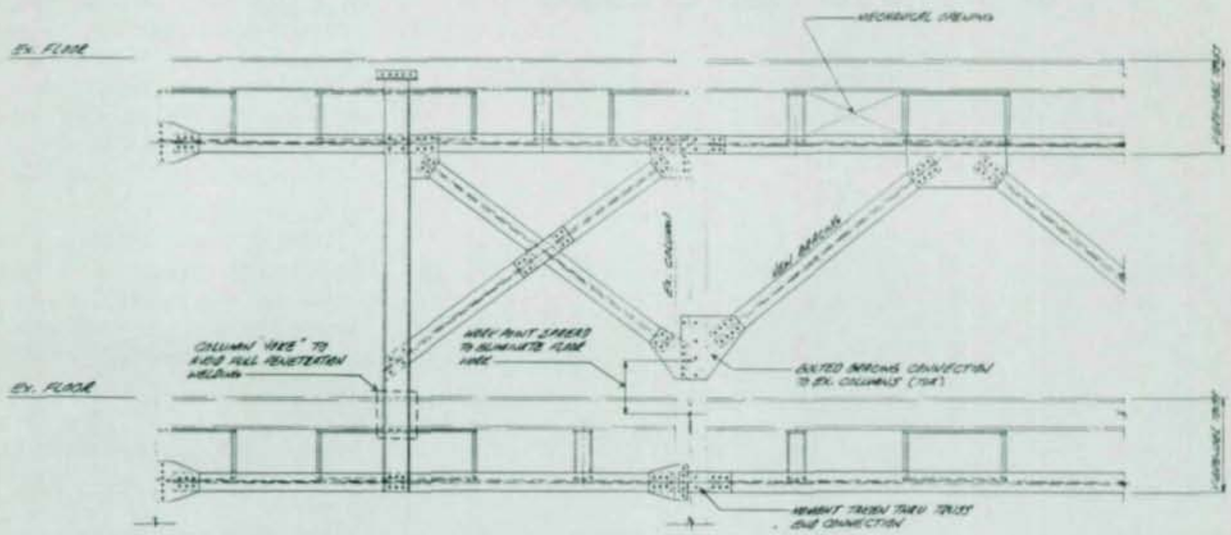


Figure Two: Alternate Vierendeel (Bolted) Design



The top diagram shows the original welded design, while the second diagram shows the bolted design that was developed to meet the owner's requirements and reduce cost. The photo at right shows the truss in place.

bracing between the ninth and tenth floor to form two "hat" trusses the width of the building to increase the resistance to overturning.

While making the ninth level connections was clearly possible, the project's construction manager, Geupel Demars, Indianapolis, initially felt that it would be very complex, time-consuming and expensive due to the tight working conditions, which required cutting the reinforcing members into small pieces, transporting them in the building's main elevators, and then welding them back together.

In addition to its complexity and cost, the original construction plan did not meet the owner's other requirements, which included minimal or no welding by requiring very restrictive and costly smoke and fire protection controls (due to smoke contamination of existing office environment and the possibility of fire) and no interruption or disruption of employee operations.

Geupel Demars called in AISC-member Broad Vogt & Conant, Inc., a Detroit-based structural steel renovation specialist, to assess the situation and recommend construction solutions that would meet all of the owners requirements while also reducing project costs through value engineering. Broad, in turn, retained Ruby & Associates, P.C., Detroit, as a value engineering structural consultant.

Bolted Vierendeel Truss

"The original scheme was generated with diagonal bracing, which delivered large axial forces into the existing floor beams," according to David Ruby, P.E., of Ruby & Associates. "In order for the existing floor beams to sustain these axial forces, the original scheme added significant reinforcing to the beams, which required overhead full penetration welds of reinforcing to the existing column webs; as well as top flange plates welded to the column flange and web."

After a brainstorming session between the entire project team, a consensus was reached that some redesign work was necessary and a new construction plan was needed. The redesign spearheaded by Ruby and Robert Piro, general manager and vice president of proposals with Broad, had two main elements.

First, the alternate design raised the lower work point of the braces at the column centerline to a point above the concrete slab. This eliminated the costly and time-consuming task of removing concrete and relocating the slab embedded electrical and mechanical chases that interfered with the installation of the welded gusset plate to beam flange connection called for in the original design. A computer analysis using the Structural Expert Series from ECOM Associates, Inc., confirmed the additional eccentric moment could betaken by the existing floor beams and columns. Another advantage of this revision was to allow the use of a single wide flange diagonal, instead of four angles, and allow the connections to be made with gusset plates bolted to the flanges of both the columns and diagonals.

Second, the original design called for the installation of an 18"-deep reinforcing beam to be fit beneath the existing ninth and tenth floor beams at each bay with web penetrations required for existing ducts and pipes to pass. This required temporary removal and replacement of portions of the duct work and several other utilities, including fire protection. As an alternate, Ruby devised a plan using a Vierendeel truss (see Figures One and Two).

"This alternative provides a solution that allows the existing structural steel framing to sustain the wind and seismic design loads without direct modifications to the end existing connections," Ruby explained. "We introduced a Vierendeel truss—effectively introducing a 42"-deep beam, to distribute the load. Instead of stability occurring from just column bending and bracing, we introduced frame bending. This also reduced the column length, which reduced story drift and allowed the connections to be primarily bolted instead of welded."

Eliminating Field Welds

The revised construction plan devised by Mark Douglas, an engineer and senior project manager with Broad, Vogt & Conant—which was also chosen as the project's fabricator and erector—made use of a flying jib and conveyor (see photo), allowing for the outside transport of the material in whole pieces, thus eliminating field welded splices.

The Vierendeel truss bottom chord, as well as the 25'-long diagonals, were hoisted, using a flying jib, through windows on the eighth and ninth floors and onto a gravity conveyor system located inside the building. The pieces were then rolled into place and hoisted with chain falls—all of which created little interference to the building's occupants.

Advances In Welding Technology

By David G. Howden, PhD

During the past few years, innovations and advances in welding technology have helped fabricators and erectors cut costs and increase productivity.

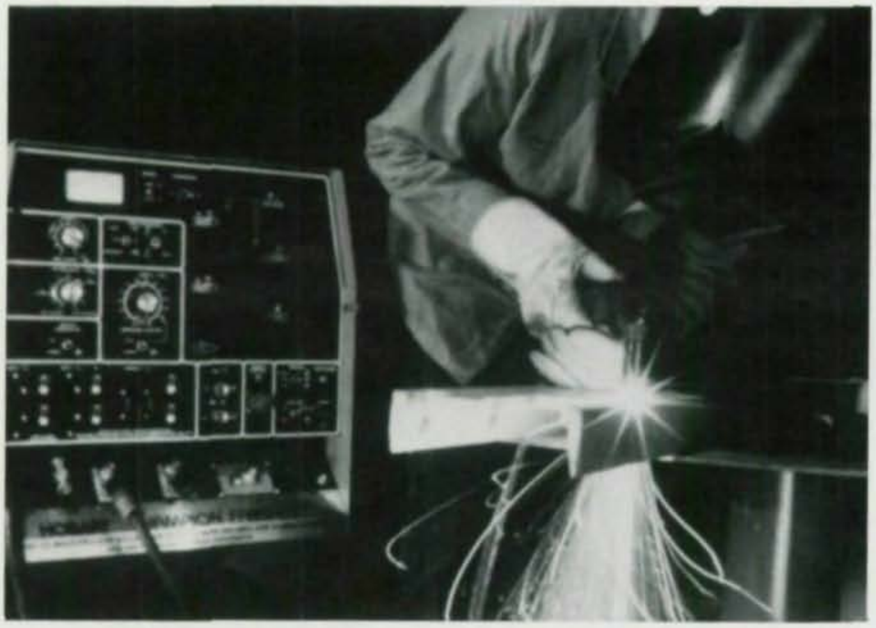
New Power Sources

By their nature, conventional power sources have always been heavy and awkward, as well as somewhat inconsistent in terms of their performance. Ask construction professionals what they want in respect to their welding operations, and you'll undoubtedly hear a desire for smaller, lighter power supplies with better arc performance with more interface features.

The introduction of inverter technology about six years ago delivers on most of these requests. Although inverter power sources are becoming more common, some mystery still surrounds them.

While they appear very complicated, they actually are fairly easy to understand. Inverters are basically electronic equipment that convert raw primary power into manageable power specifically suited to the requirements of the welding arc. They take advantage of the latest innovations in solid state and control electronics—as well as advances in electromagnetic materials—to offer a welding power source unprecedented in the industry.

The major benefits include portability, performance and flexibility.



Inverters offer portability, improved performance and flexibility.

Inverter machines are drastically smaller and lighter than conventional machines of comparable output, allowing easy movement and minimizing floor space requirements. A 600-amp machine typically can be carried by two people from location to location. And some 300-amp machines can be carried by one person. With conventional power sources, a forklift truck or crane is needed to move both 600- and 300-amp machines.

Performance is enhanced by ultra-fast electric circuitry and variable inductance features, which provide good, clean blast-free starts. The quality of arc initiation are greatly affected by the initial rate of rise of current in the electrode, the amount of inductance in the stabilizer, and the response time of the power source. With an inverter, starting is enhanced by rapid but controlled current rise, a minimum of inductance, and circuitry that responds quickly to the drastically and rapidly changing conditions at the output of the machine during starting.

Inverters also provide a large degree of flexibility in that they can be modified to suit essentially all welding processes—including flux cored welding, shielded metal arc welding, plasma cutting, and plasma gouging (however, for stud welding, inverter use is limited to 450 amps). Also, inverters can readily be used on either three-phase or single-phase input power at either 50 or 60 Hz. Because the input is immediately rectified and converted from AC to DC, the type and quality of input power is less critical than with conventional power sources.

The fast response time of inverters operating in the 25 Khz range and the ease with which they can be made to respond makes them ideal for multiprocess machines that ordinarily may compromise performance in or more areas.

Plasma Arc Cutting

While much of the news lately has concerned innovations of laser

By replacing the tungsten electrode with one made with hafnium, the plasma arc cutting torch can be operated efficiently and effectively on structural steel without using expensive inert gases.

welding and cutting, more important for the construction industry are the improvements in plasma arc cutting. While laser cutting is only practical for very thin materials, plasma arc cutting has far fewer restrictions.

An outgrowth of the gas tungsten arc welding process and the plasma arc welding process, plasma arc cutting has been popular for the last 20 years for non-ferrous materials, such as copper alloys, nickel alloys, and aluminum and stainless steel.

Previously, however, the process was limited by a significant disadvantage: a relatively inert gas had to be used for the plasma gas to avoid rapid deterioration of the tungsten electrode and the copper orifice contained within the welding torch. This is particularly true for the addition of oxidizing gases such as oxygen.

In the past five years, though, the plasma arc cutting process has been improved for both shop and field operations—mainly by replacing the tungsten electrode with one made with hafnium. Essentially fulfilling the same job requirements as tungsten, hafnium does not react as rapidly with air. The bottom line then is that the plasma arc cutting torch can be operated efficiently and effectively on structural steel without the use of the more expensive inert or special gases. Further, the process does not use fuel gas (such as acetylene), and therefore the possibility for explosion is reduced.

Advantages of plasma arc cutting include: can cut any metal; cutting speeds are faster than other processes, particularly for thinner sections; dross is easy to remove; safety is improved; use is simplified; and cost per cut is lowered.

The process does have some disadvantages, however, including: higher equipment costs than for other processes; limited metal thickness (up to 1½"); wider kerf (or width of cut); and both sides of kerf are not square, which may necessitate grinding after the plasma cutting process.

David G. Howden, PhD., is associate professor, Department of Welding Engineering, at The Ohio State University in Columbus, OH, and vice president of the American Welding Society. Also contributing to this article was Marv Schiedermayer from the Miller Electric Manufacturing Co., Appleton, WI.

Welding Products

Inverters

PowCon's inverter welding power sources allows up to 75% reductions in the size and weight of the transformer and up to 45% energy savings as compared to SMAW or GTAW power supplies. When combined with an engineered plastic housing, the result is a power source that weights only one-third that of a conventional transformer and takes up significantly less floor space, yet provides the same power output.

PowCon, which also sells plasma cutting tools, has entered into a marketing agreement with Kemppi to sell their product line of inverters in the U.S.

For more information, contact: Wade Chase, PowCon Inc., 8123 Miralani Dr., San Diego, CA 92126 (800) 833-9353.

Inverters

A free bulletin from The Lincoln Electric company describes the features, benefits and applications of the newly introduced Invertec V300-PRO, a multi-process 300 amp arc welding inverter power source that operates on single- or three-phase input power. The inverter features: an efficient, lightweight, compact design and quick disconnect for portability; a process mode selector to modify the arc for five constant voltage or constant current welding processes; arc force and pinch control for fine tuning arc characteristics; a thermostat to prevent overheating and extend life; built-in voltage compensation for weld consistency; and "cold electrode" circuitry for added safety.

Lincoln also manufacturers a line of plasma arc cutting and welding tools.

For more information on inverters, request Bulletin E930 from: The Lincoln Electric Co., 22801 St. Clair Ave., Cleveland, OH 44117-1199 (216) 481-8100.

Stud Welding

TRW Nelson Stud Welding Division offers a wide range of stud welded shear connectors, concrete anchors and threaded fasteners for bridge, building and industrial structures connection requirements. Complete stud welding systems and power generators are available for sales or lease.

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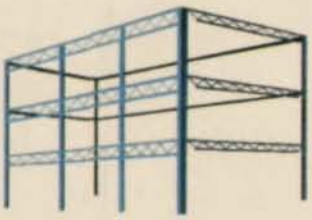

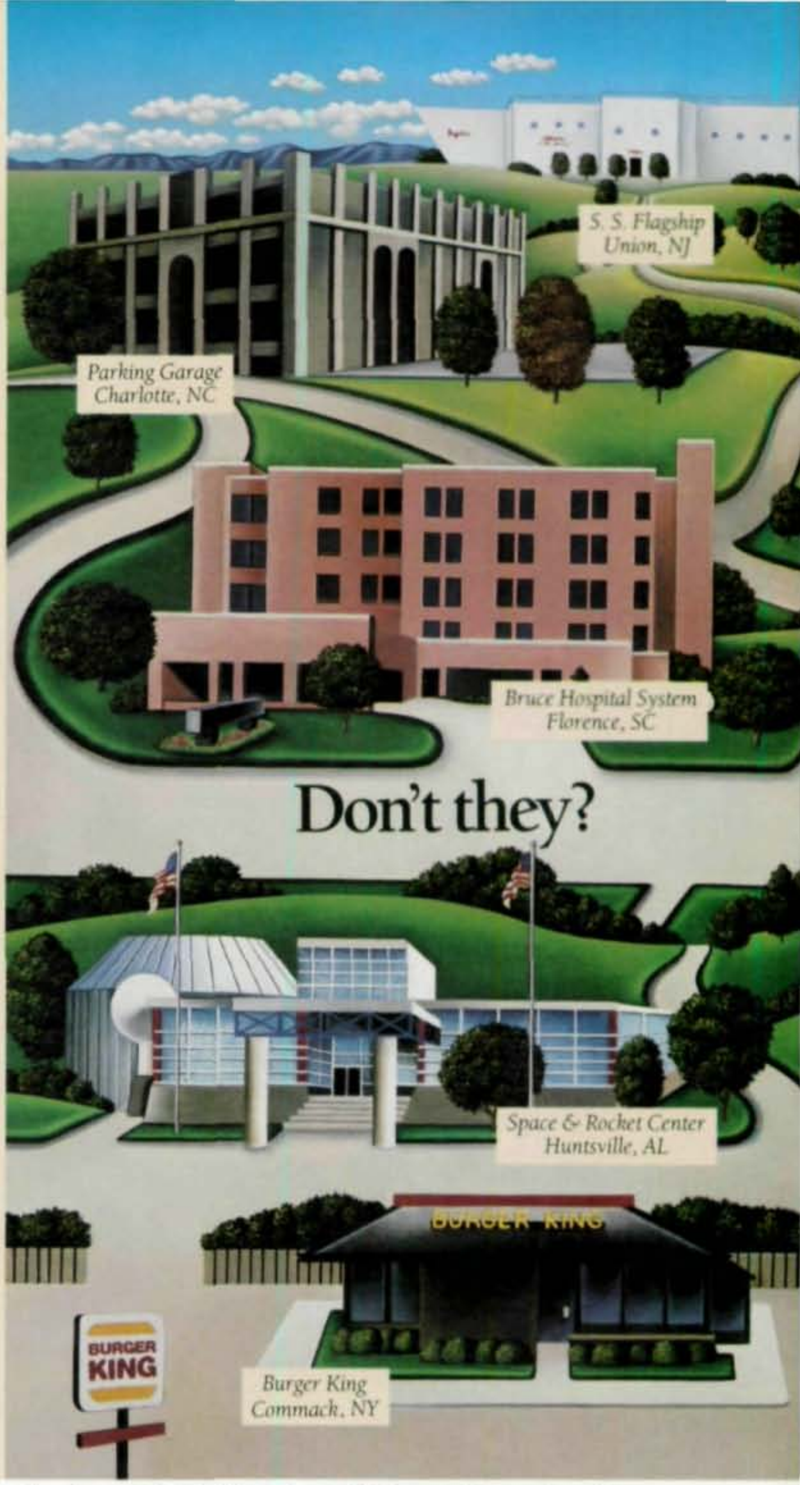

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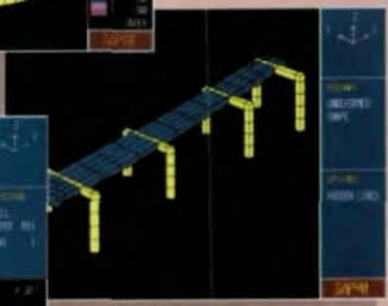
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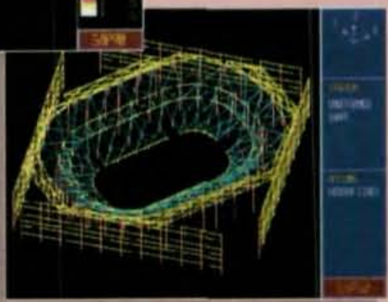
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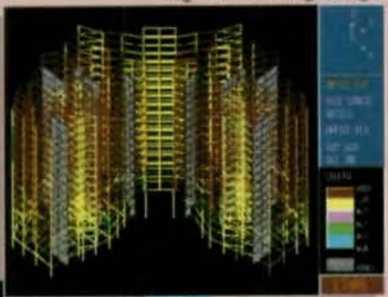
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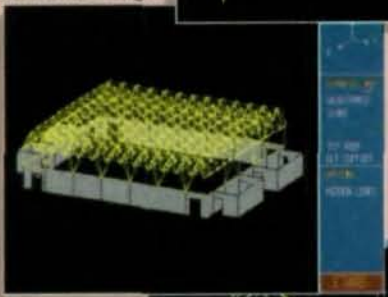
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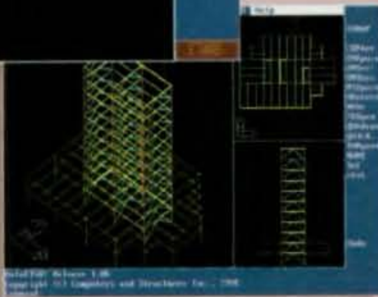
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