MODERN STEEL CONSTRUCTION

December 1993

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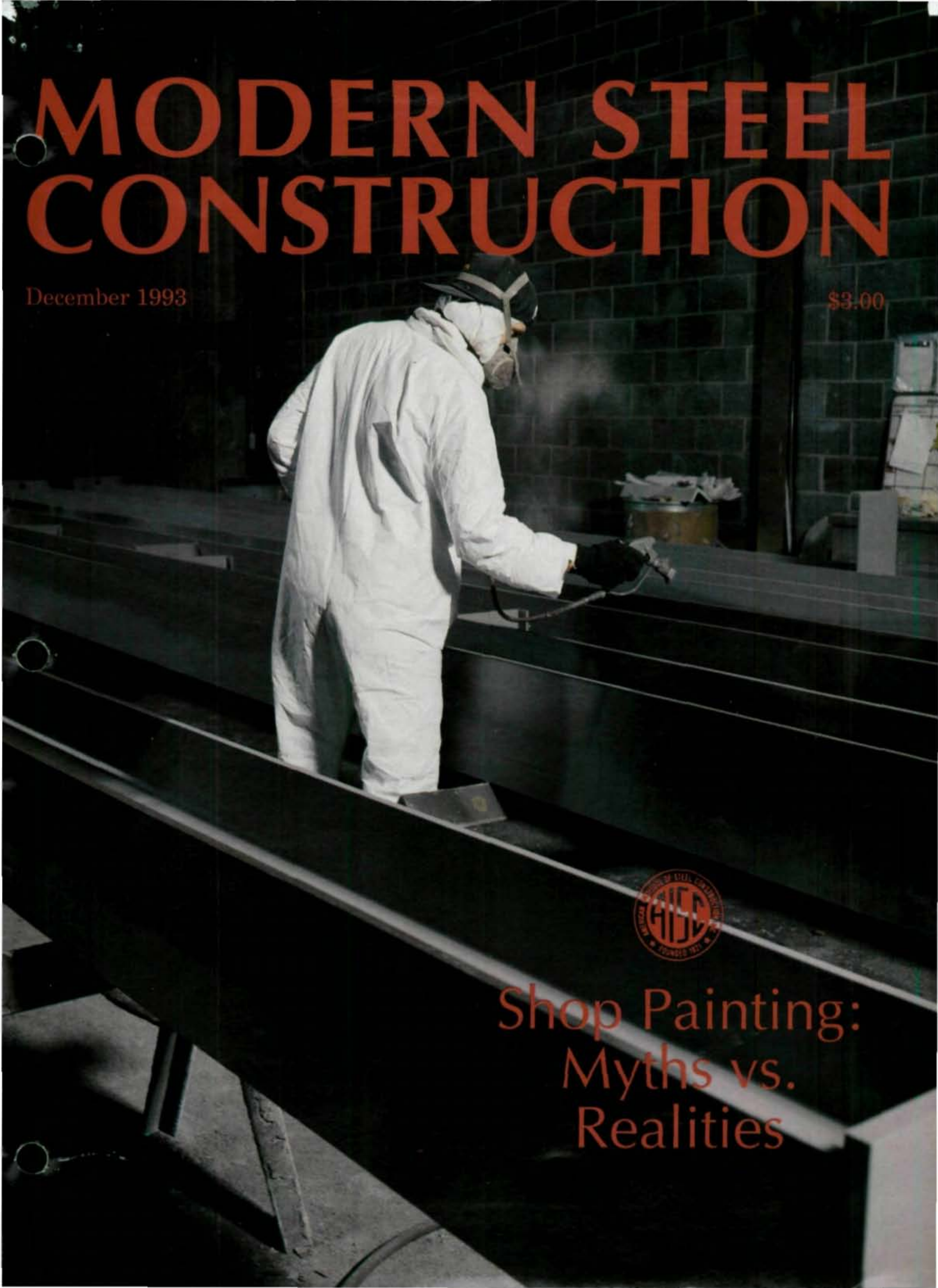

Shop Painting: Myths vs. Realities

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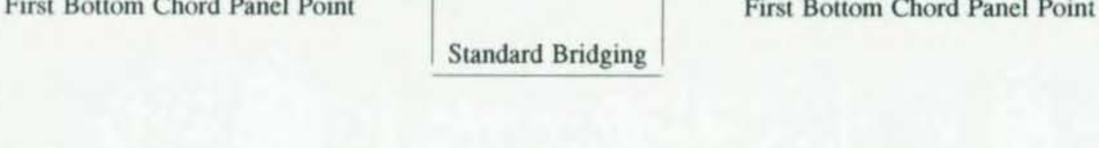
JOIST DESIGN DATA SHEET
No. 2

HOW TO DESIGN FOR WIND UPLIFT

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



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MODERN STEEL CONSTRUCTION

Volume 33, Number 12 December 1993

Eric Kline and James Machen from KTA-TATOR take a look at some commonly held myths about shop painting in a story beginning on page 32. Photo courtesy of Tnemec Co.

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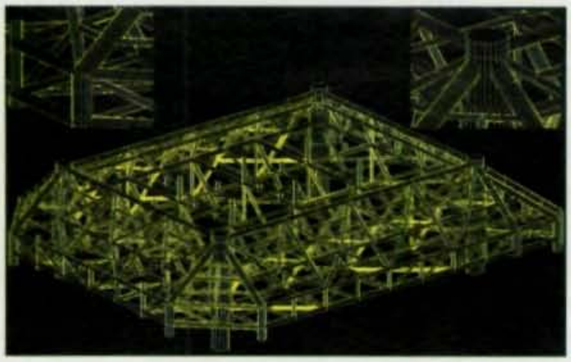
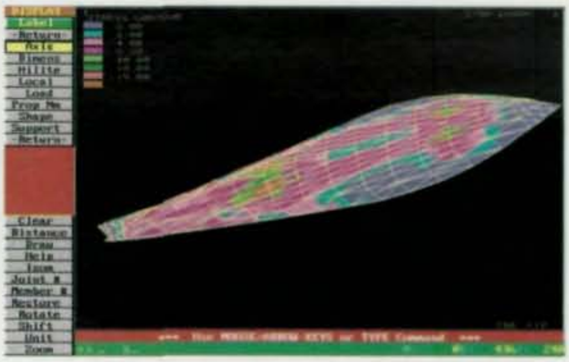
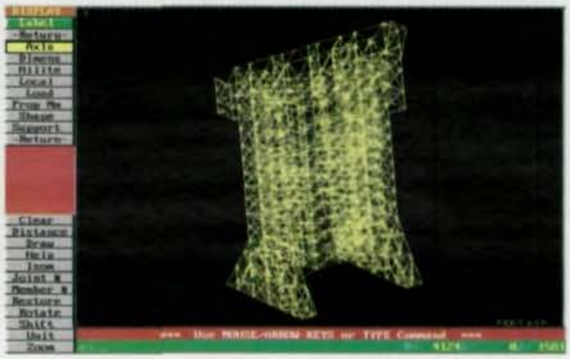


Offshore structure. Courtesy AKER Engineering, Norway



Concrete Canoe, Courtesy Drexel University, Pennsylvania

Roller Coaster. Courtesy Arrow Dynamics, Utah



NASA Launch Pad. Courtesy Brown & Root, Texas

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More Steel Info

If you enjoy reading the "Steel Interchange" section in this magazine, you'll probably also like reading *Steel Inspection News*, a bimonthly newsletter published by the Steel Structures Technology Center. In addition to information on such topics as alternate weld specifications and the Fastener Qualification Assurance Act, the newsletter offers advice on common design problems.

For example, on page six of the September issue, there's a discussion on whether you can substitute an SAE Grade 5 for an A325. Two pages later, the newsletter handles the topic of deck penetrations: "You know the problem. The roof deck or floor deck has been installed by the decking contractor, perhaps even inspected and approved. Then those darn mechanical contractors come along and start cutting holes everywhere for their vents, conduit, piping and such. The engineers on the project have tried to provide mechanical chases and framed openings, but it's never enough. The question is—how big a hole may I tolerate, and how do I go about repairing or reinforcing the opening?"

A one-year subscription costs $36, while a two-year subscription is $60. For more information, contact: Steel Structures Technology Center, Inc., 40612 Village Oaks Dr., Novi, MI 48375-4462 (313) 344-2910; fax (313) 344-2911.

While *Steel Inspection News* will help you with your every day practice, if you want to stay on the cutting edge of steel design, I recommend a subscription to the AISC's quarterly *Engineering Journal*. The first three issues of 1993 presented such subjects as: "Serviceability Limit States Under Wind Load," "Design Strength of Concentrically Loaded Single Angle Struts," "The Warping Constant for the W-Section with a Channel Cap," "Simple Equations for Effective Length Factors," "Design of Pipe Column Base Plates Under Gravity Load," "Strength of Shear Studs in Steel Deck on Composite Beams and Joists," "Composite Girders with Partial Restraints: A New Approach," "Shear Lag Effects in Steel Tension Members," and "Design Aid of Semi-Rigid Connections for Frame Analysis.

Of course, *EJ* also features some articles of immediate use. For example, in the 3rd Quarter of 1993 there was a feature on "A Tentative Design Guideline for a New Steel Beam Connection Detail to Composite Tube Columns." And in the 2nd Quarter of 1993, there was an article detailing "SI Units for Structural Steel Design."

A one-year subscription to *EJ* is only $15 (within the U.S.), while a three-year subscription is $36. Payment can be sent to AISC, P.O. Box 806276, Chicago, IL 60680-4124.

To further whet your appetite, we're reprinting an article from the 1st Quarter of 1993, "The Economic Impact of Overspecifying Simple Connections," beginning on page 24. Good reading. **SM**

THE POT OF GOLD...

Canam's quality and service for

GRAND CASINO

Grand Casino Inc. - Gulfport, MS
PHOTOS: FOTOWERKS/miller c. henry & assoc.

Owner: **Grand Casino Inc.** • Architect: **Cuningham Hamilton Quiter, PA**
Structural engineer: **Reigstad & Assoc., Inc.** • Contractor: **Roy Anderson Corp**
Steel fabricator: **Ellis Steel Co., Inc.** • Steel erector: **Bracken Const.**

Winning the pot of gold is the target of any gambler. But building a casino like the Grand Casino in Gulfport, MS, is not a question of luck.

Grand Casino is a unique construction project, more a boat than a building. The design considerations took into account a floating foundation, including hurricane storm factors. Utilizing a steel design minimized structural weight for this three level structure. Canam Steel submitted a cost-saving alternative of angle chord bowstring trusses in lieu of the original tube steel design.

Ellis Steel of West Point, MS, was the steel fabricator who selected Canam Steel to design and manufacture the bowstring roof trusses, as well as two floors of long span joists, totalling over 500 tons.

Gordon H. Reigstad, Ph.D., P.E., states "the time schedule for this project called for construction to be completed in 153 days. To accomplish the owner's request for this aggressive schedule, it was necessary that everyone work as a team. Canam's capability in providing expedited service with its prompt turnaround on fabrication and delivery is a credit to this team effort."

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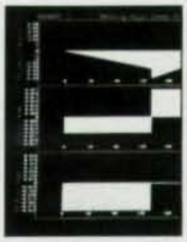
How to get from here



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Steel Interchange

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

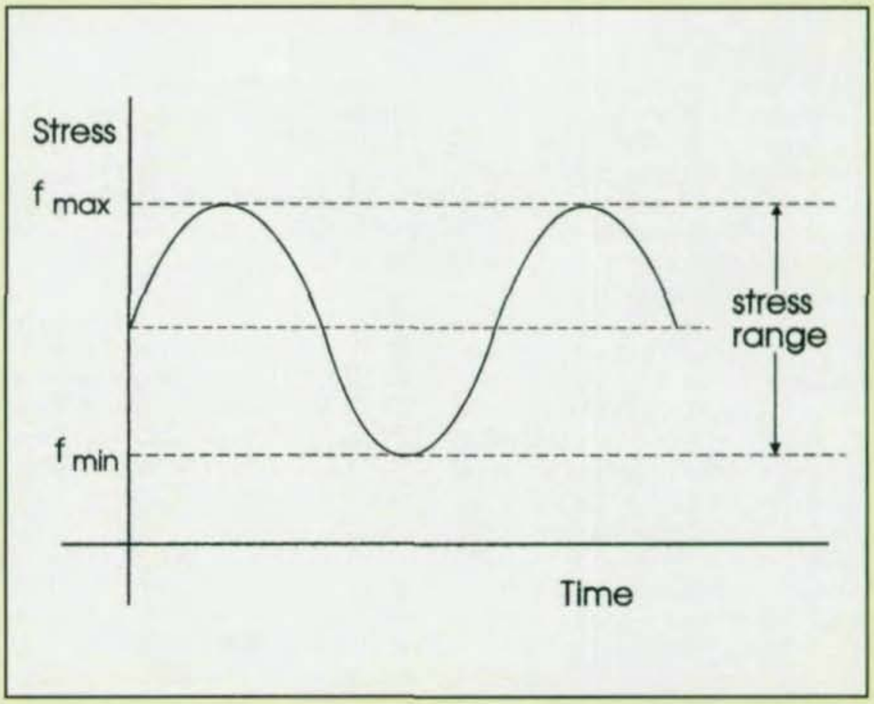
Steel Interchange
Modern Steel Construction
One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001

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

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

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

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



Is the section in the Specification concerning fatigue appropriate for variable amplitude fatigue loading?

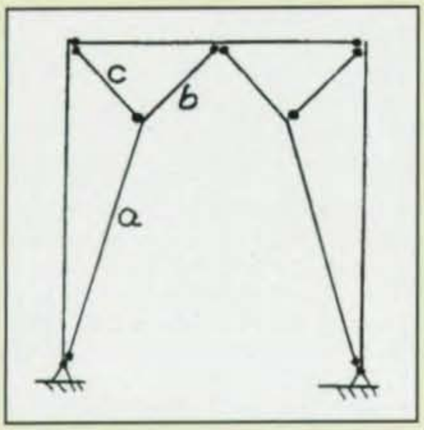
The allowable stress ranges used in the AISC Specifications Appendix A-K4.3 are for constant amplitude loading. For variable amplitude loading an effective stress range can be calculated using Miner's rule as used by Schilling, C. G. and K. H. Kleppstein, New Method for Fatigue Design of Bridges, Journal of the Structural Division, Vol. 104 No. ST3, ASCE, New York, March 1978.

Due to a clearance requirement, a frame has the configuration shown. For out-of-plane buckling, what will be the unbraced lengths for members a, b, and c with the following conditions: (1) a and b are rigidly connected and (2) a and b are released at their ends.

Strictly from an engineering standpoint, the best solution to the problem of the kinked frame is to provide out-of-plane bracing at the intersection of Members a, b, and c, in effect creating a 3-D truss. Members a, b, and c could then all be designed with K equal to 1.

If out-of-plane bracing is not possible, members a and b must be continuous at their intersection point, and must be restrained torsionally at their bottom and top ends, respectively. If a and b are pinned at their intersection, and free to twist at their end points, the frame is unstable for out-of-plane buckling; i.e., its theoretical buckling capacity is nil. No information is given on member sizes, loads, or connections. A heavy gusset plate connection might provide enough torsional restraint, depending on the loads and geometry; a lighter gusset plate might not.

Determining the exact effective length of kinked member ab is not a simple problem. Member ab must resist out-of-plane buckling under its own axial load, and must also provide out-of-plane bracing for pin connected Member c, which increases the stiffness requirement. A hand analysis model could be developed; however I would suggest using structural analysis software to estimate the buckling load. If software capable of performing a buckling analysis (eigenvalue analysis) is not available, software which can perform a P-Delta analysis could be used. The frame should be



modeled as a 3-D frame, with only the top three and bottom two joints supported out-of-plane. The actual frame loads should be applied, along with a slight out-of-plane load, at the intersection of members a, b, and c. The ratio of the P-Delta displacement to the first order displacement (out-of-plane) at this joint can be taken as a (slightly unconservative) estimate of the amplification factor $1/(1-P/P_{cr})$. From this, P_{cr} can be estimated.

In either an eigenvalue or P-Delta analysis, the inelastic behavior of the members must be taken into account. This can be done by using the analysis only to determine an effective length factor (setting estimated $P_{cr} = \pi^2EI/(KL)^2$, and solving for K). Then using the usual AISC column formulas to calculate the strength of the member. Alternatively, the stiffness reduction factors on Page 3-8 of the 9th Edition of the AISC Manual of Steel Construction (Allowable Stress Design) can be used to reduce the moduli of elasticity of the members used in the analysis.

David O. Knuttunen, P.E.
LeMessurier Consultants
Boston, MA

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

I recommend throwing the lug away and making a new lug using a drilled hole with chamfered edges. My second choice would be to require that all the hardened material at the flame cut hole be removed by grinding.

Flame cutting produces a locally hard brittle zone with microscopic cracks—ideal for points of crack initiation. Lugs are usually made from thick plate material of unknown Charpy impact (low temperature) properties and used outside in all ranges of temperatures — often on a repetitive basis with some impact loading and non-redundant lifting devices. The cost of a good hole is very small compared to the potential loss of life and damages due to brittle fracture at a lug hole.

George D. Conlee, P.E.
St. Louis, MO

Where is the best place to get information on foreign specifications and requirements?

In 1991 the Structural Stability Research Council (SSRC) published Stability of Metal Structures - A World View (The World View) 2nd Edition. The World View is a 940 page comprehensive world-wide study of over 100 specifications and codes on stability design of metal structures. It is the only book in the world that evaluates specifications and codes, compares and contrasts them, and explores some of the major reasons for their differences. The geographical regions covered are: Australia, China, East Europe, Japan, North America, and West Europe. Divided into 14 topics, the World View condenses the specification provisions and then gives regional and international comparisons and comments. The topics are:

- Compression Members
- Built-Up Members
- Beams
- Plate and Box Girders
- Beam-Columns
- Frames
- Arches
- Triangulated Structures
- Tubular Structures
- Shells
- Cold-Formed Members
- Composite Members
- Earthquakes
- General Provisions & Design Requirements

The World View cost is $85 ($68 for SSRC members). It can be obtained from: Structural Stability Research Council, Fritz Engineering Laboratory, Lehigh University, 13 E. Packer Avenue, Bethlehem, PA 18015-3191, Phone: (215) 758-3522, Fax: (215) 758-4522.

Donald R. Sherman
University of Wisconsin-Milwaukee
Milwaukee, WI

New Questions

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

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

When asked to design a temporary bracing system for steel beams and columns during the erection phase of construction, what loads are used and what factors of safety are employed for the bracing and its connections? California OSHA requires the bracing to be designed by an engineer but does not specify the loads.

Larry Borsclaire
Pace Engineering
Redding, CA

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

John P. Keating
Whirlwind Building Systems
Houston, TX

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Finding Low Cost Bay Designs

A new software package from AISC Marketing, Inc. condenses 2,400 composite beam and girder designs onto one IBM-compatible 3.5-in. diskette. The data allow design engineers to quickly analyze a wide range of bay alternatives to determine which will be the least expensive.

Designers can select different bay sizes, deck designs, model codes, AISC Specifications, live load reductions and live loads. After the design parameters are entered, the program provides 15 design alternatives, each listed in order of least cost.

"It's an invaluable tool for quickly estimating the cost of floor design and framing," according to Andy Johnson, vice president of marketing with AISC Marketing, Inc.

Each of 48 data files contains design information for 25 bays ranging in size from 20-ft. x 20-ft. to 40-ft. x 40-ft. in 5-ft. increments for both the filler beams and girder spans.

Because of the flexibility of composite design with regard to full and partial composite action and the optional use of camber to compensate for dead load, the least weight alternative is not always the most economical. Instead, when listing relative cost the database considers such factors as the amount of steel and camber and shear stud cost. However, since the relative cost of erection, connections, fire protection and other items are roughly the same for all alternatives, those items are not considered.

The design parameters considered in the bay studies include:

- Deck Slab—
  - 2-in. metal deck with 3-in. regular weight concrete;
  - 3-in. metal deck with 4.5-in. regular weight concrete;
  - 2-in. metal deck with 3.25-in. lightweight concrete;
  - 3-in. metal deck with 3.25-in. lightweight concrete;
- Specification—
  - LRFD-1986 or ASD-1989;
- Live Load Reduction—
  - BOCA; UBC; or SBC
- Live Load—
  - 50 psf or 80 psf

"The diskette contains an enormous amount of data. If you're talking to an architect or owner and want to establish the prelimary bay spacing, the disk will provide the least cost option," explained Abraham J. Rokach, director of building design with AISC Marketing, Inc.

The disk costs $50 and is available from AISC Publications. For ordering information, call (312) 670-2400 ext. 433.

New Publications from SSRC

A new 305-page book from the Structural Stability Research Council presents the latest information on plastic-hinge based methods of analysis.

Reseach has demonstrated the potential of these techniques for advanced analysis/ design application. The term "advanced analysis" indicates a method of analysis that sufficiently captures the limit states behavior encompassed by specification equations and for member proportioning such that separate member capacity checks are not required. When properly formulated and executed, this type of analysis holds the promise for rigorous assessment of the performance and maximum strength of framing systems and their components.

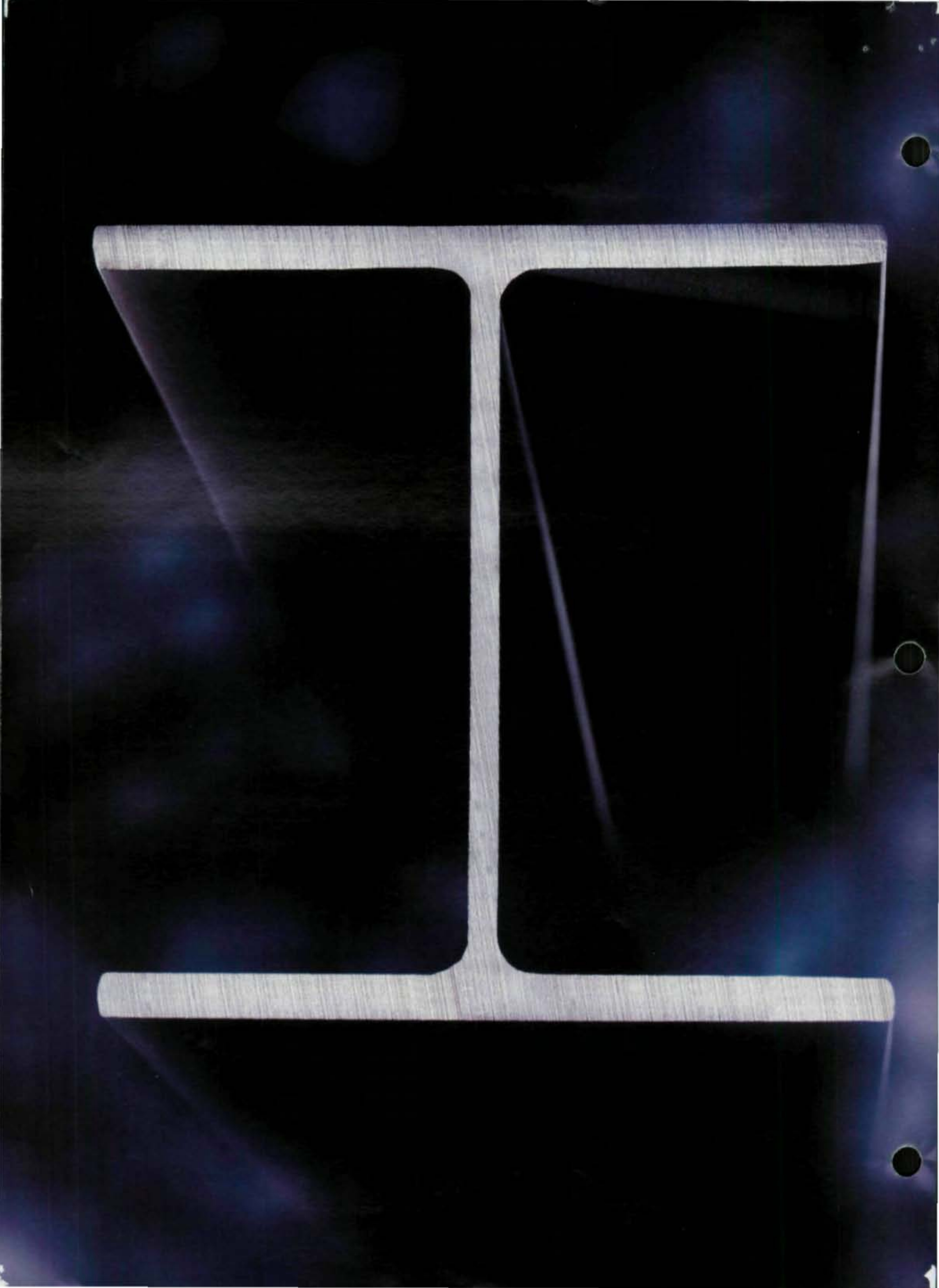
The book is divided into three major topic areas: Specifications & Analysis; Practical Implementation & Use; and Verification & Benchmarking Problems. Each topic area contains several contributed papers, followed by workgroup summary reports from the 1992 Task Group 29 workshop.

"Plastic Hinge Based Methods for Advanced Analysis and Design of Steel Frames" is available for $40 ($30 for SSRC members) from: SSRC Headquarters, Fritz Engineering Laboratory, 13 E. Packer Ave., Lehigh University, Bethlehem, PA 18015 (phone: 215-758-3522; fax: 215/758-4522).

Also available from SSRC is the Proceedings from a conference on "Is Your Structure Suitably Braced?" held last April and sponsored by SSRC, AISC, AISI and MBMA.

The book contains 21 papers plus panel discussion questions and answers. Topics deal with the design and evaluation of bracing systems for beams, columns and frames, and bracing strength and stiffness in various metal structures. Keynote sessions presented overviews of bracing requirements for beams, compression members, frames and building systems, all of which is covered in the book.

The book is available for $50 ($40 for SSRC members) from: SSRC Headquarters, Fritz Engineering Laboratory, 13 E. Packer Ave., Lehigh University, Bethlehem, PA 18015 (phone: 215-758-3522; fax: 215/758-4522). A 10% discount is given for orders of 10 or more copies.



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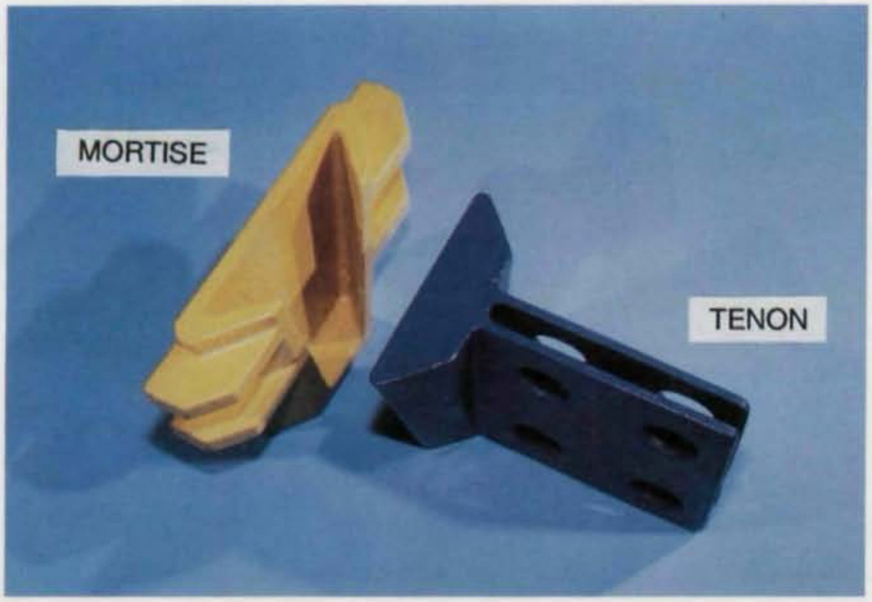
Advanced Connections Reduce Costs

A series of new connections under development at the ATLSS Center have the potential to substantially reduce erection time



By Wayne S. Lawrence, Le-We Lu, and B. Vincent Viscomi

With labor costs continuing to increase as a percent of the total cost of construction, the steel industry is searching for ways to reduce the time it takes to erect a structure. One part of this time savings may come from an innovative connector developed by the Center for Advanced Technology for Large Structural Systems (ATLSS) at Lehigh University.

The ATLSS Connector is a self-guiding, self-aligning shear connection designed to give steel erectors a means to assemble framing modules by both automated and semi-automated construction methods. It utilizes two interlocking components. The first, called a "tenon," is field bolted to a beam and has tapered slopes that match the slopes of the second member, which is called a "mortise" and is shop welded to the column.

The sloped walls are critical to avoid misalignment during construction since they guide the tenon to proper seating in the mortise. This erection concept, called "keystone coupling," also provides a beneficial wedging action so that pullout during erection is minimized. A seating bolt is included at the bottom of the connector to assure positive restraint against disengagement during and after construction. And finally, slotted bolt holes in the tenon allow for final adjustments.

The ATLSS Connector was recently tested during the construction of a low-elevation roof bay of an industrial plant. The bay dimensions were 20-ft.-by-30-ft. and the bay was designed to carry gravity load only. Since there were a large number of individual members in each bay, it was decided that the bay would be preassembled on the ground using traditional connections, and erected using an ATLSS Connection at each corner.

The entire bay was hoisted and installed with a conventional boom crane, two iron workers with guy wires, and a foreman. The assembly on the ground took 20 minutes, the attachment of the tenons to the corners of the bay took five minutes, and the lift took less than four minutes. After the large piece was placed, a three-point end piece was installed. This took approximately one minute to erect. Following successful erection, it took the iron workers seven minutes to install the securing bolts.

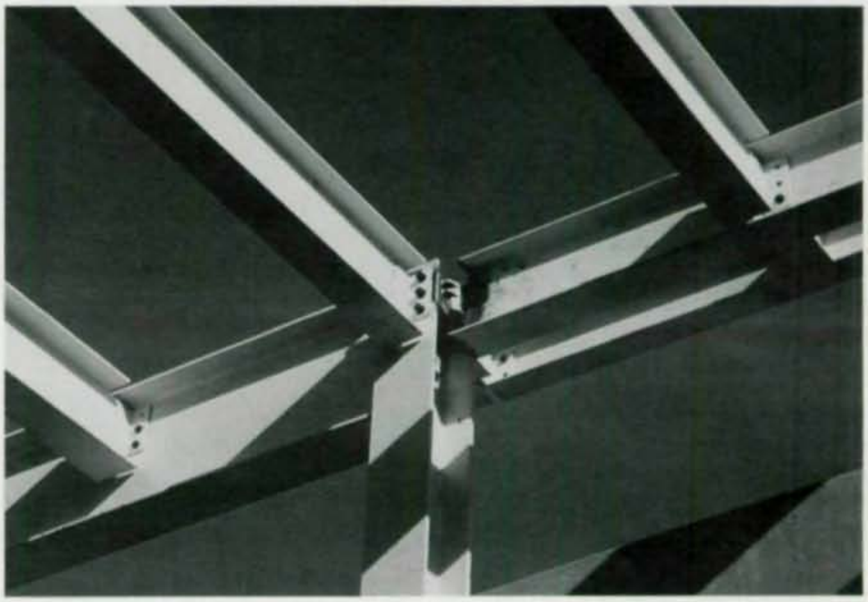
Total time of erection for the bay was 37 minutes, a substantial improvement when compared with the one hour it took to erect a similar bay in the adjacent span using conventional connections.

In addition, the ATLSS Connector provided other benefits. Because there were plans for a future vertical addition, a removable connection proved advantageous. Also, had the contractor so desired, the modular bay could have been outfitted with mechanical and electrical systems on the ground prior to erection or even off-site.

Further Developments

While the ATLSS Connector was initially conceived and designed as a shear connector, in the future there are plans to develop a moment connection incorporating the ATLSS Connector. Three different connection configurations with varying degrees of restraint were studied both experimentally and analytically. Five tests were conducted on exterior connections, scaled from a prototype frame that had been designed based on current practice.

The test program consisted of applying a monotonic and a cyclic loading, thus allowing for the assessment of static strength and stiffness as well as energy dissipation of the connections. The fully restrained connection was capable of resisting a bending moment and exceeding the plastic moment of the adjoining beam. The partially restrained connection achieved 97% of the beam's plastic moment in negative bending and 59% in positive bending. While acting as a



The ATLSS Connection was successfully proven in the field demonstration shown above. Conventional boom cranes were utilized with the aid of guide lines and the installation reduced construction time by more than half compared with conventional connections.

mid-web shear attachment, the ATLSS Connector contributed up to 16% of the moment capacity.

These experiments show that an all steel connection can incorporate a connector into its connection scheme and can achieve the full moment capacity of the beam with good ductility.

Semi-Rigid Composite Connection

Both the LRFD and the ASD Specification permit the use of Partially Restrained Connections, but it must show evidence of a predictable proportion of full end restraint and meet the following criteria:

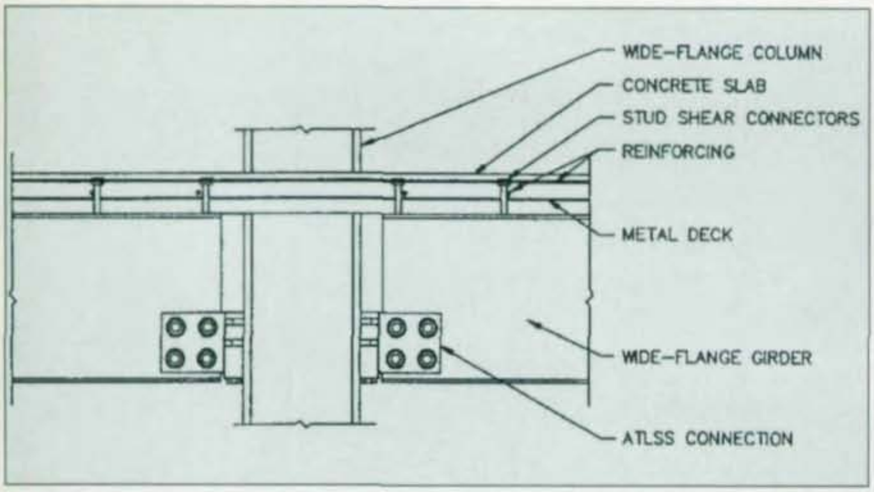
1. The connections and connected members must be adequate to carry the factored gravity loads as "simple beams."

2. The connections and connected members must be adequate to resist the factored lateral loads.

3. The connections must have sufficient inelastic rotation capacity to avoid overload of fasteners or welds under combined factored gravity and lateral loading.

Semi-rigid connections have proven to be cost effective, and tests show that the ATLSS Connector can be used in a composite design. In one successful test the ATLSS Connector was placed at the bottom of the beam while the reinforced concrete deck is at top. These two structural components act together in order to develop the necessary stiffness and strength.

Usually, the ATLSS Connector acts as both a shear connection and an erection aid. But by placing it at either the top or the bottom of the beam, it is possible to develop the necessary tensile or compressive forces required to resist the coupled forces that produce a moment. The placement of reinforcement bars in the concrete slab and around the periphery of the column will develop the tensile force needed to resist the negative moment. The stiffness, strength and seismic perfor-



Semi-rigid connections offer a promising economical design potential, and the ATLSS Connector makes the erection of such a framing system relatively easy. Shown above is an ATLSS Connection and a typical semi-rigid composite joint detail.

mance of the connection is being carefully evaluated at this time. However, it is believed that the use of this type of connection can result in substantial savings.

Tubular Columns

Hollow structural sections are an increasingly popular option for several reasons. They are available in high tensile strength grades, offer large radii of gyrations and better torsional properties than comparable open sections. In addition, where further strengthening or stiffness is required, the inside of the tube can easily be filled with concrete.

The ATLSS Connector may prove ideal for HSS since, by fitting flush to all sides of a tube, they can reduce the cumbersome detailing required with conventional connections.

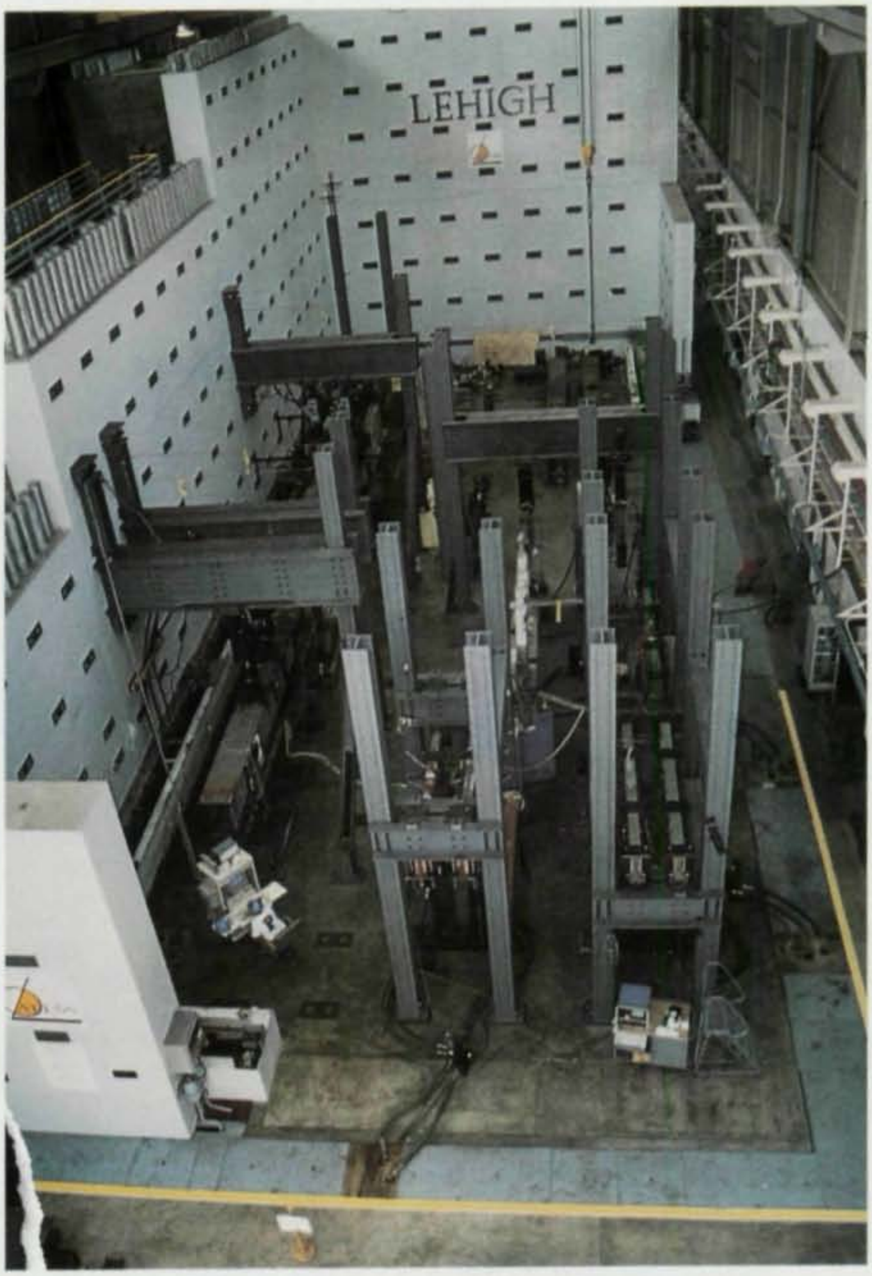
Research is currently underway to assess the behavioral characteristics of the ATLSS Connector when it is welded to tubes of varying wall thicknesses. The study includes cases in which the Connector transmits tension, compression, shear and bending. In addition to the local effect study, a semi-rigid composite connection will be designed and tested to determine its moment rotation characteristics under static and cyclic loading.

Construction Automation

ATLSS is engaged in a collaborative research effort with the National Institute of Standards and Technology (NIST) to combine the ATLSS Connector with the Stewart Platform, which could further automate construction and reduce labor costs.

The Stewart Platform consists of two platforms connected by six individually controlled linkages. Each platform has the six linkages connected to it at a total of three connection points—two linkages at each connection point. The connection points are non-colinear and are almost exclusively arranged to form an equilateral triangle and the linkages are connected to the platforms in a kind of "daisy chain" format.

The Stewart Platform is completely independent of any crane/track system and can be attached to almost any type of crane, including tower, boom and gantry bridge cranes. The orientation of the cables is such that the system has properties of a space frame and this configuration provides excellent translational and rotational stiffness when compared with a conventional boom crane. The Stewart



The Stewart Platform is a multi-degree of freedom crane that can be used in automatic and semi-automatic schemes.

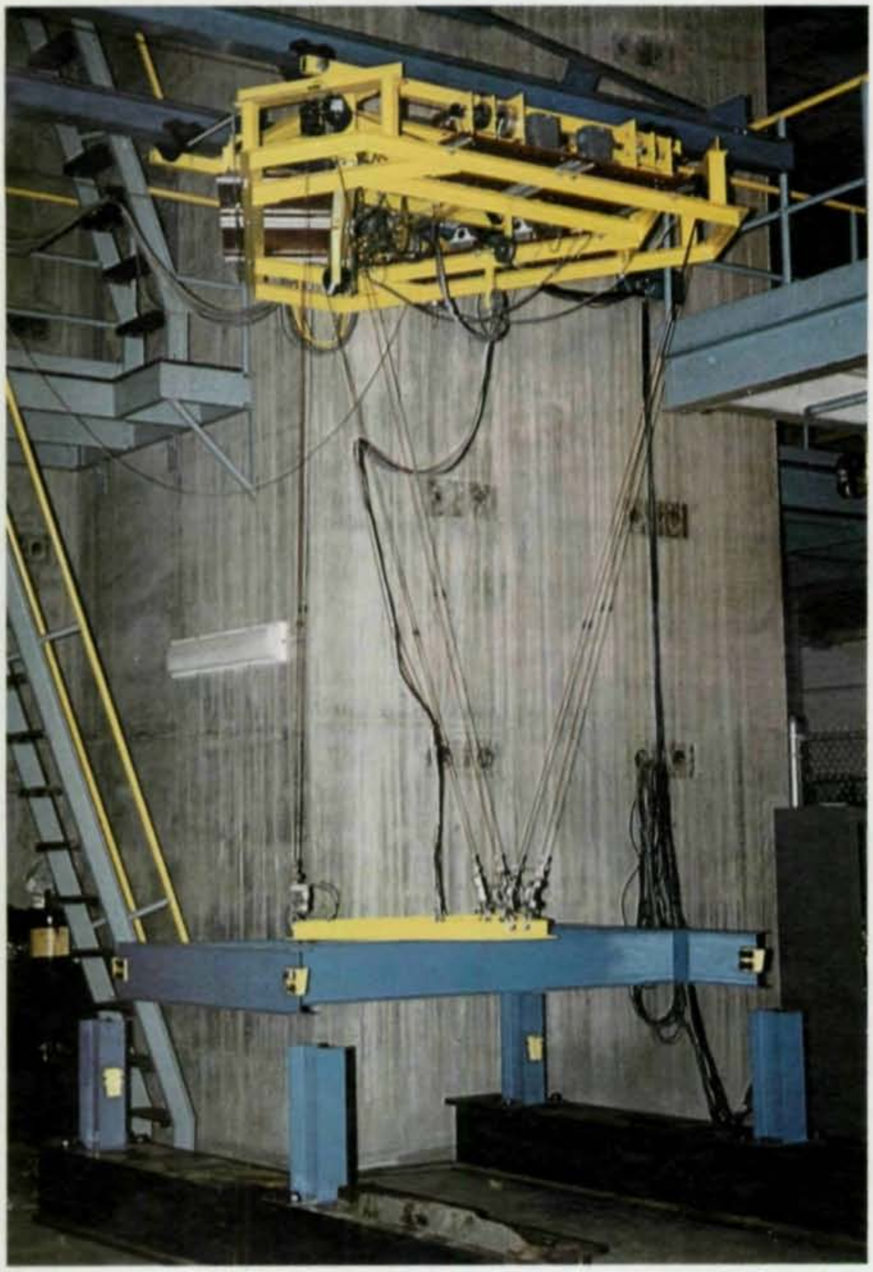
Platform can adjust the position and orientation of the lower platform with a high degree of precision in all six degrees of motion. This feature makes the platform an excellent choice for use with the ATLSS Connectors.

Future Plans

The ATLSS Center was established by the National Science Foundation in 1986 to improve the U.S. construction industry's competitiveness in the international market by reducing the overall cost of construction.

The ATLSS Connector was developed with the assistance of ATLSS industry partners, including Bechtel Corporation, Bethlehem Steel Corporation, DuPont Corporation and AISC.

Experimental castings of the ATLSS Connectors have been made using high strength low allow steel with an 80 ksi yield (HSLA-80). This material is a good candidate for the Connector



ATLSS Center's multi-directional testing facility is capable of testing full-scaled structural specimens in any direction or directions.

because of its high strength, excellent ductility and a good fracture toughness. Also, HSLA-80 is very weldable and, under normal circumstances, does not require any preheating when welded to other compatible steels, such as A36 and A572.

Eventually, a family of ATLSS Connectors will be available that will give the designer a range of sizes and properties from which to select a Connector to meet individual design requirements.

A patent has been granted for the ATLSS Connector and a commercialization strategy has been developed.

Wayne S. Lawrence is a research scholar and Le-Wu Lu is a professor of civil engineering at the ATLSS Center at Lehigh University. B. Vincent Viscomi is a research professor at the ATLSS Center and a professor of civil engineering at Lafayette College.

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Eliminating Overspecified Simple Connections

Showing actual reactions on drawings can substantially reduce connection costs

By Charles J. Carter and Louis F. Geschwindner

While the engineer of record will often request a fabricator to develop the detailed configuration of connections, all to often the EOR fails to provide all of the information needed to do so. Specifically, actual reactions are seldom shown on the contract drawings from which the connections must be designed.

Both the ASD 9th Edition and LRFD 1st Edition of the *Manual of Steel Construction* state: "For economical connections, beam reactions should be shown on the contract drawings. If these reactions are not shown, connections must be selected to support one-half the total uniform load capacity...for the given beam, span and grade of steel specified."

However, neither Manual quantifies the substantial economic benefit of showing beam reactions on the contract drawings. For simplicity, a standard configuration of the double angle connection will be considered in which only n, the number of bolt rows (and consequently, the length of the angles), varies. Based on values of n from two to 10, the cost of these standard connections will be estimated. Ranges of n compatible with each beam-size group will be identified and the percent increase in cost that results will then be determined over these ranges.

The Standard Configuration

The standard parameters of the double angle connection to be considered are as follows:

- The shop and field bolts will be $^3/_4$-in.-diameter A325-N at 3-in. spacing with $1^1/_4$-in. edge distance.
- The holes will be short-slotted in the outstanding angle legs (those connected to the supporting member) and standard otherwise.
- The angles will be $2L4 3^1/_2 x^5/_{16}$ (SLBB).

This standard configuration produces nine connections with the number of bolt rows (n) ranging from two to 10. While these connections will not satisfy every case, they will be adequate for the typical case and, therefore, will be used in this cost comparison.

Connection Costs

The costs considered in this article can be divided into three categories: material, shop labor, and field labor.

The material costs include the cost of bolts, washers, and nuts and the framing angles. The shop labor cost includes shearing and punching the angles, punching the supported and supporting members, and installing the shop bolts. The field labor cost is comprised of installing the field bolts. While material costs are readily available, labor costs are seldom a matter of public knowledge. Furthermore, labor costs will vary from fabricator to fabricator and from region to region. Consequently, the costs used in this article should be regarded as an average estimate and

Number of bolt rows required \ Number of bolt rows provided	10	9	8	7	6	5	4	3
9	11%							
8	23%	11%						
7	39%	25%	13%					
6	64%	47%	32%	18%				
5		73%	56%	38%	17%			
4				74%	48%	26%		
3						66%	32%	
2							85%	41%

Fig. 1: Percent increase in connection cost

Table 1.
Material, Shop Labor, Field Labor, and Total Cost Estimates of Double Angle Connections in Standard Configuration

n	L	Material Cost		Shop Labor		Field Labor		Rounded Total Cost
		Bolt	Angle	Time	Cost	Time	Cost	
10	29 1/2-in.	$22.50	$5.61	0.65	$13.00	2.65	$79.50	$121
9	26 1/2-in.	$20.25	$5.04	0.60	$12.00	2.40	$72.00	$109
8	23 1/2-in.	$18.00	$4.47	0.55	$11.00	2.15	$64.50	$98
7	20 1/2-in.	$15.75	$3.90	0.50	$10.00	1.90	$57.00	$87
6	17 1/2-in.	$13.50	$3.33	0.45	$9.00	1.60	$48.00	$74
5	14 1/2-in.	$11.25	$2.76	0.40	$8.00	1.35	$40.50	$63
4	11 1/2-in.	$9.00	$2.19	0.35	$7.00	1.05	$31.50	$50
3	8 1/2-in.	$6.75	$1.62	0.30	$6.00	0.80	$24.00	$38
2	5 1/2-in.	$4.50	$1.05	0.25	$5.00	0.55	$16.50	$27

should by no means be considered as universal.

The fabricators' costs assumed for this comparison are:

A325 tension control (TC) bolts ... $90.00/100 lbs.
$L4x3^{1}/_{2}x^{5}/_{16}$ ... $14.75/100 lbs
Shop Labor ... $20.00/hour
Field Labor ... $30.00/hour

These are base costs; selling costs, which would include overhead and profit, would be higher.

The material in the standard configuration are summarized in Table 1. The bolt material cost is based on a bolt length of 3-in. with one washer and nut each; about 83 lbs. per 100 count. The angle material cost is based on an angle size of $4x3^{1}/_{2}x^{5}/_{16}$, which weighs 7.7 lbs. per ft. The labor costs are based on the labor time estimates summarized in the same table. Note that, in each row of bolts, there are three bolts: one shop bolt and two field bolts. Total costs have been rounded to the nearest whole dollar.

Compatibility With Beam Shapes

The deepest compatible standard connection must fit within the *T*-dimension of the beam list in Part 1 of the ASD and LRFD Manuals. As recommended in Part 4 of the ASD and Part 5 of the LRFD Manual, the depth of the minimum standard connection should be greater than *T*/2. Given these limits, the compatibility of the nine standard connections with W-shapes is summarized in Table 2. Note that limitations such as coping, which may further restrict the maximum value of *n* are not considered.

Percent Increase In Connection Cost

Given the allowable variations in *N* in Table 2, percent increases in connection cost per unnecessary row of bolts provided are listed in Figure 1. Cells below the heavy line fall outside the spacially permissible variations in *n* given in Table 2. As an example of the use of Figure 1, consider a W18x50 and assume an end reaction that would require four rows of bolts. Using five rows of bolts instead, the largest *n* possible given the *T*-dimension of a W18, would increase the connection cost by 26%. When all beams and the full range of *n* are considered, the connection cost would increase from 11% to 85%. As the size of the beam being connected decreases, the percent change in cost increases and, as the number of unnecessary bolt rows increases, so does the percent change in cost increase.

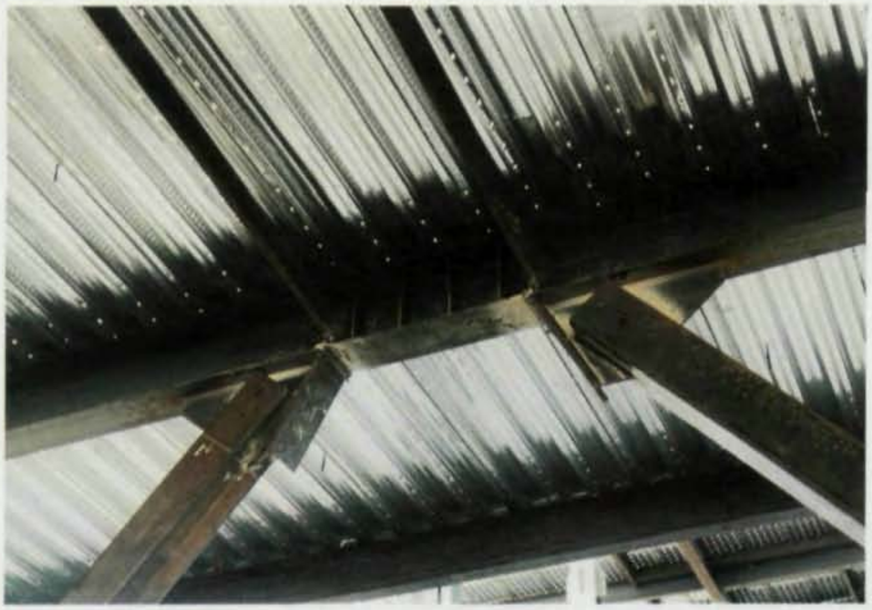
However, if only the typical simple beam sizes (those no larger than W24) are considered, the variation is narrowed somewhat. In most cases, the number of unnecessary rows of bolts will not exceed one. Accordingly, the cost increase will be between 13% and 41%.

Table 2.
Compatibility of W-Shape Beams and Standard Connection Depths

Shape	Weight	Number of Bolt Rows	
		Minimum	Maximum
W 36	359-230	6	10
	256-135	6	10
W 33	354-118	6	10
W 30	235-99	5	9
W 27	217-84	5	8
W 24	176-55	4	7
W 21	166-44	4	6
W 18	143-35	3	5
W 16	100-26	3	4
W 14	132-90	3	3
	82-43	2	3
	38-22	2	4
W 12	87-40	2	3
	35-14	2	3
W 10	112-33	2	2
	30-12	2	3
W 8	67-24	2	2
	21-10	2	2

Charles J. Carter is a staff engineer with AISC and Louis F. Geschwindner is a professor of architectural engineering at The Pennsylvania State University, University Park, PA. This article was adapted from a paper first published in the 1st Quarter 1993 issue of Engineering Journal.

Marketing Better Design



A small California structural engineering firm uses its computer expertise to attract new business

By Mick Wilson, S.E., and Rory Rottschalk, S.E.

In addition to dealing with seismic loads on very soft soils, the design of the new western regional headquarters for a major manufacturer of audio/video equipment was complicated by the marriage into a single structure of a four-story office and an enclosed "high stack" warehouse. The gravity analysis for this San Francisco Bay-area building was further complicated by an architectural design that called for a different configuration on each office level and the location of several clear-story atriums.

Structural steel was the obvious choice for the structural framing system due to its ability to provide an economical structural system despite the non-repetitive framing patterns, high floor-to-floor heights, and relatively large bay sizes. The typical bay is 40-ft. x 40-ft. in the office space and 40-ft. x 45-ft. in the warehouse. The second floor height is 21 ft., while the floor heights above are 15 ft. The warehouse roof height was 51-ft. to allow for tall storage racks.

Total office floor area is 413,000 sq. ft. and the warehouse area is 76,000 sq. ft. Also,some of the office space is provided in a two-story portion of the building that is immediately adjacent to, but seismically separate from, the four-story structure.

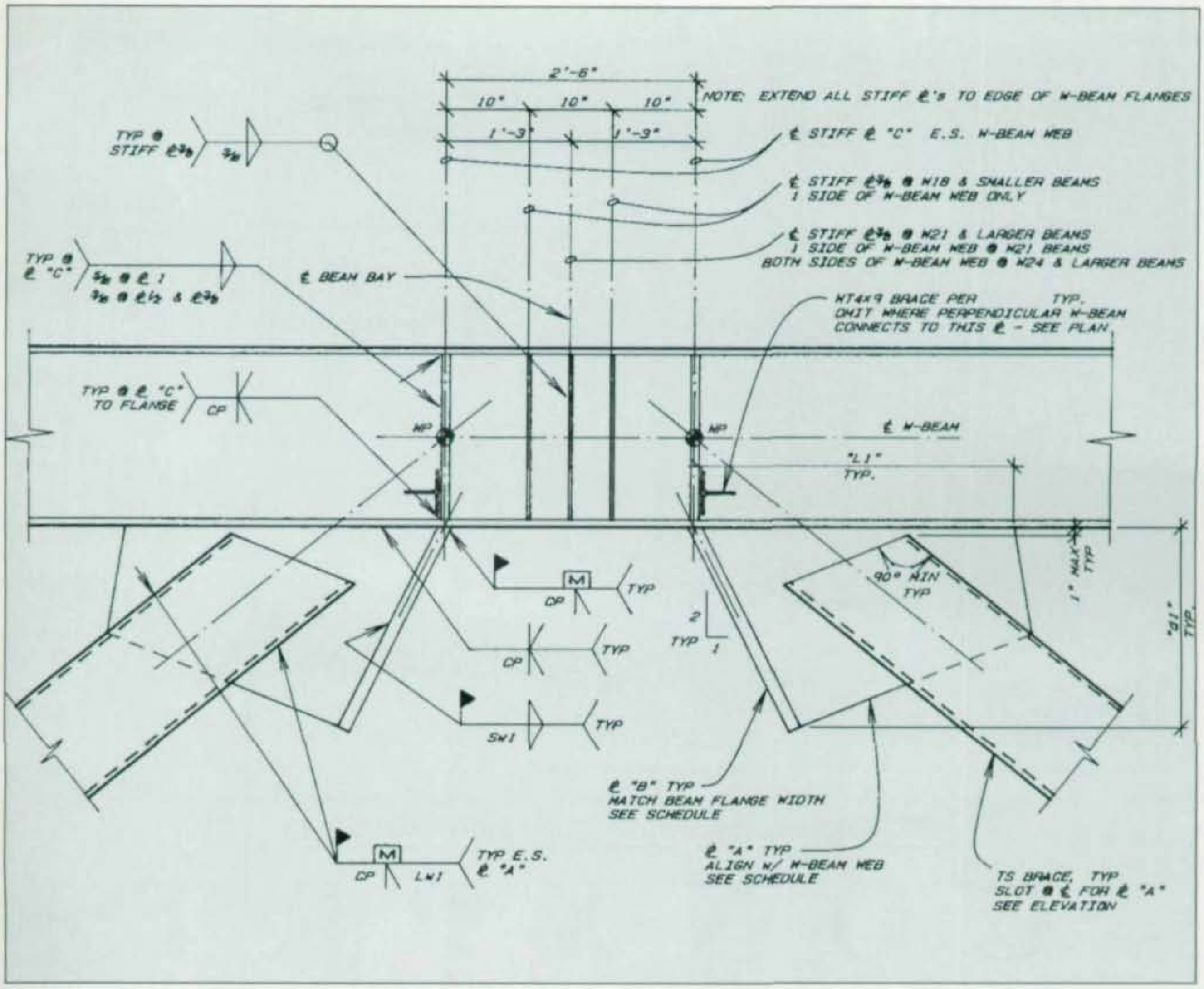
> The owner considered structural engineering to be something more than just a commodity

Marketing Design Capability

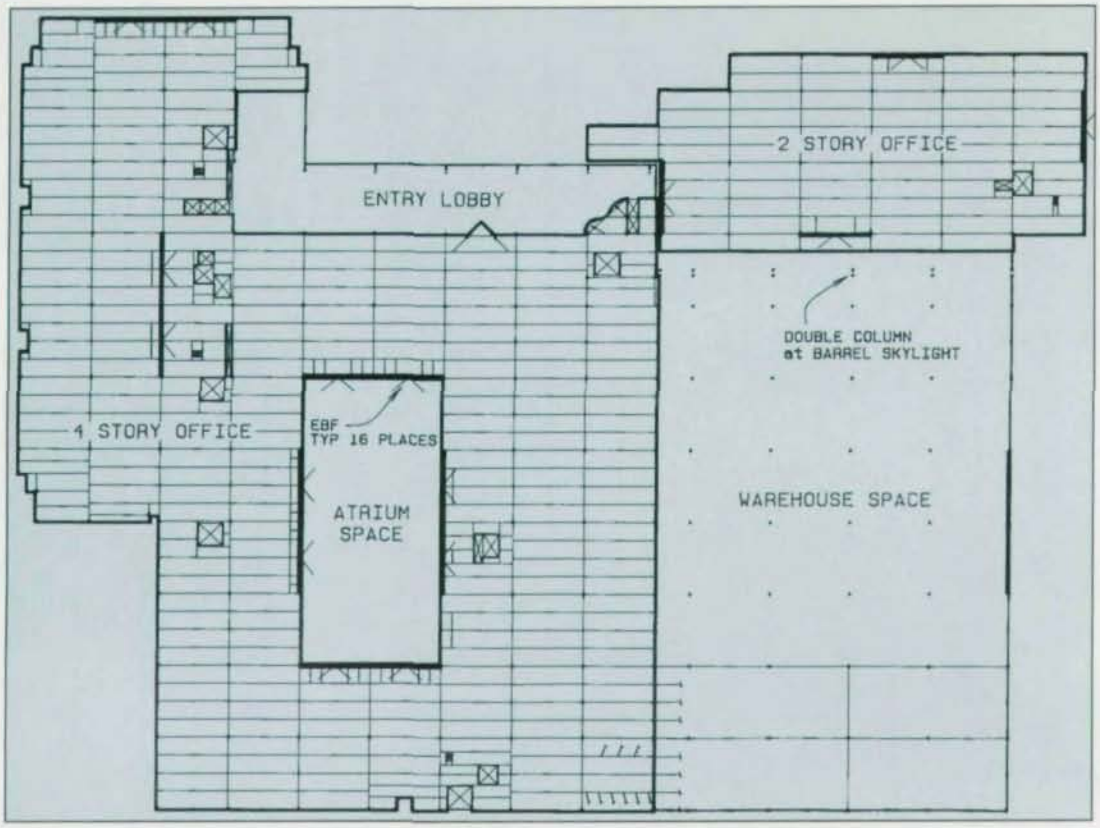
Surprisingly, this project did not involve the usual procedure of comparing design proposals from prospective "pre-qualified" structural engineers and choosing the engineer with the lowest fee. Rather, Culp & Tanner was hired for this project due to the company's ability to present our steel design capabilities as a measurable benefit to the client. On this project, unlike many others, the owner considered structural engineering to be something more than just a commodity to be purchased—like doorknobs.

Our marketing approach is to show clients how we can benefit their project. To do this, we use Ramsteel, a graphic software package for steel frame design. During a one-hour visit with a client, we graphically develop, design and discuss a building model. When presenting our capabilities to contractors, we develop a generic building model during the visit and then focus our presentation on the program's optimization potential, the complete material take-off produced, and the ability to generate very early, very accurate information. In the case of this project, we received a call from the contractor one day after our presentation—obviously our message was successfully communicated.

Using Ramsteel, we were able to produce a complete set of



Eccentrically braced frames were used to resistnt he horizontal seismic and wind forces, with the seismic forces controlling the design. Large members were used in the EBFs to minimize the number of braces required. Also, the EBFs were located along exterior walls when possible to minimize the impact on exterior office space.



In addition to seismic concerns, the designers of this 489,000-sq.-ft. building had to accommodate both office and warehouse space in an odd configuration.

design development drawings for all levels, complete with takeoffs, in just three days.

Gravity Design

The total gravity support framing tonnage was established at 2,700 tons during the preliminary phase of the project using Allowable Stress Design (ASD). However, while producing the working drawings our office had decided to investigate using the Load and Resistance Factor Design (LRFD) procedure. While most of the roof framing was unchanged using LRFD, the gravity framing tonnage dropped to almost 2,400 tons, a 10% savings over ASD. Steel fabricator on the project was AISC-member PDM.

The reduction brought up serviceability concerns, but these were quickly put to rest. Our investigation of vibration indicated that although beam sizes are reduced with LRFD, vibration perceptibility is not necessarily increased. Amplitudes increase with beam size reduction, but frequency is r

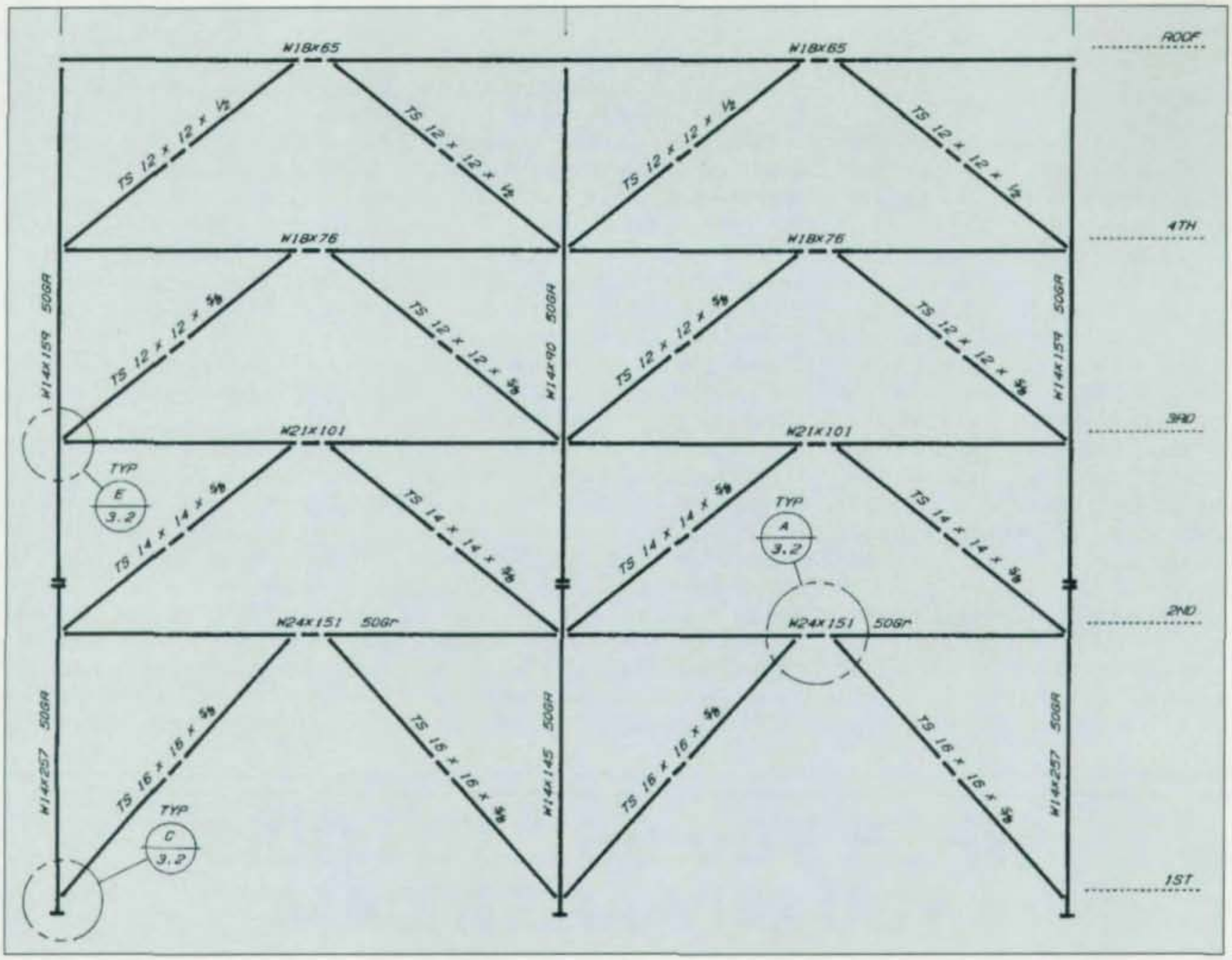
Reduced, resulting in a movement on themodified Reiher-Meister scale that is parallel to the lines of perceptibility.

A more serious concern was the need for future flexibility. Since the building was being molded to suit the owner's specific needs, there was concern that the building would not be versatile enough, if the owner ever decided to sell it, and this would limit the number of prospective buyers. To remedy this, the columns in the 51-ft.-high warehouse area were designed for a future floor at the second and third levels.

To accommodate this potential future modification, the warehouse columns, which were initially designed using tube sections because of their long unbraced length and relatively light loads, were changed to wide flange sections. All plates for the future beam-to-column connections were provided prior to erecting the columns. This was due to poor accessibility to make future connections and the difficulty of shoring the 51-ft.-high roof while welding connection plates to the columns in the field.

Two unique features in this building required special considerations:

A 66-ft.-high glass wall for the lobby entry is supported by 16-in.-diameter pipe wind girts at approximately 20-ft. centers spanning 40 ft. The wind girts are supported by W33x291 columns, which also support the roof 66 ft. above. Weak axis bracing of the W33 columns was provided by the wind girts, and the entire design of the wind girts



and columns was controlled by the deflection limitations during wind loads.

Located between the four-story building and the seismically separated two-story building is a 15-ft. radius, *quarter circle*, barrel vault skylight. Double columns were chosen to provide support on one side of the skylight at the seismic separation. This avoided the intricate slip details otherwise required and allowed the seismic joint to be located in a preferred configuration for waterproofing. The double columns are 51-ft. high with a beam along the top having no horizontal bracing in either direction. To provide this bracing and reduce the span requirements, a moment frame using TS12x6x$^1/_2$ members at 10-ft. on center rolled to conform to the skylight was provided. These rolled tubes are supported at their base by the roof of the two-story structure and at their top by the line of columns at the seismic joint. Intermediate TS12x6x$^1/_4$ tubes span between the rolled tubes and formed a *three-dimensional moment* frame that horizontally braces the columns to the lower roof structure.

Lateral Design

The project is located in seismic zone 4. The lateral system uses eccentrically braced frames (EBFs) to resist the horizontal seismic and wind forces with the seismic forces controlling the design. The tall second floor height and the 40-ft. column spacing was not conducive to a conventional moment frame structure while still possessing a ductile mechanism for energy dissipation during major seismic events. Large members were used in the EBFs to minimize the number of braces required and the EBFs were located along exterior walls when possible. This resulted in minimal impact to exterior office space. The total steel attributed to the EBFs is 610 tons (2.5 psf). ETABS from Computers & Structures, Inc. was used for the lateral design.

The EBFs were designed in accordance with the 1991 Uniform Building Code (UBC) as follows:

1. Preliminary beams are chosen based on the shear in the link. The beam chosen should just meet the required shear area with a maximum bending strength to force a shear controlled yield prior to flexural yield. Effects of gravity loads and axial loads must be included.

2. After the link beam is designed and the plastic strength is known, the diagonals are plastically designed for the axial and bending loads corresponding to 150% of the linkbeam plastic shear strength.

3. After all link beams and diagonals are designed, the columns are designed to be elastic for the axial and bending loads corresponding to 125% of the plastic strength of all link beams loading the column.

4. Link beam rotation is checked by converting the allowable angle of rotation to an allowable story drift ratio and comparing the drift ratios from the program output used to model the frames. This process uses the relationships presented in the SEAOC "Blue Book" commentary.

5. Connections are designed for the capacity of the diagonal braces by drawing free-body diagrams at the critical sections at each end of the braces.

EBF Connections

Connection configuration in the EBFs were kept as compact and concentric with the lines of action as possible to minimize local stress concentrations and connection plate slenderness. Double connection plates were used at the beam-to-column connection to both increase the bolt capacity and to avoid weak axis or torsional forces in the connection caused by the large seismic forces transferring through the connection. While groove welds were shown for the TS brace connection to the gusset plates, an alternate was issued at the contractor's request allowing large fillet welds to be substituted.

The design of the beam/brace-to-column connection was accomplished with four steps. First, the gusset plate height and thickness above the beam were determined by checking the shear, axial and bending loads at the column face between the top of the gusset and the bottom of the beam. Fixity provided by the column was neglected. Next, the length and thickness of the gusset plate along the top of the beam were designed using the load distribution along the gusset-to-column connection determined in the first step and the brace axial and bending load to determine the loads along the top of the beam. The beam bolted connection was then designing using the column-to-plate loads from the first step. Lastly, the weld of the brace to the gusset was designed for both the axial and bending loads. Theplastic strength of the connection must be greater than the plastic strength of the brace.

The design of the beam to the top of the brace connection is similar to the column connection. The critical section is checked at the bottom of the beam. In this case, the gusset is flanged on the link end so as not to project into the link length. This results in a "T" section, which is sized such that its neutral axis falls on the line of the action of the brace. Additionally, the flange was inclined to improve the connection compactness. This had no impact on erection because, either way, the flanges could not be installed before the diagonal braces were in place. Link beam stiffener plates and braces were provided as prescribed in the UBC.

At the foundation level, the braces had been modelled fixed since the foundation support was provided by friction piles. Horizontal forces were transferred to the slab on grade diaphragm and the vertical and bending loads were resisted by the piles.

Although EBFs are a recognized lateral system addressed in the code, some uncertainties still exist with design implementation. Some of these involve fixity at the ends of the diagonals, the effect of gravity beam connections on the link length, and whether the foundations should be designed for the brace elastic capacity. Regarding diagonal fixity, we found that considering the ends of the diagonals "pinned"resulted in a 20% savings in the diagonal braces. However, if the ends are

cosidered fixed, it is advantageous to minimize the brace-to-beam relative stiffness. To accomplish this, use tube sections with $^1/_2$ in. and $^5/_8$ in. wall thicknesses to minimize the flexural stiffness of the diagonals.

Floor Construction

All floors were constructed of 3-in.-deep metal deck with lightweight concrete, using Porte Costa lightweight aggregate, for a total thickness of $6^1/_4$ in. While we have come to expect concrete fill on metal deck to experience significant cracking, the Porte Costa aggregate concrete has given excellent performance. At our last visit, the only cracks were hairline size over the tops of the girders. Most of these floors were reinforced with #4 bars at 16-in. on center or #3 at 12-in. on center each way, but those areas with 6x6-w1.4xw1.4 mesh had no more cracking than the areas with rebar.

Contractor on the project was Koll Construction Co., Pleasanton, CA.

Mick Wilson, S.E., and Rory Rottschalk, S.E., are, respectively, a project engineer and a vice president with Culp and Tanner, Structural Engineers, Inc., a design firm headquartered in Chico, CA.

Shop Painting: Myth Versus Reality

A look at what is practical, cost- effective and necessary within the painting industry

By Eric S. Kline and James D. Machen

From the days of our youth, many of us will recall the tales of a giant lumberjack, Paul Bunyan, who with his blue ox Babe, performed superhuman feats on the American frontier. Or perhaps you were more fond of the larger-than-life cowboy, Pecos Bill, who dug the mighty Rio Grande with his bare hands.

The myths of these American frontier characters, which have been passed down from generation to generation, are not unlike certain painting practices or trains of thought that have been prevalent in the shop painting industry. While these shop painting "myths" are not as grandiose as those of Paul Bunyan or Pecos Bill, they take on mythological qualities when the overall importance of these concerns are overstated to the point where instituting or enforcing them is no longer practical, realistic, reasonable, or cost-effective. While many of these myths may actually have a foundation as an obscure finding in some scientific study or accelerated testing process, they take on mythical proportions when their importance is over-exaggerated.

The purpose of this article is to shed the "light of reality and practicality" on these commonly held myths. The views expressed are built upon years of hands-on shop painting experience and field performance observations of what is realistic, practical, cost effective and necessary within the industry.

Punched holes can often result in very sharp edges, which must be broken.

Myth: All sharp edges must be ground to a minimum $^1/_{16}$-in. radius.

Reality: This myth is probably rooted in the common knowledge that coatings draw thin on sharp edges due to the forces of surface tension and capillary action. The reduced thickness of coating in such areas can lead to edge failure.

Even though this was true when most industrial coatings were oil-based, it was never true where even these materials were applied on rolled edges. Rolled edges have never been shown to require any additional preparation for painting, as the rolling process leaves a rounded edge, even though it might not be a $^1/_{16}$-in. radius. Even when edges are sheared or burned, it is not certain that grinding to a $^1/_{16}$-in. radius is always necessary for paint performance all the time. Highly pigmented zinc-rich paints do not flow away from the edge, and in addition, provide galvanic throwing power to protect any edges or areas not coated.

Also, these materials resist corrosion undercutting. Therefore, the requirement that burned or sheared edges always be ground to a minimum $^1/_{16}$-in. radius is questionable. The process of rounding these edges is not reasonable, practical or cost effective.

Another important fact is that technological advances by coating manufacturers have yielded other highly pigmented solvent and solvent-free epoxy formulations that have modified rheological properties and better resist flow when applied to edges. In fact, as reported in a past *JPCL* article (December 1989, pp. 16-20), as much as 80% or more of the applied film thickness of some solvent- free epoxy coatings remain on the edge. This is remarkable given standard solvent-containing coatings maintain only 30% to 50% of their applied thickness in these areas. This is strong evidence to suggest that this "myth" is highly impractical. In fact, improved specification language should include provisions that reflect the following:

Sharp edges, such as those created by flame cutting and shearing, shall be broken prior to surface preparation. (Breaking

The hackles on this girder have been repaired by grinding. In a case such as this, re-profiling is necessary.

the edge can be accomplished by a single pass of a grinder in order to flatten the edge.)

The rolled edges of angles, channels, webs, and I-beams are presumed to need no further rounding.

Machine fillet welds are considered a paintable surface with no further treatment required. Only weld spatter need be removed.

Related Myth: Surface imperfections such as ridges, slivers, fins or hackles must be ground flush since they also are sharp edges.

Reality: Such anomalies are surface imperfections on rolled sections and plates. They result when small (usually less than $^1/_2$-in.) areas of the steel surface are not well bonded to the surrounding surface and are bent upward during the blast cleaning, usually by a metallic abrasive). It is typically only necessary to cut off the head of isolated hackles, with no further grinding necessary. An exception could occur if there were extensive hackles in a small area; in such an instance, some further attention may be warranted.

Myth: Due to the development of scattered "rogue peaks" when abrasive blast cleaning, an abrasive size should be used that will produce a low profile, typically some 1.0 to 2.5 mils (some specifications limit maximum profile to 1.9 mils). This is important since rogue peaks stick up through the paint and adversely affect coating performance.

Reality: "Rogue peaks" are described as peaks which are abnormally high (as much as three to four times the average profile height) and infrequent and occur on an abrasive blast cleaned surface. By using abrasives that produce a very low profile, the height of these peaks is limited to 6.0 to 8.0 mils. This reasoning suggests that the rogue peaks notably harm paint performance because they actually stick up through the paint, lead to pinpoint rusting and hence premature paint failure. At first glance, this logic appears to have some merit. However, upon further examination, "real world" facts suggest it is not a significant factor in premature coating failure.

Realistically, if rogue peaks were a significant factor, one would expect to see millions and millions of tons of blast cleaned and prime coated steel having considerable evidence of pinpoint failure. We have observed hundreds of thousands of square feet of shop primed structural steel that has been stored outdoors for extended periods of time and have not observed this to be the case.

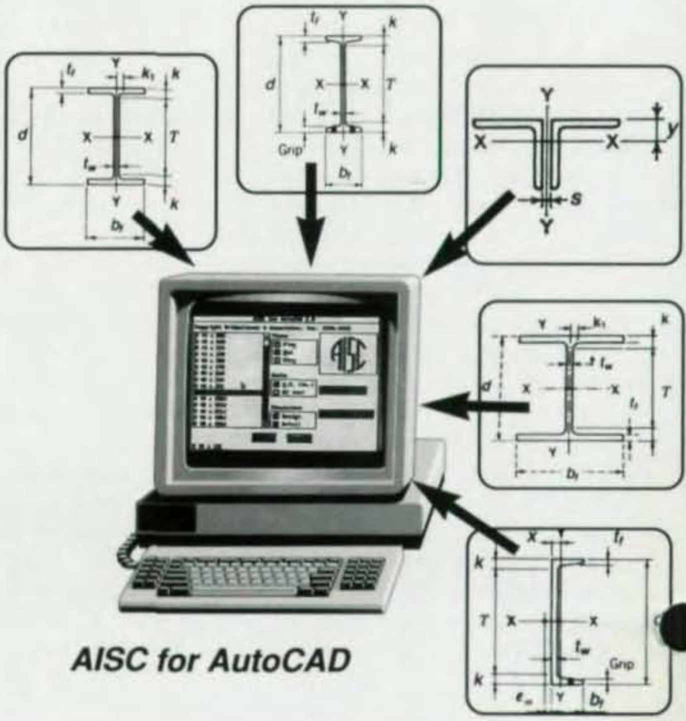
While the existence of rogue peaks is not denied, their real-world effect appears to have been tremendously exaggerated. An early detailed study by Joseph Bigos ("Anchor Pattern Profile and Its Effect on Paint Performance," *Corrosion*, August 1959) found that anchor pattern profile did not significantly affect paint performance. This study provides support for the conclusion that controlling the blast profile within a very narrow range is an unnecessary expense. In the Bigos study, the amount of corrosion from rogue peaks was so slight that surfaces had to be examined microscopically in order to detect any rust at all. Bigos noted that surfaces containing pinpoint rust were rated on the ASTM D610 rust scale as 9.5. ASTM Rust Grade 10 equals no more than 0.01% rust, while ASTM Rust Grade 9 equals not more than 0.03%. Bigos concluded: "Anchor pattern profile below 4.0 mils in height has insignificant effect on paint performance in atmospheric exposure when as little as 1.5 mils (dry) paint are applied." To put this into perspective, imagine 1,000 sharpened pencils placed upwards, their pointed tips representing the peaks of a blast cleaned profile. Pinpoint rusting as observed by Bigos represented rusting on the tip of only 20 pencils. The cost of applying additional paint to cover the highest peak would not be cost effective, justifiable or practical. In addition to his comments about profile, Bigos also was concerned about coating performance and noted: "No evidence or justification was found for the rule that paint thickness should be three times the anchor pattern height. Instead, about 1 mil equivalent of dry film thickness should be allowed to fill normal anchor patterns over the thickness of paint known to give paint protection desired on smooth steel."

As with the sharp edge issue discussed previously, advances in coating technology have resulted in higher film build and high solids coatings with advanced barrier properties. This trend would seem to further diminish the importance of these isolated peaks. Additionally, much shop priming is done using zinc-rich primers. Among the well-known advantages of zinc-rich coatings are their "self-healing" properties, since the zinc rusts before iron. Another fact is the "throw power" or the ability to galvanically protect adjacent bare steel. While Bigos found that blast profile up to 4 mils was acceptable, we believe that a profile with a range of 1 to 3 mils is adequate for most coating applications, and that the impact of very tight profile control on paint performance is overstated.

Myth: The commonly used specification phrase, "paint all steel surface," means coating the inside of bolt holes.

Reality: Painting inside bolt holes is non-traditional. Let us be reasonable in this area; if painting inside bolt holes is not specifically required by specification, it should be normally assumed that it is not a specification requirement. In addition, it is believed that the benefits anticipated from coating the interior of bolt holes have been overstated, as the extra time, effort and expense necessary to prepare and coat these areas does not appear to provide any worthwhile long-term advantages. If uncoated bolt holes were crucial, millions of bolted connections would be bleeding rust. Again, we have examined thousands and thousands of bolted connections (both slip-critical and bearing) and only rarely see rust bleed-out from a bolt pattern. This is true even though these holes will have uncoated threaded bolts twisted, torqued, and often forced through them. While the nominal clearance in the pattern is so tight that additional film build from conventional coating, not to mention the new technology high build products, necessitates coating removal for proper fit-up. Even if the holes and fasteners were prepared and coated, bolt placement and nut installation would leave coatings severely damaged or completely removed.

If some additional protection in the bolt pattern is desired, a viable alternative with roots in

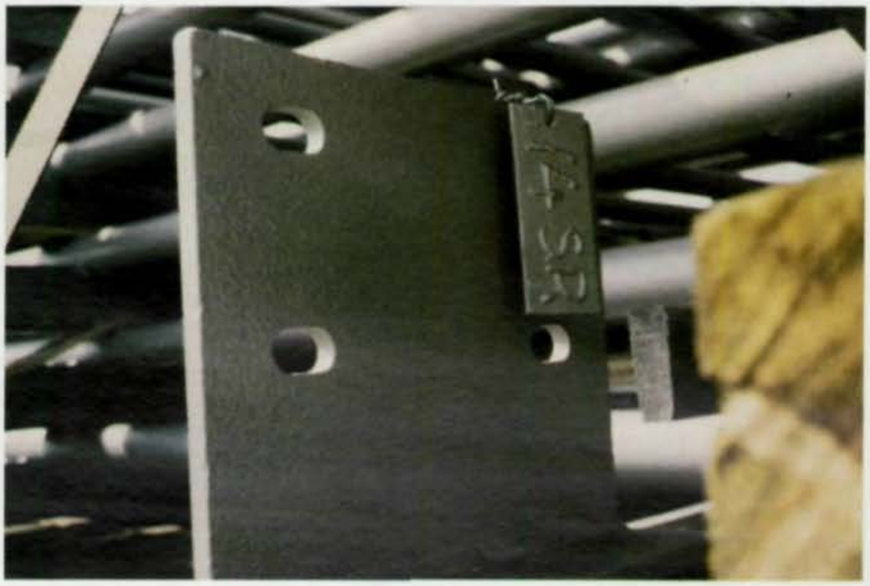


the new coatings technology now exists. The recent development of low viscosity, penetrating sealers approaching 100% solids have shown considerable promise with their inherent ability to "wick" into small crevices around connection plates, bolts, nuts and washers. These "wicking-type" sealers provide an additional measure of corrosion protection. These sealers have demonstrated the unique ability to effectively prevent premature corrosion on bolted connections as well as other difficult-to-coat areas. Rather than pay to have the interior of bolt holes prepared and coated, cost-effective specification provisions should consider the following facts:

During the coating application process, any coating that lands inside bolt holes shall be allowed to remain.

If additional protection is needed after installation of fasteners in the shop or field, a penetrating sealer can be applied on a best-effort basis to seal crevices, fasteners, nuts and washers.

While not traditional, if extraordinary special protection is



Shown above is a near-perfect coating application on the interior of slotted bolt holes.

needed, an approach that would provide a better seal could include a sealer and wet sponge holiday testing on a one-time, best-effort basis. This level of protection should allow the detection and cost-effective remediation of over 90% of the holidays present.

Myth: In cool weather (below 50 degrees F), it just takes longer for water-borne industrial acrylics or latex coatings to achieve full cure, but they're okay.

Reality: While there have been enormous advances in industrial water-borne coatings technology, a limiting factor may be their application in cool weather, particularly at temperatures less than 50 degrees F. In many instances, curing problems with this type of coating material may go undetected, because the finished product visually "looks okay." In reality, the curing process has been disrupted on a microscopic scale, leading to permanent irreparable damage to the applied film. These coatings dry and cure by initial water evaporation and particle coalescence. Co-solvents are used that are of vital importance to the coalescence process, but at temperatures below 50 degrees F the co-solvents cannot perform their intended function. An oversimplified explanation of the curing process is as follows: water is used to carry the latex particles to a side-by-side orientation on the substrate. Co-solvents then act to physically soften these individual particles and essentially "melt" or "fuse" them together into a continuous paint film. However, at temperatures below 50 degrees F, while the water carries the particles into proper alignment, the particles are not sufficiently fused together. Even though the temperature is low, these co-solvents will still evaporate and leave the coating film without performing their intended function. At this point, with co-solvents evaporated,. the now-porous film is permanently damaged. Even though the coating may appear to be satisfactory, in fact, when so applied it will be incapable of providing the performance built into the coating formula.

In this case, the myth is wrong, because it does not just take longer for waterborne industrial acrylics or latex coatings to achieve full cure (below 50 degrees F); in fact, they may very well never achieve satisfactorily cured film properties.

Myth: Many two-part epoxy coatings have critical recoat times. When these recoat times are exceeded, the best method of preparing the surface to receive additional coating is SSPC-SP7 "Brush-Off Blast Cleaning."

Reality: The myth of this thinking lies in believing that SSPC-SP7 "Brush-Off Blast Cleaning" is the quickest and most cost-effective approach. Field experience has revealed that many sound, intact coatings are damaged unnecessarily by Brush-Off Blast Cleaning. The popularity of Brush-Off Blast Cleaning is due to the speed at which it can be accomplished. Consequently, it is many times deemed the most cost-effective and practical method by suppliers, consultants, and shop painters. However, this thought process may be short-sighted. The reality is that abrasive propelled at high velocities often damages adjacent sound coating. After Brush-Off Blast Cleaning, thousands of microscopic discontinuities may exist in the paint film. These holes often provide a path for oxygen and moisture to penetrate an otherwise sound and continuous coating film and affect future system performance in numerous ways.

A longer term, more practical approach is the use of strong solvents accompanied by sanding with a fine abrasive for non-immersion applications. The solvent will superficially soften the coating and sanding will roughen the substrate to provide a mechanical "tooth" for adhesion. While it initially appears that this approach may be slower and more cumbersome, a significant advantage in long-term performance improvement may be realized. Conversely, some Brush-Off Blast Cleaned surfaces have been seen to fail in a matter of a few months or years. This is particularly true of aged alkyd coated surfaces. In order to establish adequate surface preparation by sanding and solvent cleaning, it is wise to apply a test patch to check intercoat adhesion. This can be done by using the methods described in ASTM D5064-90 "Standard Practice for Conducting a Patch Test to Assess Coating Compatibility."

No further preparation is needed for a machine weld (photo courtesy of Lincoln Electric).

Myth: The highest quality paint job is achieved when the client demands unwavering, precise adherence to the specification.

Reality: Every coating job has its intricacies, and to assume that a written specification could adequately anticipate every possible situation that will arise on a specific project is expecting too much. A mixture of experience, common sense, and practicality has added more years of service life to a coating project than demands for unwavering compliance to specification details. We have seen a sound paint film ground and Brush-Off Blast Cleaned in order to remove "excess" dry film thickness, and a Near-White Blast Cleaned surface re-blast cleaned with a smaller abrasive in an attempt to lower a profile that was 1 mil above specification. While there may be occasions where this is necessary, such practices in the name of specification compliance are frequently more detrimental than simply accepting the work as is. Often, the best coating projects are the ones in which a quality end product is visualized, and the precise steps used to achieve the end product are of lesser importance. There is a "spirit of the specification" that may be difficult or impossible to express in the "letter of the specification." This "spirit" is often lost as the specification is enforced by an inexperienced and/or unyielding owner or by quality demand personnel. As a result, adherence to the written specification becomes of greater importance than the end product. The quality of a given painting project may be directly related to the spirit of cooperation between the owner, inspector and applicator. Reason, practicality and experience are difficult attributes to express in the form of a specification.

Myth: Blast cleaned surfaces, which are subsequently ground, must be re-profiled to achieve effective coating performance.

Reality: No data has been uncovered by us to support this notion. To the contrary, a small study undertaken by SSPC has shown that steel that had been blast cleaned, ground, and recoated performed as well in salt fog tests as steel that had been re-profiled and recoated. Limited test data and extensive field experience have shown that the small areas (approximately 1 sq. in. and smaller) that have not been re-profiled perform as well as surfaces that were re-profiled prior to coating. While the SSPC blast cleaning specifications for Commercial (SP6), Near-White (SP10), and White Metal (SP5) address the necessity of correct damage to the profile resulting from operations that take place after blast cleaning, common sense requires the recognition that small areas covering only a few square inches may be ground and not re-profiled without adversely affecting coating performance. Likewise, common sense dictates that if large areas are involved, re-profiling may be required.

Eric S. Kline is manager of technical services and James D. Machen is a consultant with KTA-TATOR, Inc. This article was adapted from a white paper presented at this year's National Symposium on Steel Bridge Construction. A future article dealing with other myths is under consideration. If you have any suggestions, please contact the authors at: KTA-TATOR, Inc., 115 Technology Dr., Pittsburgh, PA 15275 (412) 788-1300; fax 412/788-1306.

Coating Thickness Measure

A new range of Coating Thickness Gauges has been introduced by Elcometer Instruments Ltd. The Elcometer 345 is pocket-sized, yet offers a number of advanced features, including an angled screen with backlighting, so that even in the darkest corners these instruments can be easily read. Options include the entry of limits for tolerance checking and the ability to average readings. Also, one model offers memory for up to 10,000 readings.

For more information, contact: Elcometer Inc., 1893 Rochester Industrial Dr., Rochester Hills, MI 48309 (800) 521-0635; fax (313) 650-0500.

Corrosion Resistance

A high performance, single-component urethane coating from Wasser High-Tech Coatings eliminates most application restrictions for temperature, humidity and dew point. The company's coatings will cure as quickly as four hours in temperatures down to 15 degrees F, can be applied in humidities of 6 to 99 percent, and can be wetted or immersed after as little as 20 minutes after application. The coatings offer superior corrosion resistance and can be used to encapsulate red lead. They have been specified by more than a dozen state DOTs as well as more than 10 major utilities.

For more information, contact: Wasser High-Tech Coatings at (800) MC-PAYNT.

Fast-Curing Epoxies

Tnemec Co. has introduced Series 160 Tneme-Fasprime and Series 161 Tneme-Fascure. These fast-curing, high-performance coatings can be handled in two hours and recoated in three, making them ideal for shop applications. Tneme-Fasprime is a rust-inhibitive primer for steel where extra corrosion resistance is needed. Tneme-Fascure is a versatile epoxy polyamide coating that can be used as a primer, intermediate or finish coat. It protects against abrasion, moisture, and certain chemicals.

For more information, contact: Tnemec Co., Inc., P.O. box 411749, Kansas City, MO 64141-1749 (816) 483-3400; fax (816) 483-1251.

Specialized Coatings

E-Poxy Industries, Inc., offers a variety of coatings for the construction industry. A full line of EVA-POX specialized coatings are available for both new construction and renovation. The company also produces Evazote 380 ESP, an expansion joint material which handles 60% compression and 30% tension.

For more information, contact: E-Poxy Industries, Inc., 14 West Shore St., Ravena, NY 12143 (518) 756-6193.

Environmentally Conscious Coatings

Southern Coatings, a subsidiary of Pratt & Lambert, Inc., is a leader in providing environmentally conscious primers and topcoats for the steel fabricator and joist manufacturer. The Enviro-Guard line offers lead- and chromate-free primers and coatings offering superior protection against rust and corrosion.

Complete information on Enviro-Guard VOC compliant primers, as well as Chemtec 606 Water Base Epoxy Zinc Rich Primer, Chemtec 608 Inorganic Zinc Rich Primer and Dura-Pox 646 Epoxy Mastic High Build system, is available by contacting: Southern Coatings, Inc., P.O. Box 160, Sumter, SC 29151 (800) 766-7070; fax (803) 254-4833.

Steel Maintenance

Carbomastic 15 Low Odor from Carboline is recommended for the maintenance painting of rusty steel or for upgrading old coatings on steel bridges, metal buildings, and exposed structural steel. Only a single coat is required for most applications and hand or power tool cleaning often is acceptable. The product has excellent immersion resistance to both salt and fresh water and resists acid, alkali and solvent spillage.

Carbo Zinc 11 is used as a single coat protection of steel structures in weathering exposure and as a base coat for organic and inorganic topcoats in more severe services. The self-curing, inorganic base coat protects steel galvanically, eliminating subfilm corrosion.

For more information, contact: Carboline, Technical Service Department, 350 Hanley Industrial Circle Ct., St. Louis, MO 63144-1599 (314) 644-1000; fax (314) 644-6883.

Technical Information

The Technical Information Packet Service (TIP) from the Journal of Protective Coatings & Linings provides a collection of photocopied information previously published in the magazine. A typical TIP runs 50 to 100 pages and will include feature length articles as well as shorter articles. Subjects from the 30 standard TIPS include: Soluble Salts and Other Non-Visible Contaminants; Achieving VOC Compliance; Cleaning and Painting Weathering Steel; Lead Paint Removal; Generic Coating Types and Their Uses; Coating Economics; and total Shop Painting. The base cost is $5 + $0.20 per page. Customized TIPS also are available.

For more information, contact: TIPS, (800) 837-8303.

Mechanical Fastening Takes Hold

While welding has a proven track record, mechanical-fastening systems are gaining in popularity

It's not Jordan vs. Barkley. Or Apple vs. IBM. Or even Coke vs. Pepsi.

But it is a burning question facing steel-deck contractors today: Whether to weld steel decks or to attach them mechanically.

Traditionally, welding has dominated the roof deck fastening industry-and today still accounts for nearly 85% of the market. But during the past decade, a number of contractors have begun utilizing mechanical fastening-both powder-actuated pins and self-drilling screws. Both welding and mechanical fastening systems perform up to par. Both techniques have Factory Mutual code approvals, test data that support their use and plenty of loyal supporters. There are, however, differences.

The Established Route

"Whoever develops a method first sees that method adopted; in the steel deck industry, that method was welding," explained Randy Parrish, an engineering consultant from Germantown, TN.

According to Parrish, the biggest advantage of welding is that it has a proven track-record. "Because it has been around so long, welding has become natural for the people who install the deck." Another advantage is welding's relative low material cost. Welding rods are less expensive than the pins, loads and screws deployed in mechanical systems.

However, welding is not without its drawbacks—primarily the possibility of "burn-throughs."

Welding of steel decks has a long track-record of success.

Because most decks are lightweight, hot weld rods make it relatively easy for inexperienced welders to burn holes through the metal. Burn-throughs can damage not only the deck, but also the bar joist to which it is attached. When that happens, contractors must go back over the deck, repair the holes and repaint the underside. In addition, temperature differences, changes in the weather, and differences in deck material all can cause inconsistency between welds.

One solution to most of these problems is the use of weld washers, which also make the weld stronger by providing more contact between the deck and the weld-especially for sheet thicknesses less than 0.028". Unfortunately, they also can double the labor time.

Welding consistency may be further compromised by operator fatigue, which is aggravated by constant bending, stooping and working with hot equipment. Fatigue, in turn, can contribute to safety problems. According to U.S. Department of Labor data, 10% of steel-deck injuries are caused by burns.

One other drawback to welding is that visual inspections are difficult. The only way to inspect welds are with destructive tests, which is both expensive and time consuming.

Decreasing Operator Error

Mechanical-fastening systems eliminate many of the disadvantages that occur with welding-the lack of consistency and burn-throughs. Automated systems also decrease the margin for operator error, and in turn increase the consistency both between mechanical attachments and between jobs. The greater consistency translates into lower safety factors. While screws and pins have a typical safety factors of 2.35, welds usually have a factor of 2.75, according to the Steel Deck Institute.

However, despite the higher safety factor, in some cases welds will have a higher design shear than mechanical fastenenings when similar fastener patterns are used.

One often-cited advantage for mechanical-fastening systems is the lower operator fatigue factor and the lower risk of back injury, especially for those systems that allow the operator to stand up during the fastening operation.

While pins and screws are more expensive than welding rods, mechanical fastening systems generally require a lower up-front cost than welding systems. Some mechanical systems do not even require hoses. Inspections also are easier with mechanical fasteners. A quick glance will generally show

Proponents of mechanical fastening claim the technique saves time and therefore cuts costs.

whether a screw is properly installed.

Some contractors have been hesitant to use mechanical fasteners in the past due to perceived safety problems. These related primarily to direct systems, where the pin was "shot" from the fastening gun at a high velocity. These systems are no longer sold in the U.S. and have been replaced with indirect fastening systems, which prevent stray pins from randomly penetrating the deck.

According to its proponents, the biggest advantage of mechanical systems is in reduced time. While experienced welders may make a weld every 10 seconds, a less experienced welder may take twice as long. "Though work habits and operator expertise play big roles, mechanical systems are generally much faster once the contractors learn how to operate the equipment."

And because the time it takes to mechanically fasten deck does not vary as much as it does with welding, contractors are able to calculate their bids more precisely.

Deck Products

Custom Metal Deck

United Steel Deck has enhanced its ability to produce custom deck systems by networking with the affiliated companies of Nicholas J. Bouras, Inc. Special finishes, such as plasticol, or materials such as stainless steel, are being used to produce custom decks and panels that solve durability and environmental problems caused by some industrial atmospheres. Special finishes combined with the roll forming and bending capabilities of United Steel Deck, Inc., can provide solutions to most unique decking demands.

For more information, contact: United Steel Deck, Inc. (Nicholas J. Bouras, Inc.), 475 Springfield Ave., Summit, NJ 07902-0662 (908) 277-1617; fax (908) 277-1619.

LRFD Expert System

The Steel Deck Institute has released a new expert design system, based on LRFD, for composite and non-composite beams and girders with steel deck. This software is part of the design Advisor expert system developed for the AISC and the SDI by Structural Engineers, Inc., of Radford, VA. Complete bay design as well as individual beams and girders can be investigated and optimized for the least cost. Design tables in the SDI format can be produced using any combination of material properties. Detailed reports are produced showing vibration analysis and provide stud spacing. Concentrated loads and line loads can be applied in addition to uniform loading. Cost is $295.

For more information, contact: Steel Deck Institute, P.O. Box 9506, Canton, OH 44711 (216) 493-7886.

ITW Buildex

The Autotraxx ICH Deck Fastening System is used to attach steel deck in a stitch or structural steel application. The system has two components: a stand-up tool that includes a screwgun, special fastener guidance system, depth sensitive nosepiece and unique drive socket; and Traxx fasteners with an ICH (Internal Cone Head) design.

The fasteners have either a Traxx/1 point for stitch applications or a Traxx/5 point for structural attachments. The design allows the tool drive pin to engage securely with the fastener for consistent drilling.

For more information, contact: ITW Buildex, 1349 West Bryn Mawr Ave., Itasca, IL 60143 (708) 595-3549.

Power Distribution

Walker Division of Butler Manufacturing has introduced a new concept in PLEC distribution for steel-framed buildings-one that combines the triple-service capacity and aesthetic appeal of an in-floor system with the up-front economy of a poke-thru system. The new, low-cost Presource III bottomless activation modules are installed in a grid pattern on standard steel deck before the concrete pour, providing access to services in a predetermined pattern. Activation costs are deferred until the time of fit-out, and activations are accomplished in much the same way as with a poke-thru, except that no core drilling through structural concrete is required.

Also new from Walker is a line of service fittings in flush, pedestal and multiplex configurations for new construction or renovation.

For more information, contact: Mary Williams, Walker, P.O. Box 1828, Parkersburg, WV 26101 (800) 222-PLEC.

Fastening System

Hilti Inc. has designed a new powder actuated fastening system for the fast, economical attachment of metal roof and floor decking. The DX 750 fastening system offers such features as single-handed operation, a power regulator and an optional fastener magazine. While 15% more powerful than the DX 650, the new introduction is 10 lbs. lighter and can be used in temperatures ranging from -13 degrees to 113 degrees F. It also features a silencer to reduce noise levels.

For more information, contact: Hilti Customer Service (800) 879-8000.

Bridge Decking

Grid Reinforced Bridge Decks, comprised of both a fabricated steel grid and concrete, are lighter than traditionally reinforced decks and are still strong enough to withstand high traffic volumes over long periods of time (some applications are already in their sixth decade of service). The Bridge Grid Flooring Manufacturers Association maintains a computerized data base of grid related research and welcomes inquiries. The association can provide design recommendations and also publishes a newsletter.

For more information, contact: BGFMA, 231 South Church St., Mt. Pleasant, PA 15666 (412) 547-2660.

Bridge Deck Form

Epic Metals Corp. has introduced MAXSPAN BRIDGE DECK FORM, an entirely new concept in the design of permanent metal deck forms for bridge deck slabs. The forms are designed to accommodate today's wider girder spacing with greater efficiency at spans ranging from 10' to 18'. They provide a flat top surface, which reduces concrete usage and slab dead load. This results in allowing virtually all the concrete to contribute to the structural strength of the slab.

For more information, contact: Robert Paul, Product Engineer, Epic Metals Corp., Eleven Talbot Ave., Rankin, PA 15104 (412) 351-3913.

PMD Form

Bowman Metal Deck offers permanent metal deck forms for bridge construction. According to the manufacturer, PMD forms offer three distinct advantages: time savings; cost reduction ($4/sq. ft. estimated savings compared to wood forms); and increased safety (installation of a PMD form provides an immediate and safe working platform for all crews). In addition, PMD forms provide a lower cost means of using more widely spaced girders, which results in more cost effective steel framing. Some research also indicates that stay-in-place forms may slightly decrease deck cracking.

For more information, contact: Bowman Metal Deck Division, ARMCO Inc., P.O. Box 260, Pittsburgh, PA 15230-0260 (412) 429-7560; Fax (412) 276-6057.

Bridge Bolts

Mid-South Bolt and Screw, a distibutor of all types of fasteners for the structural steel industry, is a specialist in the manufacture of anchor bolts. The company has worked closely with several DOTs, the FHWA and various bridge fabricators to develop expertise in fasteners for bridges. Mid-South supplies domestic bolts with full traceability, lot heat certification, lot integrity and in-house testing.

For more information, contact: Tim Weaver at (800) 366-BOLT or Randy Graves at (800) 251-3520.

Inspection Walkways

Heavy Duty Grip Strut bridge inspection walkways are suspended beneath bridge deck to enable clos inspection of load-carrying members. The well-made catwalks span 24' openings with minimal deflection, which reduces the need and expense of extra supports. Also, gravel, mud, snow and ice fall through large diamond-shaped openings. Choices include 9, 10, or 11 gauge grating with serrated or non-serrated steel, and widths up to 36" with 5" integral toe-boards, which eliminate extra welding.

For more information, contact: GS Metals Corp., R.R. 4, Box 7, Pinckneyville, IL 62274 (800) 851-9341 or (618) 357-5353 inside Illinois.

Grid-Reinforced Concrete Bridge Decks

The nation's bridge engineers are rediscovering Grid Reinforced Concrete Bridge Decks. Developed and first used in the 1930s, this unique construction method results in a bridge deck, comprised of both a fabricated steel grid and concrete, that is lighter than traditionally reinforced decks, and highly armored to withstand high traffic volumes over long periods of time. In fact, some of the initial installations are well into their sixth decade of service. Supplied in large panels complete with shop-installed concrete formwork, Grid Reinforced Decks are ideal for fast-track redecking projects and also are well suited for precasting. There are a wide range of deck choices available to the designer, with deck profiles from 3" to 10", and weights ranging from 50 psf to 90 psf.

For more information, contact either: Bridge Grid Flooring Manufacturers Association, 231 South Churck Street, Mt. Pleasant, PA 15666 (412) 547-2660; or the Exodermic Bridge Deck Institute, P.O. Box 374, Westwood, NJ 07675 (201) 666-5116.

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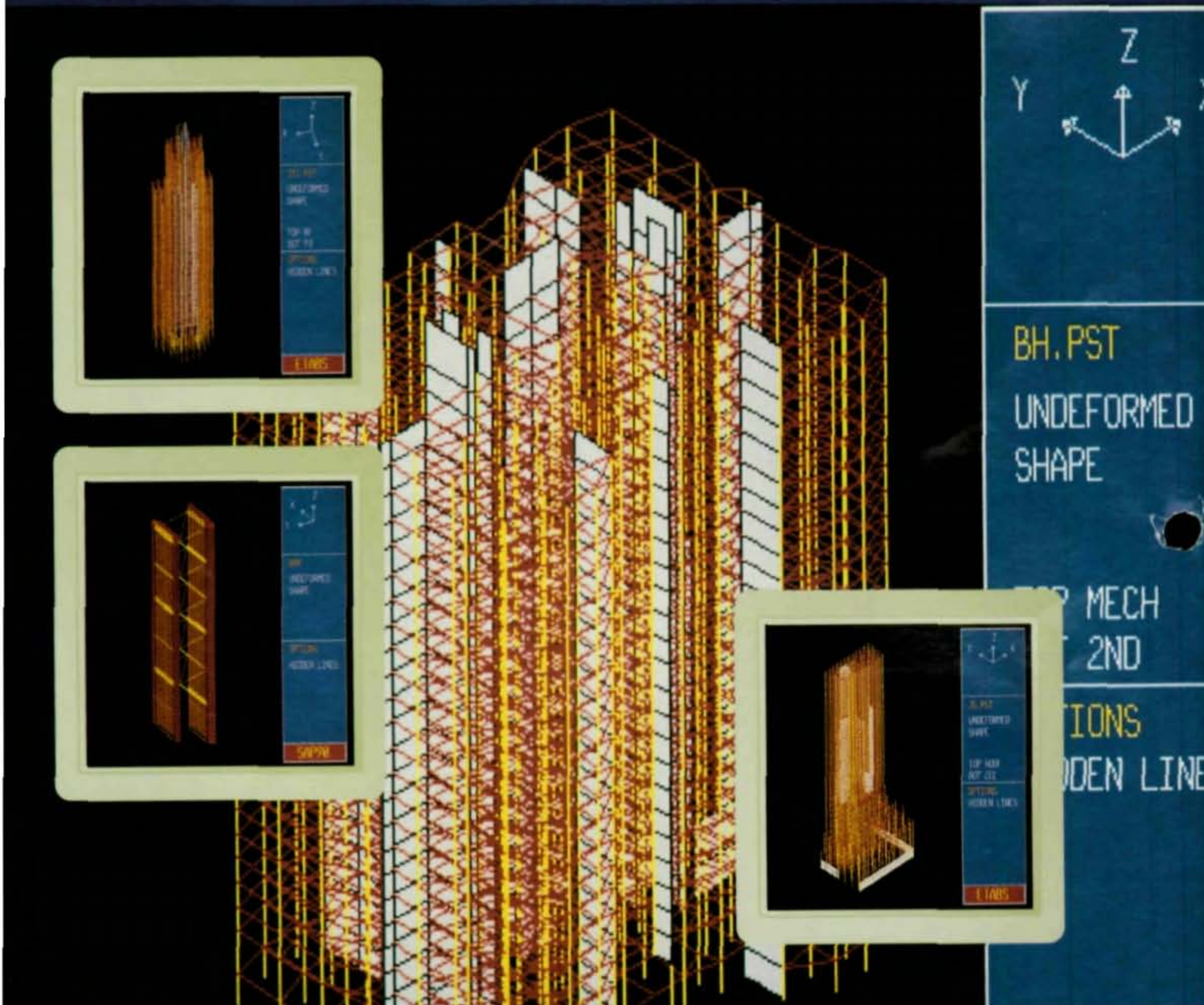

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