MODERN STEEL CONSTRUCTION

September 1995

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deck design data sheet #23

ASTM specification change

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

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

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

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

Old Grade Name Under A446
grade A
grade C
grade E

⇩

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

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

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

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

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

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

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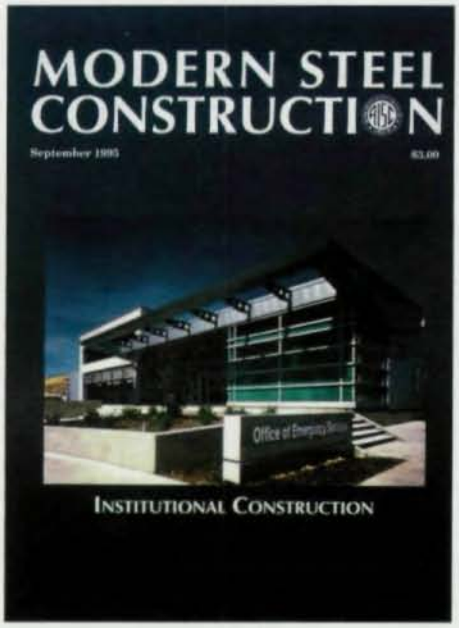
MODERN STEEL CONSTRUCTION

Volume 35, Number 9 **September 1995**



Regardless of earthquakes, fires or other disasters, the new Emergency Operations facility for Alameda County was designed to remain operational. The story behind this project begins on page 20.

FEATURES

DEPARTMENTS

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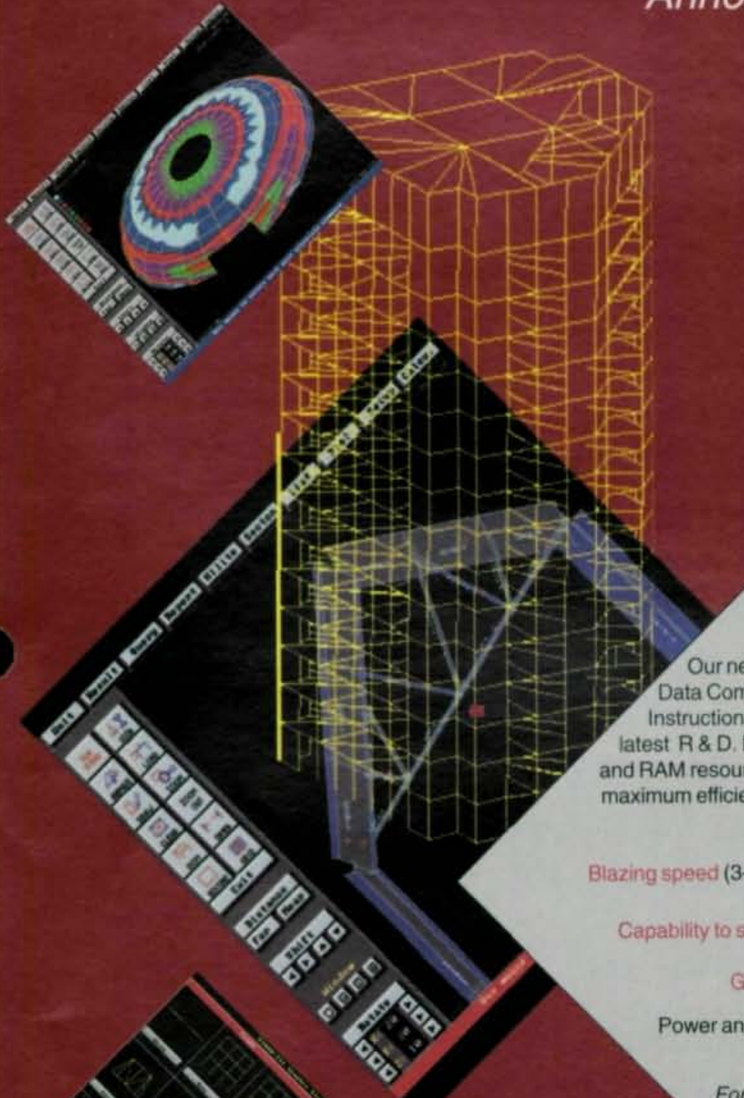

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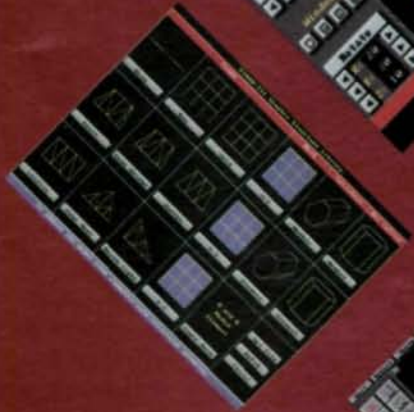

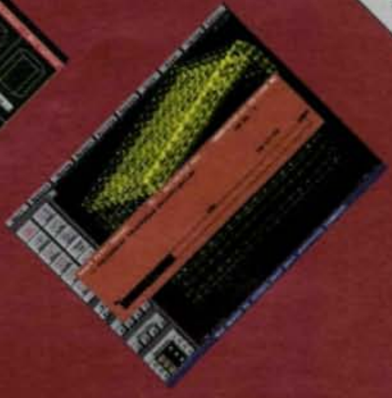

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Better Design

GOOD DESIGN IS MORE THAN SIMPLY MAKING SOMETHING LOOK GOOD. I was reminded of that recently while reading a book, *The Design of Everyday Things* by Donald A. Norman, recommended to me by my brother-in-law, a human factors engineer.

The author makes the point that beyond looking good, and even beyond functionality, a good designer creates items that are easy to use. Two examples from early in the book—doors and phones—struck my fancy, the first because our offices are afflicted with the problem of a poorly conceived portal, and the second because our offices are blessed with a well-designed voicebox from AT&T.

The main doors to our office are perfect examples of bad design as cited in the book. The doors are architecturally elegant: Two glass slabs with concealed hinges and artfully identical handles on both sides. They are also exceedingly annoying. Anyone with a rudimentary knowledge of fire safety would expect entry doors to open outwards. The problem with these doors, however, goes beyond their inwards opening. The calamity is that there are no visual clues to indicate whether to push or pull. Even people who have worked here for several years often try to open them the wrong way.

In contrast, our phones are wonderful. The problem with many phones, according to Norman, is that functions are arbitrary. For example, how do you put someone on hold or how do you transfer a call? Often, these functions involve punching in some arcane code. Likewise, how do you tell if someone is on hold? Too often there is no indication on a modern business phone. Our phones, though, are remarkably well designed. A solid green light shows an active line; a flashing green light shows someone on hold. To put someone on hold, press a button clearly labeled "hold". To transfer, press a button labeled "transfer", dial a new number, and press "transfer" again. In his book, Norman contrasts the average phone to the average car. If you walk into someone else's office, how likely are you to be able to use all of the "advanced" features on his phone? But if you borrow a neighbor's car, it's likely that all of the controls will be readily obvious.

Whether you're designing phones or buildings, doors or shop layouts, cars or computer programs, it's crucial to consider how a user will react. The clearer the design, the less likelihood for error. And in our business, errors can all-too-easily escalate from annoyance to tragedy. **SM**

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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:

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

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

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

Treatment of Simple Shear Connections Subject to Combined Shear and Axial Forces

AS ITS NAME IMPLIES, A SIMPLE SHEAR CONNECTION IS INTENDED TO TRANSFER SHEAR LOAD OUT OF A BEAM while allowing the beam to act as a simply supported beam. The most common simple shear connection is the double angle connection with angles shop bolted or welded to the web of the carried beam and field bolted to the carrying beam or column. This note will deal with this connection.

Under shear load, the double angle connection is flexible regarding the simple beam end rotation, because of the angle leg thickness and the gage of the field bolts in the angle legs. The AISC Manuals (A.S.D. 9th Ed. p. 4-9, LRFD 2nd Ed. Vol. II, p. 9-12) recommend angle thicknesses not exceeding $^5/_8$ in. with the usual gages. Angle leg thicknesses of $^1/_4$ in. to $^1/_2$ in. are generally used, with $^1/_2$ in. angles usually being sufficient for the heaviest load. When this connection is subjected to axial load in addition to the shear, the important limit states are angle leg bending and prying action. These tend to require that the angle thickness increase or the gage decrease, or both, and these requirements compromise the connection's ability to remain flexible to simple beam end rotation. This lack of connection flexibility causes a tensile load on the upper field bolts which could lead to bolt fracture and a progressive failure of the connection and the resulting collapse of the beam. To the author's knowledge, there has never been a reported failure of this type, but it is perceived to be possible.

Even without the axial load, some shear connections are perceived to have this problem under shear alone. These are the single plate shear connections (shear tabs) and the Tee framing connections. Recent research on the Tee framing connections has led to a formula (AISC Manual LRFD 2nd Ed., Vol. II, p. 9-170) which can be used to assess the resistance to fracture (ductility) of double angle shear connections. The formula is

$$d_{b\,\min} = 0.163 t_f \sqrt{\frac{F_y}{b}\left(\frac{b^2}{L^2} + 2\right)}$$

where:

d_b is the minimum bolt diameter (A325 bolts) to preclude bolt fracture under a simple beam end rotation of 0.03 radian, and

t_f is the angle thickness

b is the distance from the bolt line to the k distance of the angle (see Fig. 1)

L is the length of the connection angles.

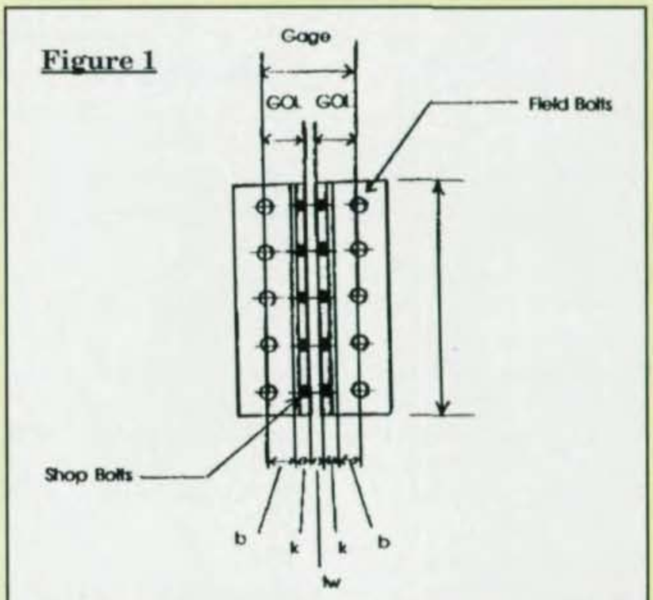


Figure 1

Note that this formula can be used for ASD and LRFD designs in the form given above. It can be used to develop a table (see Table 1) of angle thicknesses and gages for various bolt diameters which can be used as a guide for the design of double angle connections subjected to shear and axial tension. Note that Table 1 validates AISC's longstanding recommendation (noted above) of a maximum 5/8 inch angle thickness for the "usual" gages. The usual gages would be $4^1/_2$ to $6^1/_2$ in. Thus, for a carried beam web thickness of say $^1/_2$ in., GOL will range from 2 in. to 3 in. Table 1 gives a GOL of $2^1/_2$ in. for $^3/_4$ in. bolts (the most critical as well as the most common bolt size). Note also that Table 1 assumes a significant simple beam end rotation of 0.03 radian, which is approximately the end rotation that occurs when a plastic hinge forms at the center of the beam. For short beams, beams loaded near their ends, beams with bracing gussets at their end connections, and beams with light shear loads, the beam end rotation will be small and Table 1 does not apply.

Table 1
Estimated Minimum Angle Gages (GOL) for A36 Angles and A325 Bolts for Rotational Flexibility

ANGLE THICKNESS (in.)	MINIMUM GAGE OF ANGLE (GOL)*		
	$^3/_4$ in. dia. bolt (in.)	$^7/_8$ in. dia. bolt (in.)	1 in. dia. bolt (in.)
$^3/_8$	$1^3/_8$	$1^1/_4$	$1^1/_8$
$^1/_2$	$1^7/_8$	$1^5/_8$	$1^1/_2$
$^5/_8$	$2^1/_2$	$2^1/_8$	$1^7/_8$
$^3/_4$	$3^1/_4$	$2^{11}/_{16}$	$2^5/_{16}$
1	6	$4^7/_{16}$	$3^1/_2$

*Driving Clearance may control minimum GOL.

The design of double angle connections subjected to shear and axial tension, can be accomplished as shown in the following AISC publications.

1. AISC Manual (ASD 9th Ed.), p. 4-94, Ex 34, where the beam web plays the same role as the gusset of this example.
2. AISC Manual (ASD 9th Ed./LRFD 1st Ed.) Vol. II, pp. 7-123 and 124, pp. 7-125 and 126, pp. 7-167 through 170.
3. AISC Manual (LRFD 2nd Ed.), Vol II, pp. 11-38 through 42.

While the design is being completed in the usual way as show in these publications, Table 1 can be consulted to guide the design, if appropriate.

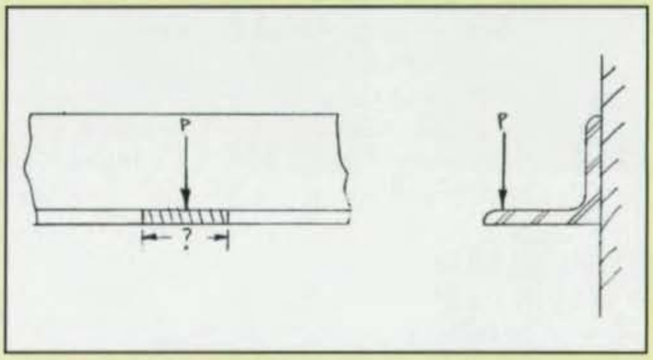
W.A. Thornton
Roswell, GA

New Questions

Listed below are questions that we would like the readers to answer or discuss.

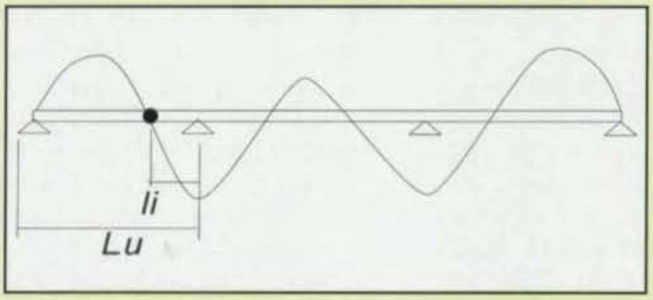
If you have an answer or suggestion please send it to the Steel Interchange Editor, Modern Steel Co nstruction, One East Wacker Dr., Suite 3100, Chic ago, IL 60601-2001.

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 considering a point load on the standing leg of an angle, what provisions are there for determining the effective allowable member width?

David Chida
Electric Power Door
Hibbing MN



When is it conservative to select the beam shown assuming the unbraced length of l_i and $C_b = 1.0$?

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STEEL NEWS

CORRESPONDENCE

Dear Editor:

While most of Mr. [William G.] Zimmerman's ideas (May 1995, pp. 30-33) are good practices and will reduce project costs, there are certain items that are not universally true and could mislead designers. For instance, field welding of beam-to-column moment connections oftentimes results in a lower overall cost to the project than bolting. This is particularly true in larger buildings where the moment loads can be very high. As you can imagine, shipping a column with long wingplates extending off of each column flange can make the width of the columns in excess of 9 ft. This width will result in permit loads which, over long distances, can be quite expensive. They also limit the amount of material you can place on a truck and the plates are subject to damage in transit. Another problem with the big plates is their large hole patterns, which can be difficult to make up in the field without reaming. The common response to avoid reaming is to use oversize holes, but this makes the plates even bigger by adding additional bolts due to lower bolt values. So, while it true that welding is more expensive than bolting in the field and does require higher-skilled people, in many areas of the country the skilled welders are available and the process is very comparable to bolting. When coupled with the savings in the shop, field welding can yield a very positive and economical result to the job, and should not be rejected out of hand.

Further, fillet welds are not always the cheapest. On very large welds, such as column splices, a partial penetration weld often is a more economical alternative than a large fillet weld.

We agree with Mr. Zimmerman that on bigger projects where large gusset plate connections for bracing, etc., are required to handle large kip loads, use of twist-off bolts, especially in multi-ply connections, is not economical. In trying to snug up multiple plies with large diameter twist off-bolts bolts, oftentimes the bolt twists off before the connection can be made tight and torqued to the proper tension. We have found success with the twist-off bolts in lighter steel and two-ply connections. Again, it's a matter of preference, but people should be aware of the subtleties and remember that, unfortunately, "one-size" does not always "fit all."

Michael J. Senneway
V.P. & General Manager
SMI-Owen Steel Company

Dear Editor:

First of all, let me say that *Modern Steel Construction* is a very interesting and informative magazine that this reader has enjoyed for several years.

However, the Steel Joist Institute feels that a response is warranted regarding your article titled "An Alternative to Joist Girders," appearing in the June, 1995 issue. Yes, joist deliveries are extended at the present time and, although joist manufacturers are hoping for a prolonged period of near capacity production, the present situation must be viewed cautiously. One only has to remember back a few months to recall joist deliveries in the one-to-two-week range, and if history is any guideline, shorter delivery times will prevail again all too quickly.

The Steel Joist Institute does take exception to the article's opening sentence. The writer's statement referencing only two major producers is a "major" misnomer. Steel Joist Institute members operate 23 joist plants that can readily provide steel joists in sufficient quantity to meet the needs of the industry during times of normal demand.

Secondly, the article contains an error in product terminology. Steel Joist Institute products lines include the "K" Series Joists, the "LH/DLH" Series Joists and the Joist Girders. We do not have "K" Series Joist Girders.

We feel that one of the primary goals of a quality technical publication, such as *Modern Steel Construction*, is accuracy of content. Our critique is an effort to support that goal.

R. Donald Murphy
Managing Director
Steel Joist Institute

CORRECTION

The telephone number for Chaparral Steel was incorrectly printed in the July Steel Producers listing. The correct number is: 800/527-7979.

BOOK REVIEW: SSRC VOLUME MARKS 50 YEARS OF PROGRESS

By Rober F. Lorenz, P.E.

Anniveraries are significant not just as an indicator of staying power, but also as an opportunity to reflect on the past and also to consider the future. The Structural Stability Research Council scores on both accounts in publishing the Proceedings of its 50th Anniversary Conference held last summer at Lehigh University.

The 400+ pages include 27 papers written by a broad range of independent experts, all connected by their technical leadership in the stability of steel structures. The papers include such topics as laterally unsupported beams, curved girders, inelastic framing systems, tubular members, shells, stability of angle struts, stability under seismic loading, influence of connection details on stability, new constructional steels, and other equally pertinent topics.

In reviewing the book, it seemed there was always some succint, easily-understood technical information in each paper, which indicated a scrupulous job of compilation and editing. Each separate work literally provides a tutorial that can be logically followed with plenty of figures and graphs from research to make a clear and rational statement to the reader.

An example of the above can be found in the paper by Takanashi and Nakashima on stability considerations on seismic performance of steel structures. Starting with the simple differential equations of single degree of freedom systems, they develop the concept of energy dissipation, and from this create simple expressions for both system and member ductility. More important, they are able to deduce preferred trade-offs between strength and deformation, so the reader can get an intuitive feel for particular decisions of varying ductility in a given structural framework.

Separate papers by Galambos and Kennedy provide a historical perspective of the early years of the Council, which originally was known as the Column Research Council. Its early task was to resolve the many variables in the design of compression members including crookedness, eccentricity of loads, end conditions, drift and residual stresses. The result was the column formulas adopted by AISC in their 1962 Specification, which introduced the well-known K-factor for the first time.

As mentioned earlier, the technical content of the book is compelling, and the diversity of the subject matter holds the interest of the reader. Because of the recent experience of destructive earthquakes worldwide, particular interest in papers by Popov, Fukumoto and Leon provide some direction for new research in that direction.

As far as the future is concerned, the paper by White is stimulating. He sees computer technology eventually creating tools to evaluate systems, where within each system, member design would be predetermined. This would open up new pathways in research in the development of reliable computational models.

Robert F. Lorenz, P.E., is AISC 's Director of Educatation and Training.

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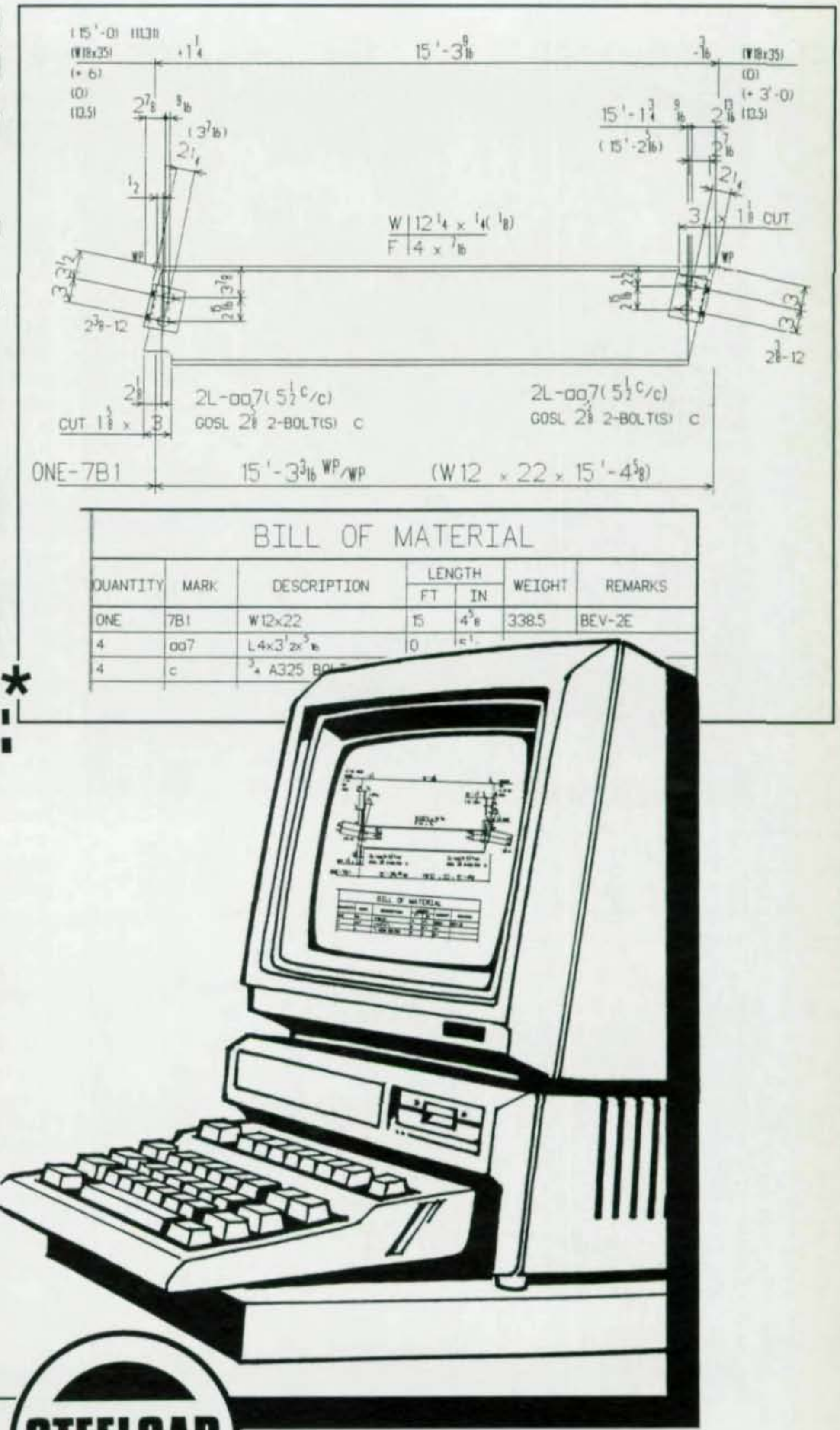

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The Man Of Steel

For more than 40 years, Bob Disque has been one of the leaders in steel design

By Katie Brown

In 1944, when Robert Disque was a senior in high school, he wrote a theme paper about the collapse of the Tacoma Narrows suspension bridge four years earlier. Though he knew nothing about designing and building bridges, he instinctively felt that "Galloping Gertie" had been doomed from the start.

"In reading the material, I knew that the design engineer on that bridge had done his mathematics and all his calculations, and they all worked according to his theory," Disque explained. "But the proportions of the bridge far exceeded anything that had been done before. I believe the engineer was so enthralled by his mathematics that he lost the overall view."

Disque, now 68, has built his career on separating the slag from the metal in structural steel construction. A former director of building technology and chief engineer of the American Institute of Steel Construction, Disque has helped scores of fabricators and engineers find solutions to the structural problems they encounter. While known as a demanding designer, his benchmark is not just excellence in design, but also practicality. And it is not uncommon for him to advise designers to talk with—and listen to—steel fabricators. But beyond the years of dispensing advice, Disque has made his mark on the industry through his contributions to the mathematical foundations for the integrity of simple shear connections and his aid in the development of bolting standards. And he's not through yet—he's now working as a consulting engineer in Connecticut and just last year he co-authored a new textbook on LRFD. He also remains something of an industry gadfly, sparking debate on topics such as the role of computers in design engineering and the use of partially restrained moment connections.

"He's pretty much known as 'Mr. Steel'," said Barry Barger, vice president of AISC-member Southern Iron Works, Inc. "Bob knows fabrication and he's an engineer. To him, there's no such thing as an impossible problem."

Practice Vs. Theory

Over the year's, Disque has devoted his energy to proving the approaches that other engineers and fabricators have developed through experience. Upon graduating in 1950 with a degree in civil engineering from Northwestern University, he took a job with a design firm and quickly learned that the real world was different from the classroom.

"I was making a [semi-rigid] design, and my boss asked me what I was doing," Disque recalled. "I told him, and he said, 'We don't do it that way. We do it this way.' I said, 'That's not correct. That's not what we were taught in school.' And he told me, 'You can do it the way we do it, or you can stay home tomorrow and start looking for another job."

Disque designed it the way he was told using simple shear connections—an approach common in the engineering offices of the 1930s and '40s, but one which had not yet been introduced to the classroom since the scientific underpinnings of the method had not yet been fully developed. "The engineers in those days didn't understand the mathematics of it, but their instincts were superb," Disque said. "They instinctively knew [PR construction] was going to work." And work it did on such superb projects as the Empire State Building and the United Nations Secretariat, which Disque likens to a sail: "It gets hurricanes thrown at it and performs very well. The bottom line is behavior—it has worked."

After realizing success on his first assignment, Disque became an advocate of simple shear connections. And then, in 1964, he became their champion. He was working as the chief engineer at AISC and the City of New York was having its building code rewritten "by people who had strong academic backgrounds but little practical background," Disque said. Because simple shear connections had never been mathematically rationalized, they were not acceptable under the proposed code.

"I heard about it, and that's what prompted me to do some real hard thinking to come up with a rationalization," Disque said. The result of his work was a paper, "Type 2 Construction with Wind Connections," which proved the science and compelled the code reviewers to allow simple shear connections.

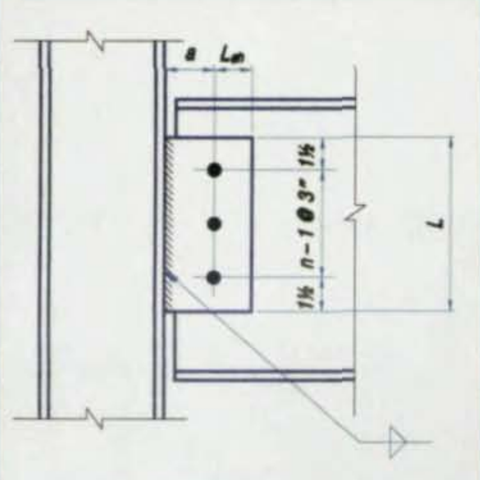
Disque has helped fabricators and engineers find solutions to their problems on a smaller scale, too. Southern Iron Work's Barger recalled the day in 1978 when he met Disque—after Barger had come from a long, discouraging meeting with a structural engineer about end-plate moment connections on a job. "They were trying to convert from a wing-plate system to an end-plate moment system," Barger said. "They had a pre-cast plank that sat on the top flange of the beam, and the wing-plates interfered with them. The engineer didn't know how to make an end-plate work in a roof application." Before flying home to Virginia, Barger decided to stop in at AISC's offices, then located in New York, to see if there was a technical person there who could help him find an answer. He was introduced to Disque and presented his problem. "Bob said, 'Oh, that's simple,' and he pulled out a book and said, 'Here's your answer. Who's the engineer?' He called the engineer, told him the solution, and then said to me, 'What else do you need?' The whole thing took about 10 minutes," Barger recalled.

In 1964, when Disque was chief engineer at AISC, he wrote his groundbreaking paper on simple shear connections in an effort to have them accepted in New York City's new Building Code.

Snug-Tight Bolts

One of Disque's major contributions to the steel construction industry was his work on bolting. In the mid-1980s, a fabricator was suing a general contractor who refused to pay him on the grounds that the bolts in the project had not been tightened fully with impact wrenches according to project specifications. Disque had read much of the research on the basic behavior of bolts and understood the problem.

When high-strength bolts began to replace rivets as fasteners on steel construction projects, the standards for tightening did not change, even though the bolts were $2^1/_2$ times as strong (100,000 psi compared with 40,000 psi for rivets). While Disque conceded that if the fabricator had not fully tightened the bolts he had violated the letter of the contract, but he contended that it didn't make a difference to the intent of the contract, which was to ensure a safe building. Ultimately, the suit was settled out of court.



Minimum Plate Thickness	
n	$t_{p\,min}$, in.
2–5	¼
6–7	5/16
8	3/8
9	7/16

Simple shear connections, which were commonly used more than six decades ago, take the form shown above in the latest AISC Manual of Steel Construction.

"But I knew at the time that it didn't make any difference from a structural point of view" if the bolts were fully tightened or snug-tight. "As a result, I proposed that under certain circumstances, bolts don't have to be fully tightened," Disque said.

"When we go from one technology to another, an awful lot of the technology you had before carries over," he explained. "If you look at the first airplane that the Wright brothers built, it looks almost like a bicycle. The first cars look like horse-drawn carriages. Going from rivets to bolts, the bolts inherited a lot of things that did not apply, and it took a while before bolts could stand on their own without suffering their inheritance."

Many academics fought against changing the regulations, Disque said, but they couldn't come up with a convincing argument against the proposal. Ultimately, in 1985, the Research Council on Structural Connections adopted the change. The benefits to erectors and developers has been tremendous: the cost of fastening bolts dropped from about $30 per bolt to $10 per bolt in 1985 dollars. "You don't have to have an impact wrench with air compression and hoses. You just have an iron worker with a wrench banging them up snug," Disque said.

While Disque has worked to make sure that regulations and standards continue to evolve, he also is a proponent of educating engineers about the current rules. In the early 1960s he initiated a lecture series that has become a staple of AISC's work and continues today with "Advances in Structural Steel Design" seminars. "There are a lot of things engineers don't get in college. AISC is in a position to educate them so they can design in steel better," said Disque, who spent a year as an associate professor of civil engineering at the University of Maine.

Disque's latest contribution to the structural steel industry is a textbook, "Load and Resistance Factor Design of Steel Structures" (Prentice Hall), that helps both students and practicing professionals switch over the LRFD. "It's a much more logical way of looking at loads," Disque said. "And in some cases, there is significant savings because the old way of doing it was so clumsy that it over-designed way beyond the point of reasonable safety."

That's Just Plain Wrong

A stickler for accuracy, detail and thoroughness, Disque's temper often flares when he encounters shoddy workmanship. "When you show Bob a bad detail or a lousy set of detail drawings, it's like a firecracker going off," said Barger. "He'll say, 'What are they teaching these people? or 'Where's this guy been?' One of his favorite statements is: 'That's just plain wrong'—followed by about 20 exclamation points. Bob is so intelligent but he can't imagine that others can't keep up with him."

Disque also is a passionate debater and will initiate professional arguments given the slightest opportunity, said friends and former colleagues. Horatio Allison, a well known engineering consultant now retired and a close Disque friend, said he enjoys debating Disque. "We get in arguments, and I'll take the adversarial viewpoint," Allison said.

"But he's outstanding. He has a wonderful grasp of knowledge for the design of steel structures."

Disque's latest venture into controversy is his criticism of what he sees as the dependency of design firms on computers. He also is concerned that the technology needed to advance the newest approach to connections—Partially Restrained Moment Connections—is not yet practically viable.

As a consultant for the last four years with Besier Gibble Norden, Consulting Engineers Inc., in Old Saybrook, CT, Disque serves as an expert witness in litigation, consults on production design work and conducts mandated peer reviews of designs by other engineering firms. "I've been around so long that it's rather easy for me in looking over drawings to spot something that looks strange or bad. I've caught undersized beams that were computer-designed. The computer assumed this particular beam was braced, and it wasn't. Now, they might have caught that during construction. They would have put the beam up and before it was loaded, they would have seen it was flexible and too light. The original designer could have been in big trouble."

Cautious About Computers

Disque is concerned that the computer, while an essential tool in design and construction, can create errors that manual calculations might have avoided. "Computers take the number-crunching out, but when you sit down to frame a building, you should do that before you engage a computer. Only after you do that basic framing do you put everything into the computer to see whether it will work or not," he said.

"Computers have improved things," he acceded. "I remember in the 1950s I was working for a consulting engineering firm and was doing some calculations that I worked on for about six months, routine calculations to see what the best way of doing this was. That can be done in a matter of minutes now. And you can get something better. But computers also can get you into some dangerous situations."

Design engineer Jim Wooten, a good friend of Disque's from Little Rock, AR, came up with a "law" and two corollaries that Disque likes to apply to computerization: Wooten's Third Law: The acquisition of uncommon knowledge inhibits the application of common sense. "When you get so smart with all your mathematics and models, you forget your common sense. The corollaries are: 'Steel is smarter than the engineers who design it' and 'the computer renders obsolete the necessity of rationalizing and simplifying problems or even of understanding them.' But I don't think that the second corollary has to be. You can use judgment with a computer."

Disque's concerns about computer dependency reflect on his feelings about Partially Restrained connections. With PR connections, the beam and girder connections are assumed to possess a dependable and known moment capacity that is not as rigid as Fully Restrained connections but are not as flexible as Simple Shear connections. The trick is knowing where the ideal point between rigidity and flexibility lies—and that's where the computer comes into play.

While Disque has his doubts about the practical application of PR connections today, his friend and former colleague Robert Lorenz, director of education at AISC, has made convincing arguments otherwise. "Lorenz says that PR connections are a lot closer to reality that I would. It takes a very, very sophisticated computer program." However, Disque does concede that PR connections are the wave of the future—it's just a matter of when technology will catch up with theory.

Another point on which he is adamant is the relationship between fabricators, engineers and architects. "Fabricators are very savvy about a lot of things. Even though they may or may not have an engineer on their staffs, they have saved an awful lot of buildings from disaster because of engineers making mistakes," Disque said. "Fabricators and structural engineers have a lot in common. They are allies, but they often get into confrontations. I listen to fabricators. I think they know what they're doing. Engineers who don't listen to fabricators are doing so at their own risk."

Katie Brown is a freelance writer based in Evanston, IL.

CONTROLLING DISASTERS

A new Emergency Services facility is designed to remain operational despite any disaster it may experience

By William A. Andrews, S.E.

CIVIL EMERGENCIES PLACE EXTRAORDINARY DEMANDS ON LOCAL GOVERNMENTS to manage and direct the agencies that respond to such events. Timely and appropriate actions in response to disasters such as earthquakes, conflagrations, floods, tornados, air crashes and hazardous materials accidents save lives, protect property and accelerate the recovery process. Clearly, the requirements of government services during such emergencies differ significantly from their daily operations. Emergency response calls for close coordination of government officials, support staff, and response teams through direct lines of survivable communication. Detailed information must be quickly gathered and disseminated to the appropriate decision-making officials, who then mobilize and direct available assets to meet critical needs. A new generation of building types called Emergency Operations Centers (EOC) is being developed and built to provide centralized coordination and direction of local and regional emergency response and recovery.

In the San Francisco Bay Area, Alameda County learned firsthand about the complexities of emergency response management through two recent natural disasters, the 1989 Loma Prieta earthquake and the 1991 East Bay Hills firestorm. With the recent completion of the Alameda County Office of Emergency Services/Emergency Operations Center (OES/EOC), the county is now in a far better position to prepare for and respond to such emergencies. The facility provides state-of-the-art audio/visual communications and data sharing technology to county personnel experienced in disaster management. The project cost, including all the high-tech systems, is $6.5 million.

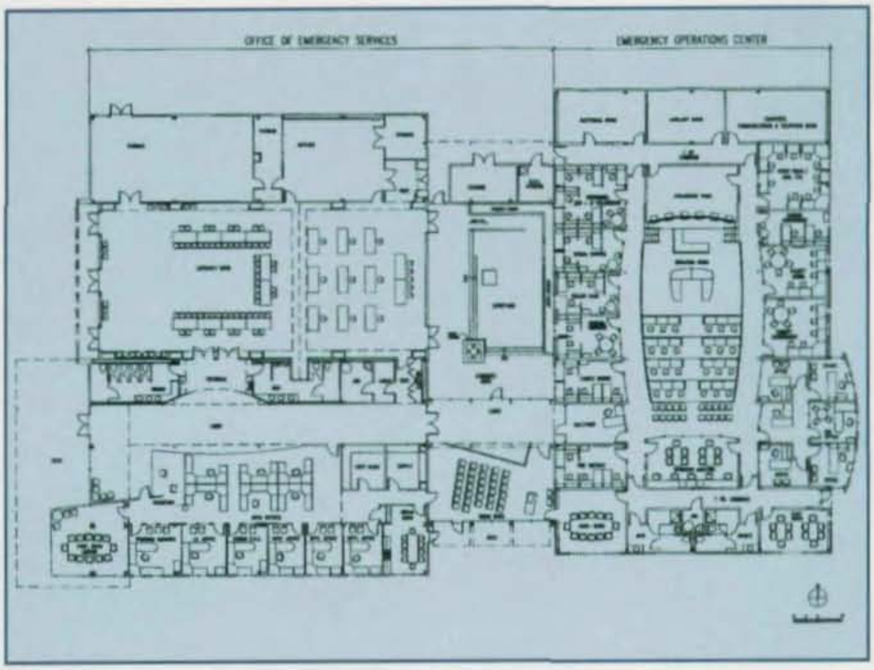
The design team, led by Michael Ross-Charles Drulis Architects, Sonoma, CA, with structural engineering provided by DASSE Design, Inc., San Francisco, selected a structural steel frame because of space planning flexibility, opportunities to feature exposed framing in the architecture, speed of erection and proven earthquake performance.

Building Functions

The county Sheriff is the Director of Emergency Services, the lead decision maker in the county's emergency management organization and provides direction and control of post-disaster operations. The facility has three primary functions:

- *The Office of Emergency* Services (OES) provides for daily operations with direct access to the EOC via restricted circulation. The OES is responsible for coordinating the emergency management program (preparedness, response, recovery and mitigation), coordinates the operation of the EOC, maintains liaison with other local and state government agencies, and directs deployment of the Sheriff's volunteer services, such as Search and Rescue.
- The EOC is a dedicated area for use as an incident command center during emergency operations. It consists of a Situation Room for data entry and retrieval and monitoring, encircled by focus rooms for specific operations and logistics, conference rooms, and a Situation Analysis Room. In the event of a disaster, communications are linked with decentralized emergency sites such as staging areas, hospitals, shelters and other governments agency command posts.
- The Sheriff's Multi-Purpose Room serves as the staging area for emergency workers, a disaster application center, a regional

A steel frame was selected for the Alameda County Office of Emergency Services / Emergency Operations Center because of space planning flexibility, opportunities to feature exposed framing in the architecture, speed of erection and proven earthquake performance



disaster response planning conference center and an assembly area for public functions.

The facility is designed for optimum flexibility and adaptability, both now and in the foreseeable future. It accommodates present and future space needs and the audio/visual and communication requirements of the OES and the EOC. The building is wired throughout for telephone, data, public address, radio and television to provide for internal communications and access to the Local Area Network (LAN) computer system, and links via phone, line radio and satellite to other facilities, field command posts and outside information services. The LAN system also utilizes a geographic information system for mapping and monitoring disasters and directing the response effort. The Situation Room has the ability to place data on any of three large projection screens and six monitors in a theater setting. An emergency power system provides uninterruptable power to all vital computer, security and communications systems. Electrical, mechanical and audio/visual equipment are seismically braced to the structure for the same seismic ground acceleration used in the building frame design, to ensure that critical building systems remain intact and operational for post-earthquake/disaster functions.

Building Description

The facility is a 25,600 SF, type II-NR single story steel framed structure. Column bays vary from 12 feet to 24 feet. The roof construction is a standard built-up roof over $1^1/_2$" x 18 gage metal deck supported by W12 beams and W16 girders. The long span roof over the multi-purpose room is framed with prefabricated steel open web joists and joist girders. General contractor was Nibbi Brothers of San Francisco. Steel joists were fabricated and erected by AISC-associate-member Vulcraft. The foundation consists of 24 inch diameter drilled piers and interconnecting grade beams. The building exterior cladding is a combination of GFRC (glass fiber reinforced concrete) panels, precast concrete panels and aluminum composite panels.

After considering the unique functions and distinct post-earthquake operability criteria of the OES and EOC spaces, the design team decided to structurally separate the OES and EOC. Adding the building separation provided the opportunity to employ a higher seismic performance criteria for the EOC frame and bracing of critical systems in a rational manner that was also economical . The EOC lateral force resisting system is a steel eccentric braced frame, selected for its combination of stiffness (drift control) and good ductility. Drift control was especially important to provide protection for the communications systems critical to post-earthquake operations. The braced frame members are W12 link beams, W8 columns and TS8x8 braces. In the OES structure, where maximum space planning flexibility was required, the lateral force resisting system is a steel moment frame. Moment frame girders are W18 sections and columns are W10 and W12 sections.

Frame analysis was accomplished with RISA-2D and the member calculations were done by hand.

Exposed Structural Steel

The structural steel frame is exposed at specific locations inside and outside the building as an integrated element of the architecture. The structure was exposed to emphasize its function in load bearing and seismic restraint, and as an armature for the architecture. The contract documents designated these members as "Architecturally Exposed Structural Steel" (AESS), as defined in section 10 of the *AISC Code of Standard Practice for Steel Buildings*, in order to apply the more rigid fabrication and erection tolerances. These tolerances were critical in the lobby, where wall finishes were detailed to recess just behind the WF column flanges. Exposed beam-column connections in the lobby are all-welded, single shear tab connections with exposed welds ground smooth. Erection holes in the beam webs and shear tabs were filled and ground smooth for a uniform appearance. Perhaps the most dramatic architectural use of AESS occurs over the

building entry, where W18 beams cantilever 20 ft. out from the building frame. The beams are fabricated with tapered ends and matching circular perforations. Intense coordination between the architect and structural engineer was required for the exposed steel members, connections, dimensions and finishes during production of the construction documents to achieve the desired architectural expressions while maintaining structural integrity and constructability.

Seismic Design Criteria

The building design was governed by the 1991 Uniform Building Code. Seismic design of essential service facilities, such as an Emergency Operations Center, using the code minimum provisions requires the use of an importance factor which increases the code minimum design base shear by 25% over standard buildings. During schematics, the owner and design team determined that in addition to the code minimum seismic requirements, the special building functions and proximity of the site to major earthquake faults merited a site specific seismic hazard study. Using site specific ground motions improves the reliability of predicting structural response and the designed level of performance. The use of unreduced design spectra allowed estimations of actual building displacements, strain levels, and discrete determination of yielding mechanisms, thereby providing valuable design information for both structural and nonstructural system components. The results of these additional analyses provided the basis for establishing performance guidelines for critical building contents. Site specific response spectra were developed by geotechnical consultants for ground motions with a 2% and 10% probability of being exceeded in 50 years. These are often referred to as the "maximum credible" and "maximum probable" earthquakes. Six known faults with characteristic earthquake magnitude greater

than 6.0 lie within an 11 mile radius of the site, the closest being $1\frac{1}{4}$ miles away. The response spectra also account for unknown potential sources of strong ground shaking emanating from concealed thrust system earthquakes, which is known as "background seismicity". (Background seismicity is considered a possible source of the high ground accelerations recorded in the Los Angeles area during the 1994 Northridge

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earthquake.) The site ground motion analysis yielded peak horizontal ground accelerations of 1.0g, two and one-half times greater than the UBC minimum seismic base shear coefficient computed for seismic zone 4.

The owner and design team decided on a seismic performance criteria for the eccentric braced frame of the EOC to respond elastically when subjected to a peak ground acceleration of 1.0g. The moment frame of the OES structure was designed to survive without collapse for 1.0g acceleration while permitting some excursion into the post-yield range. The LRFD provisions of AISC's Seismic Provisions for Structural Steel Buildings were used in the analysis and design of the moment frame and eccentric braced frame for the maximum load conditions. The intended yield mechanism of eccentric braced frames is to achieve shear yielding in the link beams, while the braces and columns resist the forces corresponding to the link shear yield limit state. An accurate assessment of the real yield strength of the link beams is critical to achieving this intended behavior. It is now common knowledge in the steel industry and structural engineering community that ASTM A36 steel has an average yield stress in the range of 45 ksi to 50 ksi. In order to confirm the link beam yield strengths, mill certificates for the link beams were submitted to DASSE early in construction for review and comparison with the material strength conservatively assumed in design.

Northridge Earthquake Repurcussions

As the general contractor was gearing up for construction, the early reports of poor steel moment frame performance in the 1994 Northridge earthquake became available. DASSE relied on field reports and interim recommendations from AISC, AWS and the Structural Engineers Association of California (SEAOC) to evaluate both the welding procedures specified on the project and the reliability of the welded beam-column moment connection. Welding quality control features, such as use of written welding procedures, use of proper electrodes, compliance with AWS D1.1 Structural Welding Code for joint fit-up, preheat and rate of weld metal deposition, and the removal of weld backing plates at the complete penetration weld of the beam flange to column flange were strictly enforced. This is in contrast with the pre-Northridge standard of practice in the industry. As a means of additional quality control, AWS qualification tests were administered on the site to welders performing complete penetration welds on the moment frame . All complete penetration (CP) welds were visually inspected and ultrasonically tested. Defective welds, which represented less than 10% of the total CP welds on the job, were backgouged and rewelded. After publication of AISC's *Northridge Technical Bulletin No. 2* in October 1994, DASSE contacted the county and recommended a review of the OES moment frame connections for the anticipated rotational demands at the beam-column joints. At that stage, construction had progressed to installation of interior partitions, mechanical, electrical and plumbing work, so a quick response and implementation strategy was necessary to mitigate unnecessary costs and disruption of any proposed connection strengthening. The beam-column joint rotation demands were studied and compared with the results of recently completed full scale tests at the University of Texas-Austin by Michael Englehardt. At beam-column locations where anticipated joint rotations exceeded the minimum rotations achieved in Englehardt's tests, beam flange cover plates were proposed in accordance with the AISC recommendations. With the county's agreement, 22 connections were strengthened, at a cost of approximately $1,100 per connection, including removal and replacement of interfering nonstructural features. The project was completed in about 15 months, three months ahead of schedule and within budget.

William A. Andrews, S.E., is a project engineer with DASSE Design, Inc., a consulting structural engineering firm based in San Francisco.

Simple Structure Yields Sophisticated Architecture

The designers of a new high-tech lab were able to cost-effectively develop a complex architectural solution by concentrating first on the building's basic design requirements

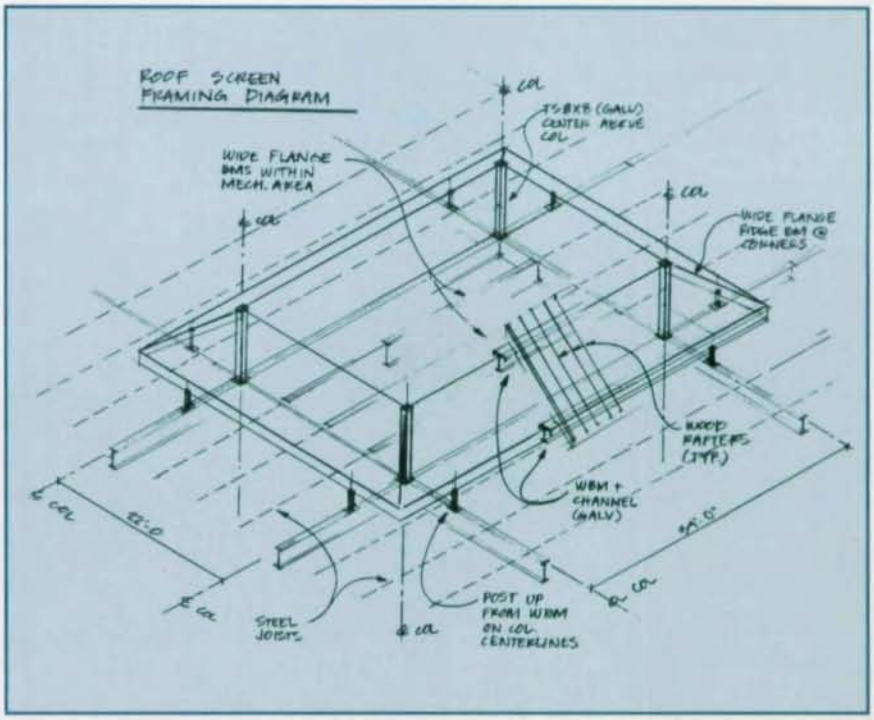
By Kevin A. Yoder

INTEGRATING A NEW, 60,000-SQUARE-FOOT NATIONAL TESTING LABORATORY FOR THE AMERICAN RED CROSS into a historic neighborhood in downtown Detroit, MI, required creative design solutions from architects and engineers at the Philadelphia-based architectural, engineering, interior design and planning firm of Ewing Cole Cherry Brott.

The design team was charged with designing the highly sophisticated facility under a conservative budget. In addition, because of its location in an historic section of the inner city, the building had to respect and respond to its surrounding environment while providing for sophisticated environmental and programmatic requirements. The American

Red Cross and the design team worked with community groups and local Red Cross officials to develop a design respectful of the historic area. The result is a prime example of how a simple, efficient structure forms the basis for high quality architectural character on the outside and high-tech capabilities on the inside.

The laboratory, part of the American Red Cross' program to consolidate its blood testing facilities nationwide, is based on a prototype designed by Ewing Cole Cherry Brott's research and development facilities planning subsidiary, Ewing Cole Brouwer. Ewing Cole Brouwer developed the prototype for this and eight other national blood testing laboratories throughout the country.



Architecturally Integrating The Facility

The two-story Detroit facility is located in the Brush Park Historic District on the site of the existing Southeast Michigan Red Cross facility. The steel-framed, masonry-clad building stands next to the Prismatic Club which exemplifies the architecture of the district.

The new building takes its visual cues from the Prismatic Club and other Victorian homes and buildings in the neighborhood. Decorative gable roof forms and pediments soften the building mass. Colored brick patterns, cast-stone trim, shingled pitched roofs, dormers, brick arches and traditionally scaled windows provide an historical presence.

Inside, the first floor features two separate main entrances — one for administrative and laboratory staff, the other for blood samples. The remainder of the floor houses a state-of-the-art, highly efficient blood testing and processing facility where all blood testing operations are performed. This laboratory is capable of testing more than 1.2 million samples a year.

Because of frequent high-tech equipment advances and

While high-tech inside, the exterior of the new Red Cross facility reflects architecture of the neighboring historic district.

changes in regulations, the flexibility of casework, mechanical and plumbing systems was of prime concern. Lab modules were designed to allow all bench-top equipment to be easily reconfigured. All lab bays were constructed with the same casework components and are completely interchangeable, allowing for change by simply adding or subtracting components from the laboratory furniture system.

Utilities are in place at each module to accommodate a wide variety of testing equipment.

The second floor houses administrative offices, classrooms, computer rooms, locker rooms, a large break room and other support areas — as well as offices for other regional and national American National Red Cross divisions including Regional Blood Information Systems, quality assurance and the senior principal officer.

The basement accommodates mechanical systems, storage areas and expansion space.

Early Planning

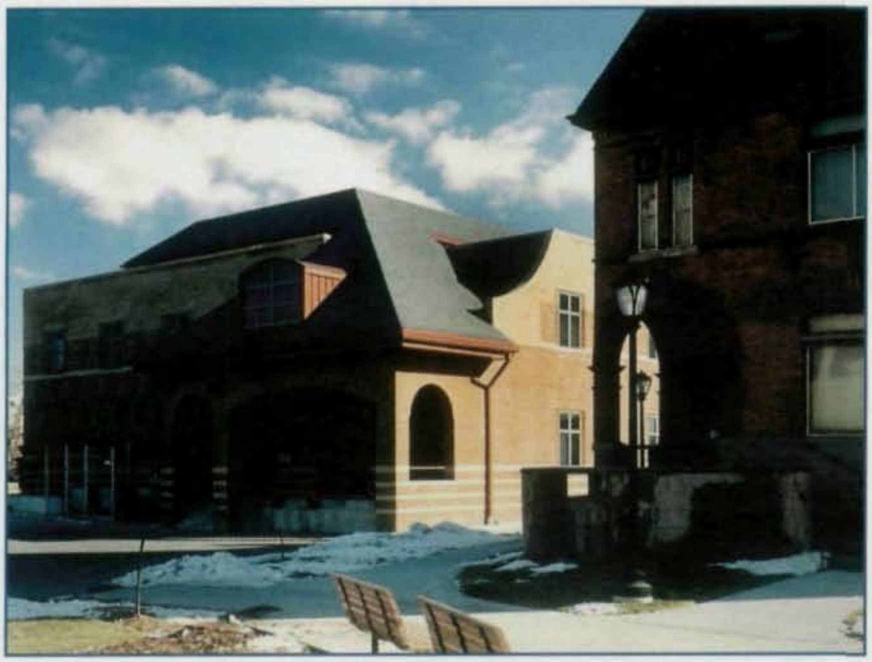
At the onset of the project, it was decided that the building had to be a simple rectangular shape to accommodate budgetary restrictions and the fast-track construction method being employed to meet the project schedule. An early structural bid package was completed prior to the final design, and construction began during the termination of the design phase. The construction management firm, Detroit-based George W. Auch Company, joined the team during the planning phase to facilitate development of the most efficient design solution. Professionals from the construction management company attended numerous design development meetings to help establish the design criteria and building systems at the onset to assure cost requirements could be met.

To meet the project schedule and budget, the majority of the structure had to be quickly and efficiently designed in the early phase of design. Structural steel was the logical choice for numerous reasons. Typical bays of steel were expediently evaluated to determine the most efficient bay size for the standard lab module. Steel would allow for speedy erection and easy modification if required during the fast-track design. A braced-frame system was chosen for its efficiency as well as its cost and time-saving contribution in both the design and construction phases. Lastly, it was decided that the placement of rooftop equipment would be limited to specific bays on the roof that would be designed further along in the design process.

It was also necessary to establish the exterior building envelope and its support structure early on. Brick was the preferred exterior finish in keeping with the vocabulary of other neighborhood buildings. "Punched" window openings were chosen for their residential scale. Brick with concrete masonry backup was desirable for its durability. Also, the use of standard precast masonry and loose steel lintels within the exterior masonry eliminated the need for the continuous hanging steel support system required for a strip window system. The typical window opening was based on a maximum practical lintel size. The brick shelf (relieving) angle was located within the depth of the spandrel beams to minimize the amount of additional steel support required.

Defining the Structural Systems

The structure's foundation system consists of concrete spread footings for the reinforced concrete basement walls and steel columns. The mechanical room is located in the basement so the weight of the equipment can be supported on the basement slab on grade. Large areaways with louvered wall panels were created in the basement walls to accommodate the



air intake and relief of the air handling units located there. This also affords convenient access in the future when equipment is added or removed.

Columns are W10 shapes. W18 and W24 beams and W24 girders span 22-ft. on the first floor and second floor framing consists of W16 and W21 beams.

A steel joist floor structure was considered but eliminated because of the vibration-sensitive laboratory and computer equipment used in the facility. At the second-floor level, $2^1/_2$-in.-lightweight concrete slab on a 2-in. galvanized composite metal deck from Bowman Metal Deck was employed. This system efficiently accommodates the office functions of this floor. The first-floor slab consists of a $4^1/_2$-in. normal weight concrete slab on a 2-in. metal deck. This thicker, heavier slab was chosen because of the laboratory functions located on this floor. The increased mass of the slab serves to dampen any floor vibrations that may affect the sensitive laboratory equipment.

A column spacing of 22 feet was utilized in the longitudinal direction of the building; this was derived from the width of the lab module. Across the short direction of the building, the three bays of 26-34-26 feet were coordinated with the lab module and casework.

At the roof level, K-series steel joists provided by AISC-associate member Canam Steel Corporation were selected for the majority of the framing members because of their economy. All beams and girders on the column lines are structural steel wide flange members to ensure the rigidity of the steel frame. Rooftop equipment was concentrated into designated bays at each end of the building. Steel wide flange beams in lieu of steel joists were required to support the loads of the air-cooled chillers. These bays received roof screens to hide the equipment; these screens were designed later in the process.

The locations of the braced frames within the structure required early coordination of the space planning and structural requirements. In the short direction of the building, one brace frame was located at each end adjacent to the stairs. To accommodate exit doors in the stair towers, a V-braced frame allowing for a door opening was used. In the other direction, one X-braced frame was located near the center of each longitudinal elevation.

Horizontal wind and seismic forces are transferred through the floor and roof diaphragms at each level to the braced frame. The bracing members consist of structural steel tube shapes connected to the wide flange beams and columns with steel gusset plates. The size of the bracing members was limited to TS4 and TS6 sections so they could be easily accommodated within the walls. Analysis of the lateral loads on the structure was performed utilizing the frame modeling capabilities of STAAD3 from Research Engineers, Inc.

Refinement of the Building Massing

After the design and documentation of the majority of the structure were well underway, the design team began to manipulate the rectangular mass of the building. To provide a residential scale appropriate for the neighborhood, several areas received particular design attention. These included the building entrance area, the southeast corner of the building at the street intersection, the mechanical roof screens and the loading dock.

To articulate the corner of the building at the street intersection, the architect chose to subtract from—rather than add to—the primary building mass. A residential scale was created by carving out the upper corner of the building and introducing a sloped shingled roof form similar to that seen on some of the near-by historic structures.

The primary structural refinement of the corner involved eliminating the corner bay of steel framing at the roof level. A sloping hipped roof was then introduced between the edges of the remaining structure. A W24 girder forms the ridge member that frames from the one-story corner column to the post atop the building column at the main roof. Roof rafters are supported on horizontally spanning steel beams at the second floor level, main roof level and the mechanical roof screen structure above.

Wood rafters were originally conceived to frame between the horizontal supports. However, galvanized metal studs were substituted for their economy, constructability and non-combustible characteristics.

Mechanical Equipment Integration

The sloping roof form was also adapted to the mechanical equipment screens. These screens were integrated with the hipped roof at the corner to form a continuous sloping roof plane. The design challenge was to efficiently design the sloped screen support structure without introducing additional beams below the roof. This was particularly important because the roof framing members around the designated mechanical areas consisted of steel joists which would not take the concentrated point loads from the roof screen posts. Additionally, it was not necessary for the roof screens to continue all the way down to the roof surface. By keeping the screens up above the roof surface, the roof could run uninterrupted under the screens. This eliminated the need for additional roof drainage and crickets at the mechanical areas.

To implement this plan, the primary screen posts—TS8x8 structural steel tubes —were first located over the building columns for direct load transfer to the columns. A continuous horizontal steel beam and channel framed between the posts to provide the upper support for the vertical sloping roof rafters.

Next, two secondary screen posts—TS4x4 shapes—were located on the wide flange steel roof members on the column centerlines adjacent to each primary post. Another horizontal beam and channel then cantilevers over the secondary posts to meet at the lower ridge point. Wide flange corner ridge members connect the end of the cantilevered horizontal beams to the primary screen posts at the ridge peak. In this manner, an efficient screen support system was created which had limited impact on the primary roof structure already designed.

The design of the loading dock and receiving area was particularly important because of its prominent location along the major pedestrian walkway to the existing American Red Cross facility and directly adjacent to the Prismatic Club. A single story of framing was added onto the main volume of the structure to provide a covered area for loading operations. A sloping roof form similar to the opposite corner of the building continued from the mechanical screen on the roof down to the one-story eave line at the loading dock. Galvanized wide flange beams again form the ridge members that frame between columns to support the roof rafters. An arched dormer window characteristic of the neighboring buildings was also located on the roof at this location, providing natural light to the office areas inside.

In keeping with the existing vocabulary of the neighborhood, arched brick masonry openings were utilized at the receiving area. The large flattened arch over the truck loading area was accomplished by incorporating a reinforced concrete beam within the brick masonry. This beam spans between the steel wide flange columns at each side of the beam. These columns are also encased in concrete for bracing and resisting the thrust of the flattened arch.

The main entrance of the facility was also designed as an addition to the main rectangular volume of the structure. Structural steel framing similar to the rest of the building was used. An exterior roof terrace off the second-floor break room articulates the mass of the building at the entrance. Half-round brick masonry arches are also used here. The inscribed cast-stone panel above the entrance is supported by a structural steel channel suspended from the wide flange spandrel beam at the second floor.

Despite many constraints, the end result is a building that architecturally responds to the surrounding neighborhood and functions efficiently as a state-of-the-art blood testing laboratory. By concentrating first on the basic design requirements of the building—and assuring those criteria could be accommodated within the designated budget—the design team created a building that is both architecturally sophisticated and cost-effective.

Kevin A. Yoder is a structural designer at Ewing Cole Cherry Brott. From its headquarters in Philadelphia, PA and offices in Washington, DC, and Haddonfield, NJ, Ewing Cole Cherry Brott offers specialized services to research and development, health care, academic, public sector, sports and entertainment clients nationwide.

Designing Steel-Framed Cut-And-Cover Tunnels

While more commonly framed with concrete, steel is often more economical for long, wide tunnels

Pictured above is the construction of the Baltimore Shot Tower Station showing excavation, cross lot bracing and the SPTC wall. Pictured opposite is work on the Central Artery tunnel in Boston.

By Brian Brenner, P.E., and David Bacon, P.E.

CUT-AND-COVER TUNNELS USUALLY ARE CONSTRUCTED OF REINFORCED CONCRETE WALLS AND SLABS. However, when design or environmental considerations require wide, deep tunnels, the concrete slabs and walls may become prohibitively thick. Wide tunnels with box spans of 80-ft. or more, and deep tunnels with backfill exceeding 20-ft., can require very thick (eight to nine ft. or thicker) reinforced concrete slabs. These slabs are difficult to construct and can make the tunnel prohibitively expensive to build by forcing its vertical profile down to provide the required vertical clearance within the tunnel itself.

One option to reduce horizontal and vertical structural dimensions is to frame the tunnel in steel. Using a combination of structural steel and reinforced concrete, roof slabs and wall members can be designed to take advantage of the inherent strength of composite construction.

Typical Tunnel Section

For steel-framed cut-and-cover tunnels, excavation support walls can be temporary or included as part of the final tunnel structure. When temporary cofferdam walls are used, a reinforced concrete base slab is cast, and then rigid steel frames are constructed within the excavation, with a pinned connection at the base. Cast-in-place concrete walls and roof slab are then made composite with the rigid frame to form a watertight tunnel structure.

Although steel roof beams can be attached to a reinforced concrete wall when cofferdam walls are to be included as part of the final tunnel structure, a more efficient utilization of the strength of structural steel can be achieved by using Soldier Pile Tremie Concrete (SPTC) walls. For this type of cofferdam wall, wide flange soldier piles are inserted into a slurry trench, then the space between the soldier piles is filled with tremie concrete. The steel soldier piles form the vertical framing element of the rigid frame, which consists of the soldier piles, the composite roof beams and the reinforced concrete base slab.

Another construction method utilizing structural steel soldier piles is to build the cofferdam/final structure wall from tangent (or secant) piles. Steel sections are inserted into pre-augured holes (typically drilled without steel sleeves that are held open with bentonite slurry and backfilled with tremie concrete).

The tunnel roof is constructed of steel roof beams made composite with a concrete deck. The base slab is constructed of reinforced concrete. Roof beam spacing matches the soldier pile spacing (or the steel column spacing) in a cast-in-place concrete wall. Due to the span lengths required in wide tunnels, roof beams are typically built-up girders. The concrete roof deck is usually six-in. to 12-in. thick and the design is similar to that prepared for a composite bridge deck. However, the tunnel section tends to have a higher percentage of dead load than a bridge deck, since soil backfill is treated as a superimposed dead load. Another difference is the much greater axial compression load. The tunnel roof acts not only in bending, but as a strut between exterior walls of the box. Conservative design practice recommends that all of the axial load be carried by the steel beam. This leads to easier connection details and staging requirements, since the concrete roof does not need to be cast and brought up to design strength before inward loads from the cofferdam walls are transferred to the members.

Because headroom clearance on the Central Artery tunnel project in Boston was only 20-ft., the 100-ft. long soldier piles needed a bolted splice detail utilizing A490 bolts.

The design force used to size the tension flange of the built-up girder should not be reduced by the full calculated axial compressive load from strut action. This conservative practice takes into account variations in earth pressure during construction staging and potential reductions in the loads applied to the cofferdam walls. For example, at dewatered excavations, the pore water pressure may not return fully to its pre-construction level for many years, thus reducing the axial compression force applied to the roof beams.

Details

Special concerns when dealing with steel-framed cut-and-cover tunnels include: field tolerances for cofferdam walls; corrosion protection; waterproofing; details for steel-to-steel connections (and for steel-to-concrete connections); and field splicing of soldier piles for SPTC walls.

When SPTC walls are used as part of the final tunnel structure, soldier pile installation tolerance is an important concern. Soldier piles cannot be installed exactly at the location specified on the construction drawings since the piles will tilt out of plumb in both axes, rotate, and deviate from horizontal tie. Therefore, it's important to consider all of these tolerances in order to correctly fabricate members and connection details. For example, if steel roof beams are to be field-connected to the soldier piles, the exact dimension for each beam cannot be determined until a detailed field survey has been conducted at all locations.

Waterproofing for steel frames constructed within temporary excavation support walls is relatively straightforward. Waterproofing materials can be installed on the side walls, below the base slab and on the roof deck prior to backfilling.

For the case of SPTC walls included in the final structure, it is not possible to provide blind side waterproofing around the tunnel box. Instead, waterproofing is installed over the roof deck and up the side of the SPTC wall. The inherent water tightness of the SPTC wall itself is the only primary protection against water intrusion through the tunnel walls. For this design, waterproofing of the connection between the roof girder and the soldier pile is critical. The connection is usually designed to allow rotation to occur and the rotation must be accommodated by movement within the waterproofing detail.

The corrosion protection of non-waterproofed SPTC walls is an important consideration that requires careful investigation of specific site conditions around the tunnel. Issues such as the fluctuation of groundwater levels, pH of the soil, stray currents in the groundwater surrounding the tunnel, and the amount of electrical work within the tunnel itself all contribute to the decision making process required to determine the corrosion protection of the SPTC wall.

Example 1: Washington Mall Tunnel

The Washington Mall tunnel (Washington, DC), the Baltimore Metro Shot Tower Station, and the Central Artery tunnel (Boston) are all examples of cut-and-cover tunnels with steel framing.

The Washington Mall tunnel was constructed in the late 1960s. The cut-and-cover highway tunnel provides a path for a six-lane expressway beneath the Mall in the nation's capitol. The tunnel features a concrete base slab, approximately five- to eight-ft. thick, with steel girders and columns spaced at four- to six-ft. on center. The roof girders are built-up sections 61-in. deep, spanning approximately 100-ft.

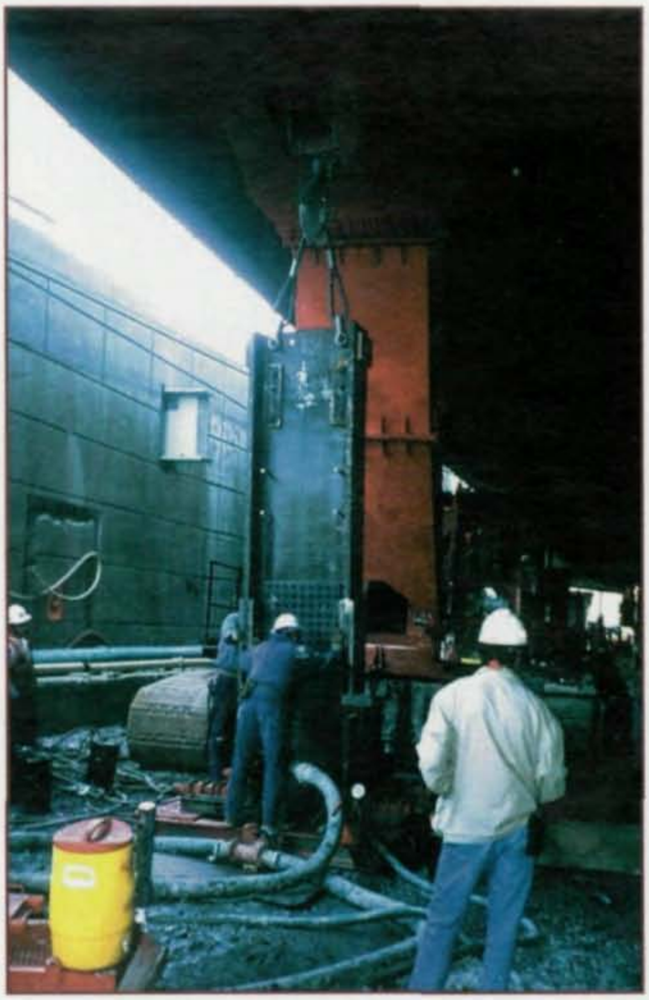
Some girders span more than 115-ft. between exterior walls. The roof girders are supported by intermediate columns that are used to help frame air ducts on both sides of the tunnel. At some sections, the girders are continuous across the intermediate columns, framing into a moment connection with exterior wall columns. At other sections, the girders are simply supported by the intermediate columns, and separate steel W sections form the roof of the side air ducts. The girders support a six-in. concrete roof slab.

Girders are supported by built-up steel columns. The columns are embedded in cast-in-place concrete walls. When the roof girders run continuous across intermediate columns, the exterior columns are detailed in a fixed connection to the concrete base slab. The detail includes high-strength anchor bolts connected to a bearing plate at the bottom of the concrete slab.

Example 2: Boston's Central Artery

The depressed Central Artery in downtown Boston will replace a six-lane viaduct with an eight- to 10-lane tunnel. Construction activity began last year on the 6,000-ft.-long tunnel, which is located beneath the original structure. The structure features SPTC walls, steel/concrete composite roof beams, and a reinforced concrete base slab. The SPTC walls are used as part of the final tunnel structure. Soldier piles are spaced at four- to six-ft., with wider spacing in some locations. The piles are heavy W36 sections made of high strength steel. Steel/concrete composite roof beams are designed with a rotationally free connection to the piles. The connection is erected by welding a seating angle or folded plate to the pile, placing the girder, and then fastening the girder web to the pile with angles. The roof beams are built-up, high-strength steel girder sections, four- to eight-ft. deep, with a 12-in.-thick concrete roof slab.

The reinforced concrete base slab varies from four- to 12-ft. thick. The base slab has a rigid, moment resisting connection to the soldier piles and the transfer of forces from the base slab, which is continuous, to the discreet pile elements is accomplished by a distribution beam that is made integral with the slab.

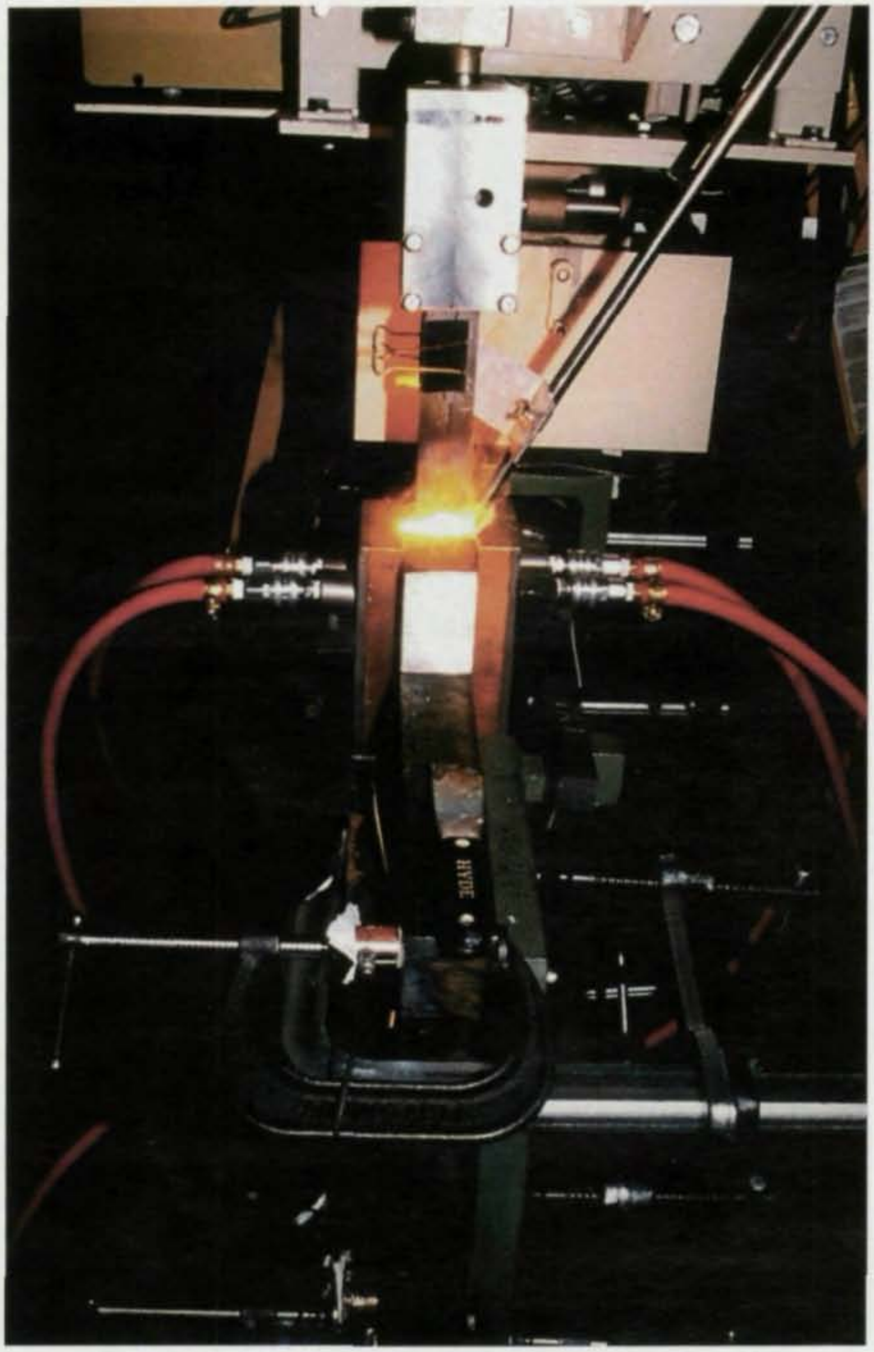


One of the difficulties with the Central Artery Project is the low head room beneath the existing Artery viaduct. Pictured is a soldier pile with bolted splices.

The requirement of keeping the existing roadway in service during the tunnel construction imposed some unusual challenges. For the SPTC walls, 100-ft.-long soldier piles will need to be installed in locations with the headroom clearance is only 20-ft. A bolted splice detail was designed with splice plate lengths that are minimized through the use of ASTM A490 bolts (bearing, with threads excluded from the shear plane). This was necessary because the available splice locations were restricted by base slab moment connection details on both sides of soldier piles. The available space was further complicated by independent gradients for the northbound and southbound tunnels, which were separated by a center SPTC wall. A bolted splice was selected because the splice must be completed quickly during the slurry trench operation to prevent a cave-in of the slurry trench walls.

The SPTC walls were designed to support underpinning loads from the existing elevated expressway. The underpinning consisted of a series of grade beams and needle beams straddling the steel column bents and concrete footings of the viaduct. The grade beams span from slurry wall-to-slurry wall.

Brian Brenner, P.E., is a senior professional associate and David Bacon, P.E., is a project engineer, both with PBQD and both working for Bechtel/Parsons Brinckerhoff.



A new FHWA program is helping engineers and fabricators learn about this advanced welding technology

Electroslag Welding For Bridges

By Krishna K. Verma

ELECTROSLAG WELDING (ESW) IS A PROCESS THAT JOINS METALS WITH HEAT GENERATED BY PASSAGE OF ELECTRIC CURRENT THROUGH MOLTEN CONDUCTIVE FLUX, which melts the filler and base metals. Now in its final stage, a comprehensive Federal Highway Administration (FHWA) R&D effort (Demonstration Project DP-102) is designed to transfer the new technology to state DOTs and bridge fabricators.

Due to its high deposition rate, ESW is considered the most productive of any welding process in joining thick components. Initially, when the process was introduced in the U.S. in the late 1960s, there was some obvious success. However, certain welding problems began to surface in terms of welding imperfections and inadequate properties, which led, in 1977, to the FHWA placing a moratorium (FHWA Notice 5040.23) on the use of EWS for weldments on primary structural tension bridge members. The notice effectively eliminated the use of ESW not only in bridge fabrication but from various other U.S. industries as well.

In the 1980s, the FHWA launched a comprehensive

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research and development in the area of ESW technology. The main objectives of the program were to better understand the specifics of the ESW process and to advance the level of applied ESW technology. ESW operating procedures and weld properties were significantly improved as a result of the initial phase of this research program, which was conducted by the Pacific Northwest Laboratory (PNL), Richland, WA, and the Oregon Graduate Institute of Science and Technology (OGI), Beaverton, OR. Foremost, the Narrow-Gap Improved (NGI) ESW technique, an improved modification of ESW process, was developed.

The NGI technique produces satisfactory welds and considerably improves Charpy V-notch (CVN) toughness. In the early 1990s, extensive field trials were conducted based on the research findings. These trials were conducted at four bridge fabrication facilities and showed satisfactory results.

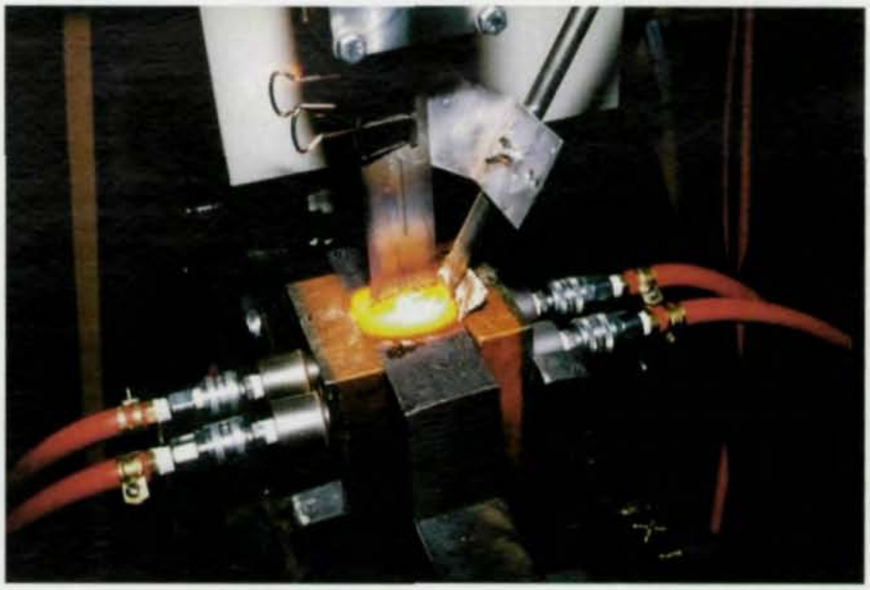
In September 1993, FHWA awarded a four-year contract to BIRL, Northwestern University's industrial research laboratory based in Evanston, IL. A team of researchers, led by Northwestern's Valdemar Malin, Ph.D., and assisted by an advisory board of federal and state transportation officials and bridge and welding engineers from private industry. FHWA's technical representative and program manager for this project is Krishna K. Verma, Welding Engineer of the Bridge Division of FHWA's Washington, DC, office.

Program Objectives

The objectives of Demonstration Project DP-102, "Electroslag Welding for Bridges," include:

- **ESW Equipment and Materials Availability.** An important task for acceptance of ESW is to ensure that the required equipment and materials be commercially available to users for making production welds.
- **Development of ESW Documentation**. A comprehensive package of specifications and guides required by the FHWA is being developed by BIRL and is close to completion. The guides contain technical, procedural, and training information on ESW. Proposed Specifications (Code requirements) are oriented toward inclusion into ANSI/AASHTO/AWS D1.5 Bridge Welding Code.
- **Verification of NGI ESW Procedure.** The equipment, the new electrode wire, the restored flux formulation, and the recommended ESW procedure have to be tested. This is crucial for the implementation of the new technology. The laboratory trials conducted at BIRL's welding laboratory have proven that acceptable electroslag welds can be produced on a regular basis.
- **Verification of Toughness in Electroslag Welds.** Testing during the demonstrations will prove that acceptable toughness of Electroslag welds can be produced on a regular basis.
- **Transfer of New ESW Technology.** To methodically transfer the new ESW technology to state DOTs, bridge fabricators and FHWA representatives, relevant information on the NGI ESW technique will be disseminated and actual ESW making would be demonstrated. ESW demonstrations will start in September 1995 and will be presented in various locations in the U.S. During the demonstration phase, a plan is in place to develop video training materials on NGI/ESW for future use by state agencies, fabricators and others.

For information on the demonstration projects contact the Federal Highway Division office in your individual state or call Krishna Verma at 202/366-4601.



Pictured opposite: The slag pool is approaching the top of the run-off tabs at the upper end of the electroslag weld.

Pictured above: The slag pool is at the top of the run-off tabs immediately after the power has been turned off.

Krishna K. Verma is a Welding Engineer for the Bridge Division in FHWA's Washington, DC, office.

FILTRATION EQUIPMENT

A new gravity flow type media filter, THE ELIMINATOR, is designed to filter metal fines from metalworking fluids. Simple in design, heavily built, easily installed and flexible, the filtration system is designed to be highly efficient, low cost and long lasting. It requires only air to operate, and handles all types of grinding dirt and machining chips. Plus, it has no rubber seals to wear out. The system can filter flow rates from 5 to 160 GPM.

For information, contact: Clarmatic Industries, 504 East Front Street, P.O. Box 780, Pemberville, OH 43450; 800/521-5246; fax: 419/287-3836 or **CIRCLE # 21**

ACCESS AND CONTAINMENT SYSTEMS

Using five patented systems in combination with custom-engineered components, Beeche System Corp. has designed safe and cost-effective access and containment systems that have traversed even the most challenging of buildings and bridges. All of the systems meet or exceed local, state and federal (OSHA) standards. The systems include:

• Containment Systems can be easily modified to accommodate all levels of lead paint containment, including SSPC "Class 1 Containment";

• Building Construction and Maintenance Systems, which significantly reduce the amount of labor required to successfully and cost-effectively complete many types of building construction or maintenance projects; and

• Bridge Access Systems, feature a modular aluminum truss frame to provide safe and efficient work platforms.

For more information, contact: Beeche Systems Corp., Building 202, Scotia-Glenville Industrial Park, P.O. Box 2024, Scotia, NY 12302; 518/381-6001; fax: 518/381-4613 or **CIRCLE # 22**

GANTRY CRANE

The MI-JACK TRAVELIFT is a self-propelled rubber-tired gantry crane able to pick and carry its capacity safely with capacities ranging from 15 to 325 tons. Dimensions are customized to the customer's application. The crane can straddle incoming and outgoing trucks and railroad cars to speed loading and unloading operations. A variety of material attachments, such as slings, magnets, hooks and clamps, can be utilized to solve more refined storage and transporting problems.

For more information, contact: Mi-Jack Products, 3111 W. 167th St., Hazel Crest, IL 60429; 708/596-5200; fax: 708/225-2312 or **CIRCLE # 115**

SAWS

A new brochure from Kaltenbach describes the company's line of semi-automatic heavy-duty bandsaws, semi-automatic heavy duty bandsaws for miter cutting, and fully-automatic heavy duty bandsaws. In addition, the literature describes the development of sawing centers tailored to individual customer's applications. Several applications are shown, including a walking beam cross loader with multiple stations; a cutting center with walking beam cross loader, bandsaw with miter base, CNC length measuring and ejection system; and a cutting center for bundled material in hexagonal, square, or rectangular configuration.

For more information, contact: Kaltenbach, Inc., 6775 Inwood Dr., P.O. Box 1629, Columbus, IN 47202; 812/342-4471; fax: 812/342-2336 or **CIRCLE # 29**

TELESCOPING BOOM

The PRO-126 is a new telescoping boom with a 6-ft. articulating jib boom and a 132-ft. working height. The boom features a hydraulically operated self-stowing platform that slides into position so the unit will fit on a conventional low-boy style trailer. The stowed length of the boom is 42-ft.-9-in. The unit's work platform is 92-in. wide with 500 lbs. capacity and can rotate continuously from 90 degrees left to 90 degrees right.

For more information, contact: Don Roach, Snorkel-Economy, Box 4065, St. Joseph, MO 64504-0065; 816/364-0317; fax: 816/364-0380 or **CIRCLE # 62**

DRILLING UNITS

A new brochure from Behringer Saws describes a wide variety of drilling equipment, including drills, punches, shears and CNC controls. The most extensive unit featured is the FP 1200, a fully CNC controlled drilling unit allowing fast working of flat bars, angles and structural sections.

For more information, contact: Behringer Saws, Inc., Morgantown, PA 19543; 610/286-9777; fax: 610/286-9699 or **CIRCLE # 69**

PUNCHES & DIES

The Cleveland Steel Tool Co. has announced that punches and dies for Hougen-Ogura hole punching machines, models 75001 through 75006, are now available. The tooling is available in round, oblong, square, rectangular and hexagonal shapes and can be shipped within 48 hours.

For more information, contact: The Cleveland Steel Tool Co., 475 East 105th St., Cleveland, OH 44108; 800/446-4402; fax: 216/681-7009 or **CIRCLE # 67**

BENDING STEEL

Contemporary architecture and design is once again calling on curved structural steel for its grace, beauty and design flexibility and NAPTech's 320,000-sq.-ft. facility is designed to meet very tight delivery times. Induction bending can be used for heavy angles, channels, wide flanges and tubes.

For more information, contact: NAPTech, Inc., 851 South Freeport Industrial Pkwy., Clearfield, UT 84015; 801/773-7300; fax: 801/773-6185 or **CIRCLE # 116**

STEEL PROCESSING

Arkansas Steel Processing can supply a fully processed product, including cutting, drilling, coping and bolted clip angles. The company specializes in floor, framing and perimeter beams, girders and columns and can either supply material or process a fabricator's material.In addition, ASP can supply WT's split from wide flange beams, can supply material cambered to a customer's specification, and can supply beam and column material cut or milled to size.

For more information, contact: Arkansas Steel Processing, Inc., P.O. Box 129, Armorel, AR 72310; 501/762-1000; fax: 501/762-1411 or **CIRCLE # 125**

ROTATING & POSITIONING EQUIPMENT

A new brochure from Romar describes the manufacturer's line of automated weld positioning equipment. Included are: conventional rotators; special rolls; automatic tank welders; pile racks; positioner/turntables; beam lines; and manipulator columns & booms.

For more information, contact:Romar Elite Automation, Inc., P.O. Box 3417, Tustin, CA 93681; 714/569-1050; fax: 714/569-1009 or **CIRCLE # 119**

PUNCHES

A wide range of punches and dies are described in a new brochure from Hougen. The punches range from the Punch-Pro 75001, a small, light portable punch suitable for light structural applications, to the Punch-Pro 75006, which can punch flat, channel, H-steel, and angle with holes up to $^{15}/_{16}$ in. through $^{1}/_{2}$-in. stock.

For more information, contact: Hougen, G-5072 Corunna Road, Flint, MI 48504; 313/732-5840; fax: 313/732-3553 or **CIRCLE # 99**

DRILL PRESSES

Jancy Engineering has introduced a new line of (beltless) gear-drive multiple speed drill presses. The heavy duty design includes solid cast iron bases and drilling tables. A rigid $4^{3}/_{8}$-in. diameter column adds strength and stability. All units ship standard with three Morse taper spindle, quick change speed controls, precision drilling depth gauge for accuracy and repeatability.

Also available from Jancy is the BM-20, a new generation portable bevel machine designed for beveling steel plates and pipe. Each machine is fitted with two milling heads holding six indexable cermet inserts allowing smooth and efficient operation while beveling.

For more information, contact: Jancy Engineering Co, 2735 Hichory Grove Road, Davenport, IA 52804; 319/391-1300; fax: 319/391-2323 or **CIRCLE # 72**

FABRICATION EQUIPMENT

A new catalog from Comeq, Inc. describes the company's line of fabrication equipment, including: Roundo beam benders, angle bending rolls, plate bending rolls and welding positioners; Adira upstroking and downstroking press brakes; Americor hydraulic and mechanical shears; and Geka single and dual cylinder hydraulic ironworkers and punches.

For more information, contact: Comeq, Inc., P.O. Box 2193, Baltimore, MD 21203; 410/325-7900; fax: 410/325-1025 or **CIRCLE # 68**

HAND-HELD GRINDERS

Chicago Pneumatic Electric Tools has introduced the most powerful 5-in. angle grinder in its class. With an 11 amp, 1,200 watt rating, the new WS1200 weighs only 4.2 lbs., for a superior power-to-weight ratio. Also new are two- and three-horsepower vertical grinders featuring a vibration isolation system.

For more information, contact: Chicago Pneumatic Tool Co., 2200 Bleecker St., Utica, NY 13501; 315/792-2600; fax: 315/792-2651 or **CIRCLE # 103**

ANGLE & FLAT BAR PROCESSING

A new product sheet from Controlled Automation describes the ABL-100-2 angle and flat bar processing line. The product offers both single and double cut 215 ton shears, double die holders, automated park marking, 115 ton punch, and automated cut-off torch for fast and accurate flat bar sizing.

For more information, contact: Controlled Automation, Inc., P.O. Box 530, Bryant, AR 72089; 501/557-5109; fax: 501/557-5618 or **CIRCLE # 104**

BENDING MACHINES

A new brochure from Advanced Fabricating Machinery describes two bending machines for hollow structural sections. The QB-76 has a bending capacity of up to $2\frac{1}{2}$-in. round sections while the QB-50 has a capacity up to $1\frac{1}{2}$-in round sections.

For more information, contact: Advanced Fabricating Machinery, #65 Route 125, Kingston, NH 03842; 603/382-1476; fax: 603/642-4813 or **CIRCLE # 105**

STEEL BENDING

Chicago Metal Rolling has expanded its capabilities to include bending or rolling all sizes of pipe and tubing from $\frac{3}{8}$-in. to 14-in., angle iron from $\frac{1}{2}$-in. x $\frac{1}{2}$-in. x $\frac{1}{8}$-in. to 10-in. x 10-in. x 1-in., square bar from 1/4-in. to 8-in., and square tubing from $\frac{3}{8}$-in. to 14-in.

For more information, contact: Chicago Metal Rolled Products, 3715 South Rockwell, Chicago, IL 60632; 312/523-5757; fax: 312/650-1439 or **CIRCLE # 66**

SAWING FLUID

Lenox Band-Ade Sawing Fluid is a new, high-performance, biodegradable sawing fluid. Formulated to provide a direct productivity increase, the fluid offers superior releasing agents to clear chips and shavings from the gullet after the teeth pass through the material being cut.

For more information, contact: American Saw & Manufacturing Co., 301 Chestnut St., P.O. Box 504, East Longmeadow, MA 01028-0504; 800/628-3030; fax: 413/525-3961 or **CIRCLE # 113**

PLATE ROLLS

New initial-pinch plate rolls are available in lengths of 6- 8- and 10-ft. and capacities from $\frac{1}{8}$-in. to $\frac{3}{8}$-in. The machines feature all steel frame, electromechanical drive, power rear roll adjustment, movable control pedestal and arrangement for options such as cone and angle bending attachments.

For more information, contact: Cole-Tuve, Inc., Box 2156, Baltimore, MD 21203-2156; 410/485-7777; fax: 410/485-7778 or **CIRCLE # 126**

ROLL FORMING

The patented ASC Break & Hump Roll Forming System features: enhanced CNC software; batch uploading or downloading by a supervisory computer; squeegy-type sheet lubrication; one-man operation; reduced floor space; reduced scrap; two-post shear configuration; four-post shear configuration for punching or notching; AC servo feed roll assembly; dual glide systems; and quick disconnect drive option. Also, no special press foundations are required.

For more information, contact: ASC Machine Tools, Inc., N. 900 Fancher Road, P.O. Box 11619, Spokane, WA 99212-1619; 509/534-6600; fax: 509/536-7658 or **CIRCLE # 128**

DUAL COLUMN SAWS

American-made, dual column cut-off saws for faster cutting with closer tolerances are described in a new catalog from DoALL Co. All the saws feature hardened, ground and chrome plated columns that support the saw head and have a hydraulic feed cylinder between the columns for balanced feed control. The design provides stability and reduced vibration for exceptional accuracy.

For more information, contact: Dave Snyder, DoALL Co., 254

North Laurel Ave., Des Plaines, IL 60016; 612/890-3300; fax: 612/895-6450 or **CIRCLE # 129**

HEAVY MOVING KITS

Safe and economical heavy equipment moving on delicate or standard concrete floors is possible with Hilman Rollers' Deluxe Moving Kits. The 11 kits range in capacity from 2 to 60 tons. Six deluxe kits feature Hilman Rollers with steel chains, while five models come with special non-marring solid nylon chain for use on coated, painted or other specialized floor surfaces.

For more information, contact: Hilman, Inc., 2604 Atlantic Ave., Wall, NJ 07719; 908/528-0880; fax: 908-223-8072 or **CIRCLE # 130**

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Weiler's Black Gold Grooved Polyflex Encapsulated Cup Brush provides maximum cutting action and low per-piece-costs when doing heavy weld scale and rust removal, pipe bevel preparation prior to welding, surface refinement and juncture bending.

For more information, contact: Weiler Brush Co., 1 Wildwood Dr., Cresco, PA 18326; 717/595-7495; fax: 717/595-2002 or **CIRCLE # 121**

BAND SAWS

A new brochure from Peddinghaus describes the company's twin column structural steel band saw with guillotine blade approach for maximum efficiency when sawing structural steel.

For more information, contact: Peddinghaus Corp., 300 N. Washington Ave., Bradley, IL 60915; 815/937-3800; fax: 815/937-4003 or **CIRCLE # 118**

SAW BLADES

Quality saw blades at competitive prices are available from SAWS International. Featured are: high-speed steel circular saw blades; segmental circular saw blades; bandsaw blades; carbide-tipped metal cutting blades; and friction/hot saw blades.

For more information, contact: SAWS International, 1206 Kishwaukee St., Rockford, IL 61106; 800/435-7297 or **CIRCLE # 120**

BAND SAWS

Yamazen offers structural band saw machines and CNC structural drilling machines. The saws feature a patented amplifying valve feed system, unique three contact point mitering method, six degree saw bow angle and carbide roller ring back-up guides. The CNC drilling machines features an exclusive "fixed work piece-travelling spindles" method, which allows for faster hole location without repositioning heavy beams and results in greater accuracy.

For more information, contact: Yamazen, Inc., 735 E. Remington Road, Schaumburg, IL 60173; 708/882-8800; fax: 708/882-4270 or **CIRCLE # 131**

USED EQUIPMENT

Structural Steel Systems specializes in buying, selling, leasing and brokering equipment typically found in structural and plate fabricating facilities. Offers technical sales personnel, service staff, remanufacturing of used equipment, retrofits and financing options.

For more information, contact: Structural Steel Systems, 763 Lower Saucon Road, Hellertown, PA, 18055; 610/838-7338; fax: 610/838-7338 or **CIRCLE # 127**

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For more information, contact: Acrotech Inc., P.O. Box 466, Industrial Court North, Lake City, MN 55041; 800/345-0583; fax: 612/345-3759 or **CIRCLE # 112**

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For more information, contact: Franklin Manufacturing, Highway 243, Industrial Park, Russellville, AL 35653; 205/332-0143; fax: 205/332-0143 or **CIRCLE # 110**

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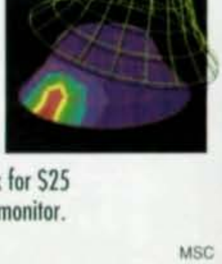

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