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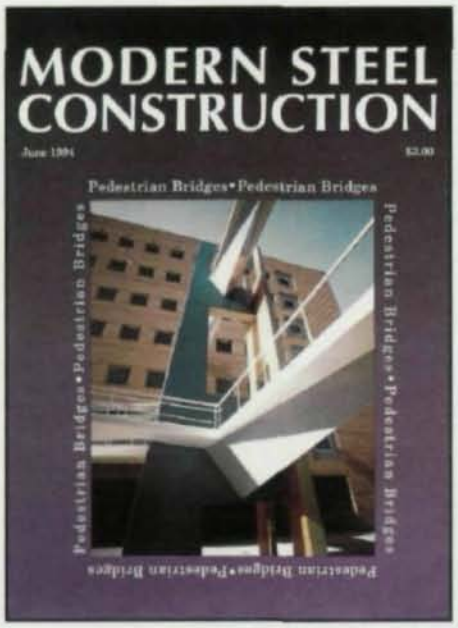
MODERN STEEL CONSTRUCTION

Volume 34, Number 6 June 1994



Users have a more intimate relationship with pedestrian bridges than they do with other types of bridges. MSC offers two stories (beginning on pages 30 and 42) this month featuring wide variety of these fascinating structures.

FEATURES

DEPARTMENTS

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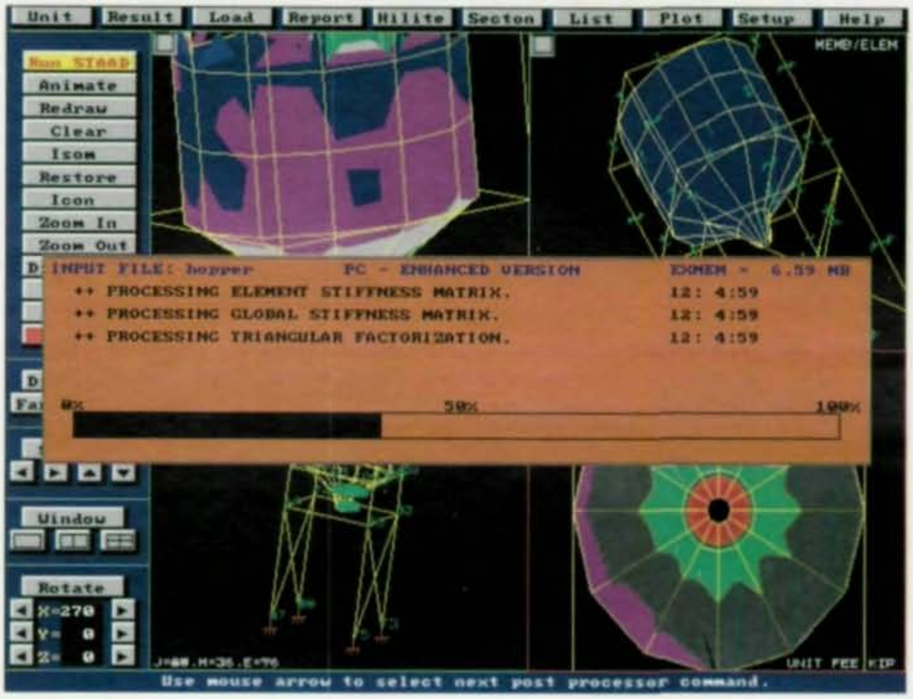

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DIGITAL REVOLUTION

WHEN I WAS GROWING UP, I SPENT LONG HOURS IN THE LIBRARY READING EVERYTHING I could get my hands on. I was particularly taken with encyclopedias and could spend hours on end paging through first one volume and then another. I often started one article, came across something of interest, skipped to that subject, and then was distracted by a third item.

As much as I loved Funk & Wagnalls, I was always frustrated by the tedium involved in searching for the specific information that struck my fancy. Fortunately, modern technology has solved that problem. I can now tuck a single CD-ROM disk (the equivalent of all those volumes on one disk!) in a personal computer and voila! All the information is at my fingertips. If I want to read about DeQuervains tenosynovitis, I can find the information in a few seconds with just the press of a few buttons.

I've been crowing for the past few months about exciting things happening at AISC, and now I can even say that AISC has joined the digital revolution. The new 2,163-page, two-volume LRFD Manual of Steel Construction is now available on a single CD-ROM. But it's not just a book on a disk. The CD version offers some amazing advantages. On each page, some of the text is highlighted in red. If you want more information on that subject, just click and the information appears. Other information is highlighted in green. This represents a circular pathway leading to other places in the manual dealing with the same subject. Click once and you find a new reference. Click again to find more references. Keep clicking and eventually you circle back to the first point.

LRFD on CD runs under Adobe Acrobat (included with the disk) and it runs fast. If you're a designer and need to look something up quickly, using the CD will be no problem. Access is almost instantaneous. As an added bonus, the CD includes two years worth of *Engineering Journal* issues—all with the same quick search features as the Manual. As I've said before, everyone interested in keeping abreast of the latest information on steel design should be reading EJ (to simply buy a one-year subscription to *EJ*, send $15 to: AISC, P.O. Box 806276, Chicago, IL 60680-4124.)

Of course, cutting edge technology doesn't come cheap, and a lot of people are going to balk at the $1,000 price tag for LRFD on CD. For engineers worried about the price, I have a simple suggestion. Become a professional member of AISC. It costs $150 to sign up (only $100 to renew), and that includes one free book (choose the manual—it's one of our most expensive publications at $72), a year's subscription to *EJ* (a $15 value), AND $250 off the price of LRFD on CD (along with 25% off every other purchase from AISC). In addition, you get substantial discounts on attending the National Steel Construction Conference and all of AISC's education seminars.

To purchase a copy of LRFD on CD, call 312/670-2400 ext. 433. To join AISC, call LeAnn Schmidt, AISC's new director of membership, at 312/670-5432. **SM**

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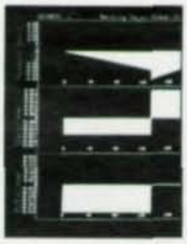

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Steel Interchange

Steel Interchange is an open forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine. If you have a question or problem that your fellow readers might help 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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Modern Steel Construction
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Chicago, IL 60601-2001

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

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

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

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

Is there any testing or research to demonstrate that metal deck, parallel to the girder, does indeed provide adequate restraint or should the beam be checked for the temporary construction condition?

The AISC specification (for both ASD and LRFD) states, "Steel deck with adequate attachment to the compression flange...will usually provide the necessary lateral support." The key words are "adequate attachment".

Professor Larry Luttrell of West Virginia University performed hundreds of horizontally loaded tests with steel deck attached to perpendicular and parallel members. These tests were part of a program to determine the diaphragm strength and stiffness provided by steel deck. Bracing the frame (by diaphragm resistance) is somewhat different from bracing the compression flange of a girder. However, the diaphragm tests showed that the attachments to parallel members were used and were significant in developing the diaphragm, and, by the same reasoning, could be used to brace the compression flange. As an example, a 5/8" diameter weld through 20 gage deck into a structural steel member will provide a design shear strength of 760 pounds. The Steel Deck Institute (SDI) specifies, "Floor deck units shall be anchored to ... perimeter support steel ... Deck units with spans greater than five feet shall have side laps and perimeter edges (at perimeter support steel) fastened at midspan or 36 inch intervals - whichever distance is smaller." The Commentary points out, "This anchorage may be required to provide lateral stability to the top flange of the supporting members." Certainly if the deck is not attached to the girder then the unbraced length would be the spacing of the beams supported by the girder. If the deck, as it is installed, does not cover to the girder and a closure piece is needed, then the closure should be attached to the girder and the deck at the spacing given by the SDI specification.

Richard B. Heagler, P.E.
Steel Deck Institute
Canton, OH

Under what circumstances does the designer have to consider torsion in the design of a beam?

Torsion occurs in beams when the line of action of any transverse force applied to the beam does not pass through the shear center of the beam. The shear center location for various sections vary depending on the section.

The response to torsion in steel members can be divided into two groups. The first consists of beams of closed sections, such as pipes and tubes, which resist torsion by shear stresses. The other group consists of open sections, such as wide flanges and channels, which resist torsion by combined shear and warping. Usually, the primary stress of interest in typical open sections is the normal warping stress component. This stress adds to the bending stress in the beam, hence, reducing its "available" capacity for bending.

It is a usual practice to repair existing members by providing side plates that would "box" the wide flange beam, and then it would act as a tube and normal warping stresses would be eliminated. In new construction, the engineer should account for the torsion directly, and either use tubes for the design or provide adequate section in the wide flange beam to resist both bending and normal warping stresses.

As for references on the subject, Steel Structures, 2nd Edition, by Salmon and Johnson provide a basic background on theory and presents a simple method for accounting for torsion in chapter 8. AISC publishes Torsional Analysis of Steel Structures with extensive tables and charts for the solution of the torsion problem. For those more inclined to equations, Roark's Formulas for Stress and Strain 6th Edition presents comprehensive solutions in the form of equations for many bound-

ary conditions and sections in chapter 9.
Hussain Shanaa, Ph.D., P.E.
AEC Engineering,
Minneapolis, MN

Serviceability is a particular concern for crane systems in industrial buildings but is not covered in standard code literature. What are deflection limits for crane runway systems?

One is referred to the Guide for the Design and Construction of Mill Buildings: Association of Iron and Steel Engineers Technical Report No. 13. Section 5.8.8 of this report limits crane runway live load deflections without impact to l/1000.

AISE Technical Report No. 13 also offers very specific design information for crane runway systems.

AISE is located at Three Gateway Center, Suite 2350, Pittsburgh, PA 15222-1097 (412) 281-6323. FAX: 412/281-4657.
Dennis T. Pay
Geneva Steel
Provo, UT

The use of channel sections or other light weight narrow flange sections as girts supporting non-bearing exterior wall assemblies against wind load is common practice. How is lateral instability of the unsupported compression flange accounted for when the wall is subject to outward pressure due to suction at the leeward face of the building? These outward forces are equal to or greater than the inward forces.

Lateral stability of the compression flange of a girt subject to suction loading is an important design consideration. The lateral force needed to stabilize the compression flange is generally considered to be something less than two percent of the strong axis load, provided that the girt is properly aligned without sag or twist. Common sag rods can provide adequate lateral support for girts up to 8" deep even though they attach only to the girt webs. For deeper girts lateral support for the compression flange can be attained by means of a continuous vertical bar stock member (used in conjunction with sag rods), say 3x1/4, attached to the eaves beam, to the inside flanges of all the girts, and anchored to the sill or floor slab. If there is an inside wall finish such as plywood (check your local fire code) or corrugated steel, it too can support the inside flange of the girt (if adequately attached). As mentioned above, the proper girt alignment is necessary for predicted performance, The attachment of the girt to the column must be adequate to prevent the girt end from "rolling."
David T. Ricker, P.E.
Payson, AZ

New Questions

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

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

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

I have searched for standards to use when designing hot-rolled or extruded stainless steel shapes made of AISI Type 304 or 316 (yield strength = 35 ksi; tensile strength = 80 ksi) without success. Cold-formed stainless steel design is covered by ASCE 8-90, but does not include the thicker walled extruded sections or hot rolled beam sections. The AISC Specifications do not include this material (Section A3) since the mechanical properties of stainless steel (an inelastic, anisotropic material) differ from those of structural carbon steels. Are there any design standards available? Or must the design engineer, with the help of available technical data and steel producer information, set the factors of safety and apply strength of materials and stability principles in designing these sections?
John M. Kropp, P.E.
Morrison Knudsen Corp.
Cleveland, OH

What type of framing is considered bracing the compression flange? Does the member bracing the flange have to be attached to the flange? If a 4 inch deep member frames into mid-depth of a 10 inch deep beam is that considered bracing the compression flange (center lines of each member at same point)? My interpretation is that it would not because I would think the web/flange could still twist and buckle. I have not witnessed any fully loaded testing to see how the beam reacts and the AISC specification is not very descriptive of what they consider bracing of the compression flange.
Joseph Cook

LRFD ON CD

LRFD on CD is more than just the entire two-volume, 2,000-page LRFD Manual of Steel Construction, 2nd Edition, on a compact disk.

This electronic manual makes extensive use of the latest hypertext technology to automatically link related sections of the manual together. Click on "connections" in one part of the manual and quickly and easily find other sections dealing with that same topic. Or click on any of the more than one thousand items that are electronically cross-referenced throughout the manual. The Manual includes a 45-page introduction to LRFD electronically linked to the Specification and Commentary.

The CD also includes:

- Complete copies of every issue of Engineering Journal published in 1992 & 1993
- Nearly 100 drawings (.dxf files) taken right from the manual that can be quickly copied to your AutoCAD or other CAD program

And best of all, LRFD on CD follows the exact format, page-by-page, as the Manual. If you're looking at nominal strength parameters on page 6-115 of the Manual, the identical page is reproduced on the CD.

LRFD on CD is available for $1,000 (or $750 for AISC members).

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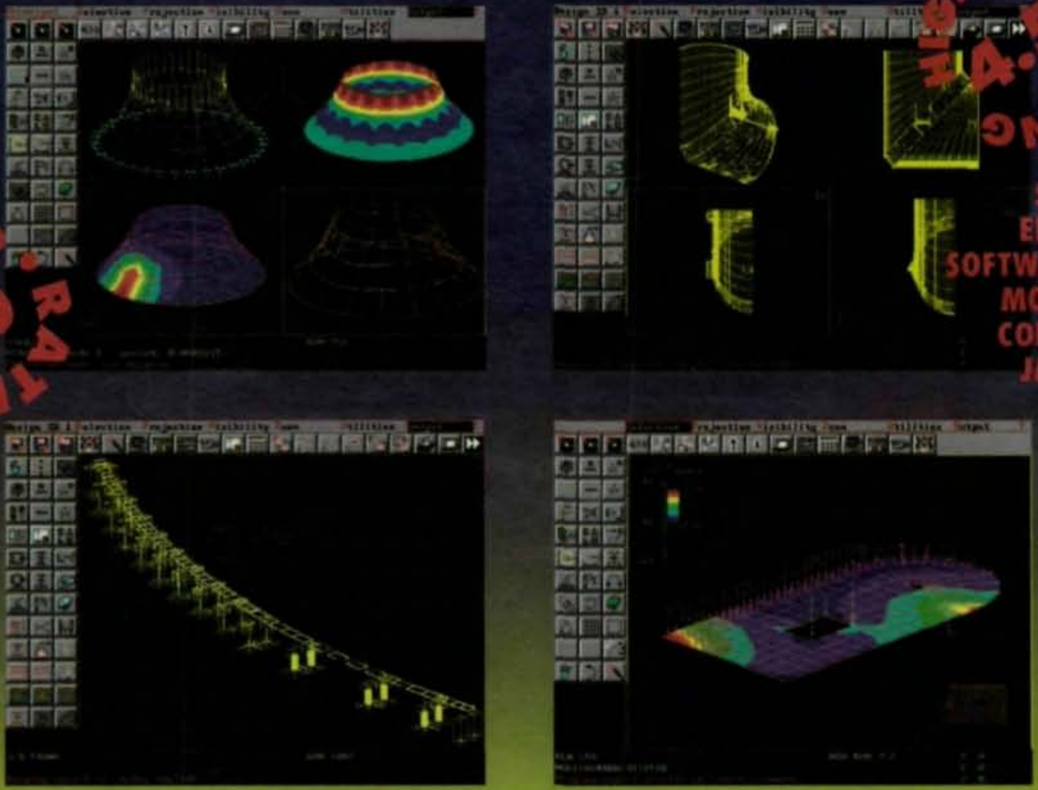

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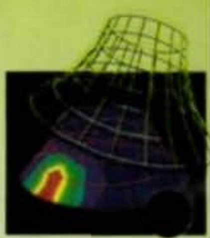

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This is the inaugural edition of a bi-monthly column dealing with legal matters of interest to designers, fabricators and contractors. We solicit your comments, concerns, suggestions and questions, both as to individual issues and subjects that you would like to see treated in this column. Some of the issues that we plan on covering in the future include: contract provisions; OSHA standards; employment law; alternative dispute resolution; and dealing with the EPA. Comments should be sent to: Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Connection Design, Insurance Coverage, And The AISC Code Of Standard Practice

Readers of this magazine certainly are not strangers to the controversy of connection design responsibility. Contract clauses and specification provisions require structural steel fabricators to forward connection design data to a project engineer by way of contract submittals that are prepared and certified by a professional engineer licensed in the state of the construction project. Oftentimes, these clauses are coupled with other contract provisions that: require the fabricator to assume responsibility for connection design; indemnify the engineer of record from any liability for connection design; and/or delete or modify section 4.2.1 of the AISC Code of Standard Practice.

AISC is currently involved in an interdisciplinary, national task force on design responsibility with the goal of resolving the sometimes conflicting interests of owners, engineers-of-record, general contractors and steel fabricators. This task force is making substantial progress and its results will undoubtedly be the subject of future columns.

In the interim, however, there are many practical questions that structural steel fabricators are asking about how to deal with clauses added to standard contracts and whether these clauses will effect insurance coverage in the event of a structural failure.

Virtually all commercial general liability insurance policies issued to structural steel fabricators contain a standard form exclusion similar to the following:

> "This insurance does not apply to 'bodily injury', 'property damage', 'personal injury', or 'advertising injury' arising out of the rendering or failure to render any professional service by or for you, including, (1) the preparing, approving or failing to prepare or approve maps, drawings, opinions, reports, surveys, change orders, designs, or specifications; and (2) supervisory, inspection or engineering services."

What does all that legalese mean? It depends on who you ask.

Popular wisdom in some circles is that connection design and/or development is the traditional work of steel fabricators. Some feel it is an incidental "means or method" of performance of the traditional fabricator subcontract and, as such, is not technically professional engineering work. These individuals believe that only the engineer-of-record for the project is capable of providing professional design services relative to the primary structural system. Others take the opposite viewpoint, pointing out that any function that requires an engineering education and engineering judgement should be considered professional engineering work.

Another popularly held belief is that so long as a licensed professional engineer in the state of a particular project does not affix his or her seal to a submittal, professional liability does not attach to work performed under a fabricator's contract. The converse of this view, held by many project design engineers, is that it is necessary to require a fabricator to affix the seal of a professional engineer licensed in the state of a particular project to all submittals in order to assure the engineer-of-record that the fabricator will bear responsibility for any errors in the fabricator's work.

Unfortunately, none of these viewpoints is necessarily true in all cases.

While we are unaware of any case law that specifically interprets the professional negligence exclusion of the standard form commercial general liability (CGL) insurance policy to the work of structural steel fabricators, there is extremely analogous case law that flies in the face of several of the commonly held assumptions previously mentioned.

The most analogous case on point is that of Harbor Insurance Company v. Omni Construction, Inc.

In this case, a foundation subcontractor agreed to "detail the specifics" of a temporary shoring and bracing system in keeping with the design intent of the "architect's plans and specifications."

One of the two engineers employed by the foundation contractor explained his work as follows:

> "[After] acquiring some basic construction documents, either provided through the owner or from the contractor...we...go through an evaluation of what is necessary...to have a working system....Forces would be computed and any reactions to determine...what kind of tie loads are required."

After completing the drawings of the system the subcontractor's engineer certified them as a professional engineer licensed in the state of the project.

The temporary shoring system thus submitted and constructed failed and caused approximately $1 million in damages to a structure adjacent to the construction site. The CGL insurance carrier for the project general contractor denied coverage based upon the standard professional exclusion quoted earlier in this article.

The general contractor argued that in the

David B. Ratterman, Esq., is Secretary and General Counsel for AISC. His firm, Goldberg & Simpson, P.S.C., in Louisville, KY, concentrates in the area of construction law.

custom of the construction industry, "professional services" performed by the subcontractor incidental to its construction work should be considered "means and methods of construction" and not be subject to the professional negligence exclusion. Among other witnesses, the general contractor put an employee of its insurance agent on the stand who testified that such was the understanding of the insurance industry, i.e., that the exclusion did not apply to "incidental" professional services performed by the subcontractors and that general liability policies were commonly understood in the industry to cover incidental professional services. An insurance expert employed by the general contractor testified that he had been assured by representatives of the insurance agency (presumably prior to executing the construction contract) that sheeting and shoring design work performed under the construction contract would be a general liability risk rather than a professional negligence risk.

THE CURRENT AISC GROUP CGL INSURANCE POLICY PROVIDES COVERAGE FOR STEEL FABRICATORS SO LONG AS THE AISC CODE OF STANDARD PRACTICE IS INCORPORATED INTO THE CONTRACT DOCUMENTS AND SECTION 4.2.1 OF THE CODE HAS NOT BEEN MODIFIED OR EXCLUDED

The general contractor made all of these arguments to try to prove to the court that actual industry practice created an ambiguity in the insurance policy that should be construed against the carrier. Unfortunately, the court did not buy any of these arguments. It held that the work performed by the foundation subcontractor was design work and that the language of the insurance policy, quoted above, clearly and unambiguously excluded coverage for these services.

The court relied upon two other analogous federal cases (Natural Gas Pipeline v. Odom Off Shores Surveys, Inc., and Noyes Supervision, Inc. v. Canadian Indemnity Company) to define the concept of professional service. These cases are of particular interest in that they hold that a court must look "to the character of the services performed, such as whether special knowledge and technical expertise are required, rather than the title or character of the party performing the services." In neither of these two cases were the individuals who actually performed the services necessarily professional engineers and in neither case did documents produced by these individuals bear the seal of a professional licensed in the particular jurisdiction.

In each case, the key issue addressed by the court did not appear to be the presence or absence of an engineering seal but, rather, whether the work involved discretion acquired by special training and the exercise of professional judgement.

Therefore, under this series of cases, connection development services could be excluded from many comprehensive general liability policies irrespective of whether the fabricator submits documents under the seal of a licensed professional engineer. On the other hand, a carte blanche requirement that submittals be sealed by a fabricator's professional engineer would not necessarily insulate an engineer-of-record from professional liability if he or she errs in the exercise of professional judgement in overall project design or in the review of fabricator submittals.

One of the objectives of the interdisciplinary design responsibility task force is to reach a nationally recognized definition of which connections require professional design input (e.g., complex moment connections on a seismic structure) and which connections do not (e.g., simple shear connections taken directly from AISC publications).

In the interim, fortunately, structures are not being built by lawyers and judges and life does not have to be this complicated. Insurance coverage is available to protect owners, designers, general contractors and fabricators in these circumstances.

The current AISC group CGL insurance policy will provide coverage for fabricators who perform connection design incidental to steel fabrication contracts so long as the AISC Code of Standard Practice is incorporated into the contract documents and Section 4.2.1 of the Code has not been modified or excluded from the contract (incidentally, unauthorized modification of the Code may violate AISC's copyright). If Section 4.2.1 of the Code has been excluded or modified, coverage will likely be denied in the event of a connection failure. However, if the foregoing conditions have been met, coverage will be provided even if the contract requires that submittals bear the seal of a professional engineer licensed in the state of the project. Owners, design professionals and general contractors need to be made aware of these facts during the course of contract negotiations.

Many design engineers are concerned because they believe that Section 4.2.1 of the Code requires the engineer-of-record or the owner to indemnify the fabricator for any errors that the fabricator may have made in the process of preparing submittals.

The interdisciplinary task force is currently also working on language that would alleviate these concerns. In the interim, it may be possible to sculpt language in individual construction contracts that would protect the interests of both the fabricator and the engineer-of-record, and provide insurance coverage for all concerned, without emasculating Section 4.2.1 of the Code. However, such language can only be evaluated on a case-by-case basis.

Competent counsel should be employed in drafting such language and the particular insurance carrier must be involved in the review process. Specific questions in the area of insurance coverage should be directed to the insurance consultant for AISC: Norman-Spencer, Inc., 377 East Butterfield Road, Suite 260, Lombard, IL 60148 (800) 842-3653.

Composite Design Work Honored

PIONEERING WORK ON THE PRACTICAL APPLICATION of composite construction has earned Lawrence G. Griffis, P.E., this year's T.R. Higgins Award. Griffis and his Houston-based firm, Walter P. Moore and Associates, Inc., have built a number of high-rise buildings utilizing composite design, including the 53-story Gulf Tower in Houston, the 49-story First City Tower in Houston and the 35-story 312 Walnut building in Cincinnati (which also utilized LRFD). Griffis' work on composite design is continuing and it is expected that in the future composite design will also be shown to have applicability for low-rise construction.

AISC's prestigious T.R. Higgins Award recognizes an outstanding lecturer and author whose technical papers have made an outstanding contribution to the engineering literature on fabricated structural steel. Recent winners have included Roberto Leon, a professor at University of Minnesota who was recognized for his work on partially restrained connections, and William McGuire, a professor emeritus at Cornell University who was honored for his work on computers and steel design.

Griffis, who received the award along with a $5,000 cash honorarium at this year's National Steel Construction Conference, will lecture on composite construction in six locations throughout the country during the next 12 months.

Stability Of Steel Structures

THE STRUCTURAL STABILITY RESEARCH COUNCIL HAS PUBLISHED "Stability of Metal Structures—A World View", a 940-page book containing comprehensive world-wide studies of more than 100 specifications and codes on stability design of metal structures. It evaluates different specifications and codes, compares and contrasts them, and explores some of the major reasons for their differences.

Divided into 14 topics, the book condenses the specification provisions and then gives regional and international comparisons and comments. Topics include compression members, built-up members, beams, plate and box girders, beam-columns, frames, arches, triangulated structures, tubular structures, shells, cold-formed members, composite members, earthquakes, and general provisions and design requirements.

The book is available for $85 from: SSRC, Fritz Engineering Laboratory, 13 E. Packer Ave., Lehigh University, Bethlehem, PA 18015; phone: 610/758-3522; fax: 610/758-4522.

Seismic Load Provisions

THE AMERICAN SOCIETY OF CIVIL ENGINEERS HAS MADE SWEEPING CHANGES to the seismic load provisions of its latest "Minimum Design Loads for Buildings and Other Structures Standard (ASCE 7-93)."

The new standard features revised earthquake load criteria and associated load combinations for the design and construction of buildings and other structures subject to earthquake ground movement. Designed to provide a more reliable and consistent level of seismic safety in new building construction, the new standard replaces the seismic portion of the previous edition, ASCE 7-88.

Representing a new methodology, ASCE 7-93 seismic load provisions are based on a strength-level-limit state and contain a much larger set of non-load provisions for seismic safety. The revisions are based on the 1991 edition of NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings. The seismic load provisions of ASCE 7-93, however, permit the use of allowable stress design standards. The new standard provides requirements for dead, live, soil, wind, snow and rain loads and their combinations that can be included in building codes and other design documents.

The standard is available for $44 ($33 for ASCE members). To order the 150 page document, call ASCE Central at 800/548-ASCE.

Bridge Conference

"THE BUSINESS OF BRIDGES" IS THE THEME for the 11th Annual International Bridge Conference and Exhibition, to be held June 13-15 in Pittsburgh. Sponsored by the Engineers' Society of Western Pennsylvania, the three-day conference will include more than 60 paper presentations as well as more than 100 exhibitors.

Topics include: design; inspection and rehabilitation; instrumentation and testing; seismic design; and construction. A full afternoon will be devoted to the developments and advances made in the state of Virginia (the IBC 1994 featured state).

In addition to the technical sessions, four half-day seminars—all offering continuing educational credits—will be held. Subjects include: An overview of AASHTO's LRFD (two separate seminars); A specifier's guide to overcoming lead paint; and implementation of seismic isolation design for bridges.

For more information, call the Engineers' Society of Western Pennsylvania at 412/261-0710.

Painting Steel Structures

THE THIRD EDITION OF "GOOD PAINTING PRACTICE," Volume One of the Steel Structures Painting Manual, is now available. The two-volume manual is a comprehensive encyclopedia on painting methods, equipment, and paint systems for structural steel.

Two new chapters have been added to Volume One. "Environmental Concerns and Regulations" describes how the application and removal of coatings affects the environment and covers the regulations and regulatory agencies that control the use and removal of coatings. Air quality, waste disposal, water quality and hazardous materials are discussed in this new chapter.

"Safety and Health in the Protective Coatings Industry" touches on some of the previous safety issues introduced in earlier editions of Volume One, but the new edition addresses proper safety and health practices for the entire coatings industry. It also includes information on the role of regulatory agencies such as NIOSH and OSHA.

Volume One is available for $90 from SSPC, phone 412/687-1113.

CALL FOR PAPERS

ABSTRACTS ARE BEING ACCEPTED UNTIL AUGUST 1, 1994, for the Nordic Steel Construction Conference '95. Sessions include: New Materials & Products (high strength steel, stainless steel, thin-walled structures, tubular structures, and welded and bolted connections), New Bridge Developments, Steel in Hybrid Construction, and Integrated Design and Construction Technology.

Abstracts should not exceed 250 words and can be sent to: Nordic Steel Construction Conference '95, Lulea University of Technology, Division of Steel Structures, S-971 87, Lulea, Sweden.

EARTHQUAKE GROUND MOTIONS

THE APPLIED TECHNOLOGY COUNCIL HAS PUBLISHED "Proceedings of ATC-35 Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice."

This 478-page report, written for practicing structural and geotechnical engineers, documents state-of-the-art information pertaining to regional earthquake risk, estimation of ground shaking, and implications for engineering design practice in five seismic prone regions of the country (Southern California, Northern California, Pacific Northwest, Central U.S., and Northeastern North America).

The report costs $45 and is available from ATC, 555 Twin Dolphin Dr., Suite 550, Redwood City, CA 94065; phone: 415/595-1542; fax: 415/593-2320.

BEAUTIFUL STRUCTURES

A NEW BOOK ON SANTIAGO CALATRAVA'S RECENT WORK has been published to coincide with an exhibition in London of the famous Spanish architect/engineer. "Santiago Calatrava—2nd Edition" is illustrated with photographs of many projects, as well as Calatrava's own sketches and drawings. Projects included since the publication of the first edition include: the Reichstag Conversion in Berlin; St. John the Divine in New York; London Underground Stations; and Trinity Bridge, Salford, England.

Copies are available from E&FN Spon, 2-6 Boundary Row, London, England SE1 8HN; phone: 071-865-0066; fax: 071-522-9623.

INNOVATIVE PRACTICES IN STRUCTURAL STEEL

AN INTRODUCTION TO THE 1993 LRFD SPECIFICATION and new Manual of Steel Construction, 2nd Edition highlights AISC Marketing's popular four-part seminar series, "Innovative Practices In Structural Steel." The lecture will include a discussion and explanation of the changes, including such items as the stability of unbraced frames, web crippling equations, and slip critical joints at factored loads.

The seminar also includes a session on state-of-the-art steel design software, the latest NEHRP Seismic Regulations, and a review of semi-rigid composite connections.

The seven-hour seminar costs $90 ($75 for AISC members), including dinner and a variety of handouts. The lecture has a CEU value of 0.4. For more information, call 312/670-2400.

1994 Seminar Dates & Locations

WEST	
Sacramento	June 15
San Francisco	June 16
Los Angeles	June 22
Los Angeles	June 23
Seattle	June 27
Salt Lake City	June 29
Phoenix	Oct. 20
Portland, OR	Nov. 15
Las Vegas	Nov. 17
MIDWEST	
Minneapolis	June 2
Detroit	Oct. 11
Indianapolis	Oct. 13
Cleveland	Oct. 25
Columbus	Oct. 26
Cincinnati	Oct. 27
NORTHEAST	
Albany	Sept. 13
Rochester	Sept. 14
SOUTHWEST	
Houston	June 21
Oklahoma City	Sept. 8
SOUTH	
Atlanta	Sept. 20
Richmond	Sept. 22
Memphis	Oct. 18
Miami	Nov. 1
Orlando	Nov. 3
MIDATLANTIC	
Edison, NJ	Oct. 4
Philadelphia	Oct. 6

STEEL DAMAGE IN LA: WHAT WENT WRONG

Case studies on localized steel problems in buildings affected by the Northridge Earthquake

IN MARCH, A TASK FORCE OF SOME OF THE LEADING SEISMIC RESEARCHERS AND DESIGNERS MET to assess the impact of the January 17, 1994 Northridge earthquake on structural steel buildings. What follows are condensed versions of case studies presented by task force members. A complete report is available from AISC (send $5 to AISC-Earthquake Report, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001).

The number of affected buildings is now reported as in excess of 50. Note, however, that in no case did the connection problems result in the collapse of any structure or in any injuries. Current seismic codes are designed for life safety, and in that respect steel buildings performed adequately.

Steel Properties Of A Selected Building

	A36 Beams	Gr. 50 Columns
Yield Strength F_y (ksi)		
Minimum	41.07	58.15
Mean	**48.44**	**64.80**
Maximum	63.28	73.70
Tensile Strength F_u (ksi)		
Minimum	60.68	72.73
Mean	**71.63**	**79.93**
Maximum	76.63	86.13

Bolted Web-Welded Flange Moment Connections

(This section is adopted from a paper presented by Thomas A. Sabol, Ph.D., S.E., president of Englekirk & Sabol Consulting Engineers, Inc., Los Angeles)

BASED UPON A REVIEW OF AVAILABLE INFORMATION, it appears that joint failures in bolted web-welded flange moment connections possess one or more of the following general attributes:

1. Failures of the frame girder bottom flange welds;

2. Failures of the frame girder top flange welds;

3. Cracks in the shear tab or shearing of web bolts;

4. Cracks in the frame column; and

5. Divots of steel removed from the face of the column flange.

By far the most common problem was failure of the frame girder bottom flange welds. Examination of the failed welds indicated the presence of a thin line of slag along the root of the weld. The next most common failure attribute is the horizontal crack across the column flange and, in many instances, across the column web. In selected instances, the crack appears to have resulted in divots of steel being pulled from the column flange at the girder flange weld on each side of the girder web rather than propagating as a horizontal crack across the column. Failures of the shear tab have included vertical cracking of the tab leaving the bolts intact as well as shear tab connections that have lost more than 50% of the bolts in shear. The least common problem is the failure of the frame girder top flange weld. It is interesting to note that there were no reported failures of column splices.

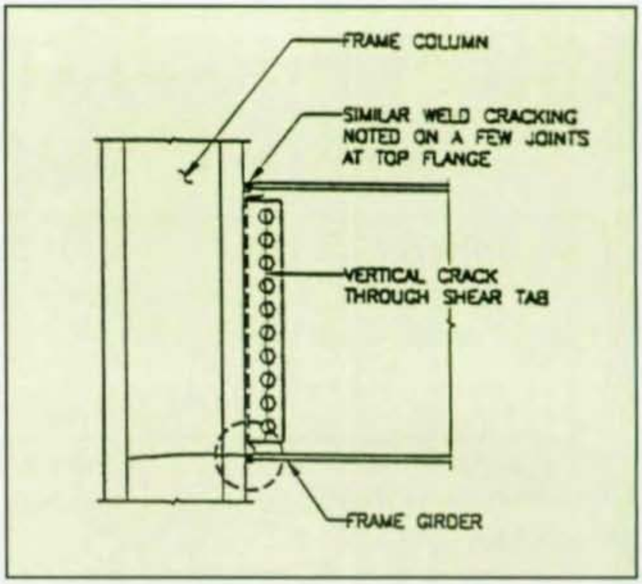
The range of building characteristics in buildings with connection problems is extremely broad, as is building location. Problems were reported in buildings from one story to as many as 22 stories, though evidence suggests that in taller buildings most problems occurred in the upper half to two-thirds of the building. The extent of the problem has ranged from less than 10% to as much as nearly 100% in specific floors and/or specific compass directions. Problems have been reported in connections with and without column flange stiffeners as well as connections with and without return welds on the shear tabs. Both wide flange columns and built-up box sections appear to have been affected.

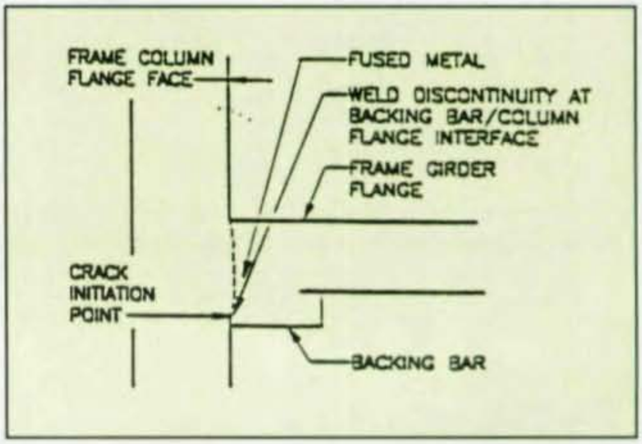
From a materials standpoint, it appears that most, if not all, of these frames were welded based on AWS D1.1. Steel for beams has been both A36 and A572 Grade 50, while most, but not all, of the columns appear to have been of Grade 50 stock. Based on mill certifications taken from one building, it appears that the mean column yield and tensile strengths exceed those of the beams for combinations involving A36 beams and Grade 50 columns. Selected toughness testing of steel from one building suggests that Charpy V-Notch toughness, though not a requirement for the sections examined, exceeded values considered the lower limits of acceptable toughness. Toughness testing of the weld metal obtained similar findings, though selected samples were at the lower end of the acceptable toughness range.

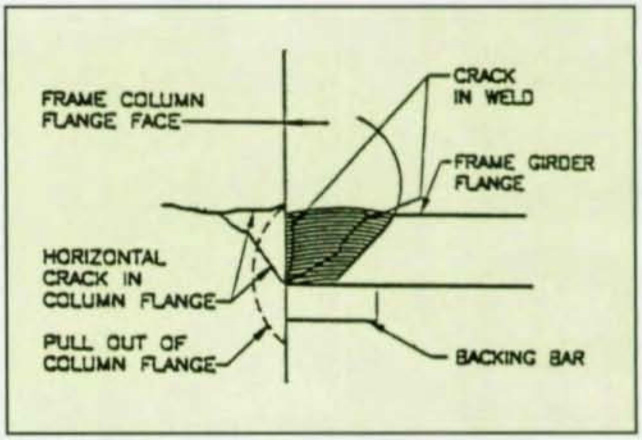
Recent tests of bolted web ductile frame connections (Engelhardt, 1993) and a reexamination of similar tests stretching back as far as 1972 (Popov, 1972; Popov, 1986; Tsai, 1988; and Anderson, 1991) suggest that these welded connections show a large variability in performance, maximum attainable plastic rotation, and weld failure rates. The observed post-earthquake condition of the failed connections appears to mirror the results of the least reliable connections in almost every aspect except for the damage to the columns, which does not appear to have been reported in the literature.

Since it is generally assumed that beam flanges transfer the majority of the moment to the column, although M_p is based on the full plastic section modulus including the beam web, developing the full value of M_p can generate stresses in the flange welds that approach or exceed their ultimate strength. As a result, even if welds are perfectly executed, the level of stress in the weld is quite high.

As previously discussed, the examination of several failed welds revealed the presence of slag inclusions all along the root of the weld. The inclusions in the weld increased in size at the web cope, possibly due to the relative degree of difficulty the welder experienced in continuing the weld to the other side of the member. These welds are highly







stressed and demand particular care to ensure that the welds have no material flaws. Special inspection of the welds using ultrasonic testing was commonplace on most, if not all, of the projects experiencing connection failures. Since weld failures were reported on projects that were known to have been constructed in accordance with general standards of practice, wide spread, faulty fabrication does not appear to be the primary cause of connection problems.

Another suggested contributing factor may be the difficulty in executing the bottom flange weld while maintaining proper weld interpass temperatures and evenly distributed cooling. The bottom flange welds were usually executed on one side of the web and then the other, giving rise to the possibility that differential cooling may lock in stresses. Examination of a few damaged welds reveals that only half the bottom flange weld has cracked, and, in addition, some welds, judging by the presence of rust in the crack, appear to have cracked prior to the earthquake.

An emerging consensus suggests that these highly stressed joints are extremely sensitive to even minor imperfections in the welds. At least at the bottom flange, the presence of the backing bar in the final connection appears to permit the initiation of a crack at levels of stress and strain that, taken for the section as a whole, are less than levels required to develop significant plastic rotation in the beam.

Repair is already underway on the known problems and in many instances has already been completed.

Case Studies

(This section is taken from a paper presented by R.H.R. Tide, Ph.D., P.E., of Wiss, Janney, Elstner Associates, Inc., Northbrook, IL)

CONNECTION FAILURES WERE DISCOVERED in two adjacent and nearly identical 18-

story structures. The column sizes were W14x311 to W14x730 and the beam sizes were W36x135 to W36x300. Column web stiffener plates were not present.

All of the 42 fractures occurred in the bottom flange of the north-south moment frames located on the east and west sides of each building. A large proportion of fractures occurred in the west frame of each building. In the east building, many fractures occurred in four floors near the upper levels of the building. In the west building, fractures occurred throughout the height of the south column of the west frame.

Because the buildings were occupied, access to each location was restricted. As a result, the repair consisted of removing a small length of beam bottom flange to provide access to the face of the column flange. The column face was dressed up by buttering layers of weld metal, ground smooth and inspected using ultrasonic testing (UT) procedures.

A flange plate with an area approximately equal to the beam flange plus one-sixth of the web area was then welded in place. The complete joint penetration weld was UT inspected after the welding was completed and again the following day. The fillet welding was intermittently inspected during the process.

A THREE-STORY STEEL-FRAMED STRUCTURE UNDER CONSTRUCTION on top of a five-story concrete structure experienced numerous fractures. Because of two north-south expansion joints, the structure probably behaved as three independent structures.

The column sizes were W24x192 to W24x335 and the beam sizes were W36x135 to W36x210. The top and bottom corners of the web shear plates were welded to the beam web to reinforce the bolted connection. The concrete floor and roof deck pouring had not been completed at the time of the earthquake.

Fractures originating in the backing bar notch area propagated in some cases through the column flange and into the beam web. Extensive visual and UT inspection located 10 complete flange fractures, 65 cracks greater than $\frac{1}{4}$ in. and up to $1\frac{1}{4}$ in. into the column flange. In addition, nine cracks were also found opposite the top flange with four of these cracks occurring in columns with flange fractures adjacent to the beam bottom flange.

Samples were removed from three locations and sent to ATLSS laboratories at Lehigh University for metallurgical evaluation. Unfortunately, the backing bars had been removed before the samples could be obtained. The column flange material conformed to ASTM A572 with excellent Charpy V-notch (CVN) toughness. The weld metal contained high carbon and aluminum content common for AWS A5.20 grade of Flux Cored Arc Welding (FCAW) used on this project. However, the weld metal CVN toughness results were low as would be expected with the quantity of aluminum present.

Because the structure was under construction, unrestricted access was available to all beam-column connections.

Hidden Damage

(This section was taken from a paper by EQE Engineering's Los Angeles office)

ONE OF THE DIFFICULTIES IN DETERMINING THE EXTENT OF THE DAMAGE caused by the Northridge Earthquake is that much of the damage was not apparent at first glance. An example of this difficulty is a complex north of the epicenter that included a four-story steel SMRF building and a one-story, irregularly shaped SMRF building.

When the buildings were initially inspected on January 19, the four-story building had no apparent structural damage. It was not noticeably out of plumb, interior damage was not excessive (some ceiling tiles fell, there was some cracking of gypboard walls), and exterior damage was limited to chipping at corners of some precast panels and some broken glass.

However, the one-story building had some obvious structural damage.

Observable damage included: the roof diaphragm cracked across the width and opened about 2 in. at the building perimeter; the roof dropped about 4 in. where the crack intersected the perimeter wall; a beam connection failed due to high collector loads; a roof beam dropped and the web landed on the bottom flange of a perpendicular beam, and the building was noticeably out of plumb (about 4 in. in each direction at one corner). When fireproofing was removed from some moment connections, some cracked column flanges and webs were discovered.

After the initial cleanup, minor separation of the fireproofing was noted at the bottom flange of one beam-to-column connection of the four-story building. When the fireproofing was removed, a connection failure was found. Adjacent columns were checked, and more failures were found.

The building's overall dimensions are 217 ft. in the north-south direction and 113 ft. in the east-west direction. The building has two moment frames in each direction and there are four bays in the N-S direction and three bays in the E-W direction.

Investigations revealed several failure modes, all of which were brittle failures, and the damage was limited to the bottom flanges.

Failures included: cracks across the width of the column flange (both extending into the web and not extending); U-shaped cracks across the column flange (not full flange width) sometimes extending into the web; weld metal fractures; and fractures at the weld metal-column flange interface. A survey of the interior columns found that the building is out of plumb (at the roof, $2^1/_4$ in. to south and $1^3/_8$ in. to the west; at the second floor, about $1^3/_8$ in. to the south and 1 in. to the west).

Sections of three failed beam flanges-to-column connections were sent to a lab for analysis and the materials conformed to A572 with good Charpy tests and the hardness increased at the welds.

Factors that might have contributed to the damage included: run-off tabs were oriented the wrong way and may have trapped slag, which could result in the lack of fusion detected at the ends of the flanges; the continuity plate thickness was equal to one-half the flange, which could promote stress concentration; the continuity plate welds changed to fillets from complete joint penetration during construction, which also could promote stress concentrations; the bottom continuity plate was in line with the bottom of the beam flange, which creates eccentricity, and some stress was transferred by direct tension across the column flange with the remainder by shear (if the tension is already present in the flange, the principal tension oriented across the crack); during original welding, preheat may not have been adequate and welds may have cooled too rapidly; and higher stresses in the welds that in the beam flanges could have combined with weld defects.

The repair of the four-story building consisted of:

1. Removing the damaged beam flange welds and the beam flange welds with a lack of fusion.

2. Air-arc damaged column flange to sound metal and weld repair. Where the crack extends through the flange, sections of flange were replaced.

3. Replacing continuity plates at moment connections with plates with a thickness matching that of the beam flange and welding to the flanges with cjp welds.

4. Re-welding beam flanges to columns.

5. Welding triangular plates to the top and bottom of the beam (see MSC April 1994). The plates were sized so that the section comprised of plate and flange provides 1.5 M_p of beam.

6. All welding of columns, beam flanges and continuity plates utilized preheat, slow cooling and peening.

The repair of the one-story building consisted of:

1. Bottom moment connections were cut (even those not damaged), shear tab welds were cut, and all beam-to-column connection bolts were loosened, and top and bottom bolts were removed to facilitate pulling the building back to plumb.

2. Wire ropes were used to pull the building to less than 1 in. out-of-plumb in each direction.

3. Moment connections were reinstated to pre-earthquake conditions.

4. Diaphragm was repaired and the capacity of the beams to act as collectors was increased.

5. Chevron bracing was installed in moment frame bays to serve as the primary lateral load resisting system.

Ongoing Research

During May, Michael D. Engelhardt, Karl H. Frank, and Joseph A. Yura, all of the University of Texas at Austin, were scheduled to begin a testing program on moment connections. Results are expected sometime during this summer.

The goal of the research is to provide short-term answers to the moment connection problems experienced by some buildings as a result of the Northridge Earthquake. The testing program will provide experimental verification of improved moment connection details for use on buildings currently under design or construction and for the retrofit of existing connections.

The testing will subject large scale subassemblages to cyclic loading. Cantilever type test specimens will consist of a portion of a column and of a beam, with cyclic loads applied at the end of the beam. This setup will provide a reasonable representation of the loading environment of the beam-to-column connection. It's anticipated that the specimens will be very large scale, with beams in the range of W30s to W36s, at about 150 lbs. per ft.

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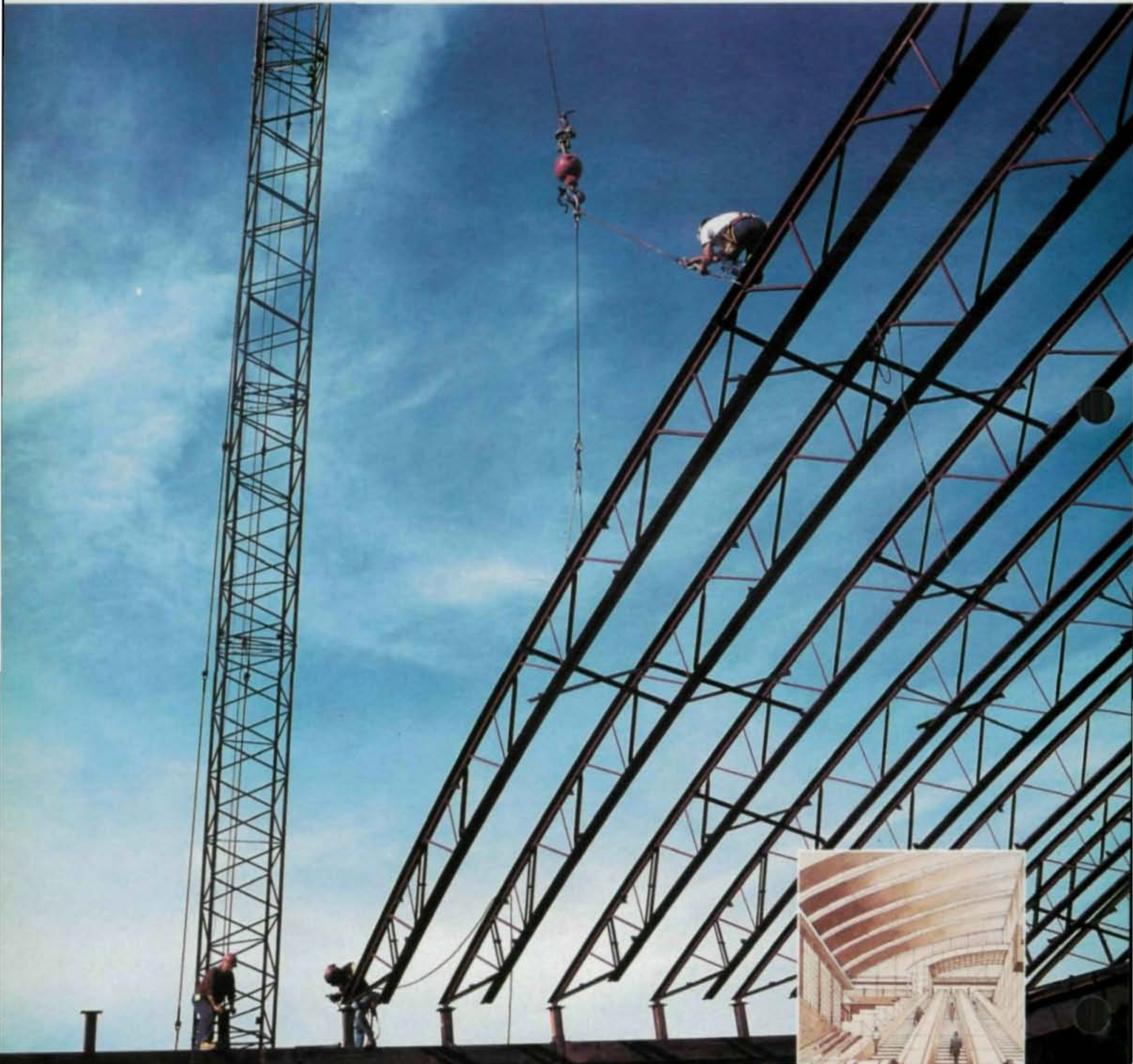

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Deflection Criteria For Spandrel Construction

By Michael A. West, P.E.

SPANDREL BEAMS AND GIRDERS ARE CRITICAL ELEMENTS in building frames because they support a major component of the building envelope, namely, the curtain wall. Careful coordination of the design of the spandrels and the design of the curtain wall and attention to detail is essential for successful building performance.

In 1990, AISC published Design Guide No. 3; "Serviceability Design Considerations for Low-Rise Buildings." One portion of this guide covered the issue of the interaction of the building frame with the design and performance of cladding. The discussion and recommendations presented in this article are largely taken from Design Guide No. 3. (Copies of this Design Guide can be purchased from AISC for $16 + $5 S&H by calling 312/670-2400 ext. 433.)

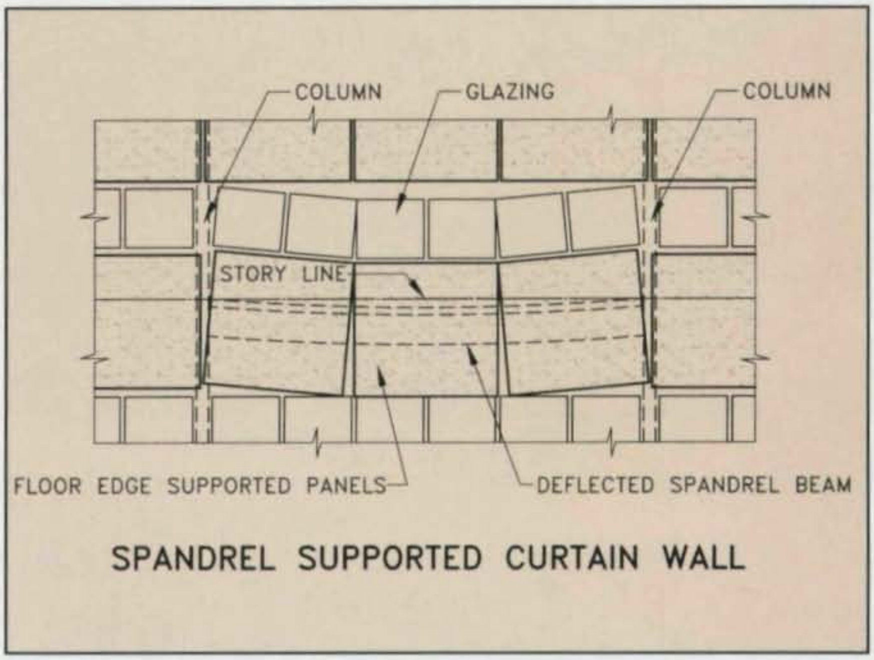
In considering this topic, two principal distinctions must be made:

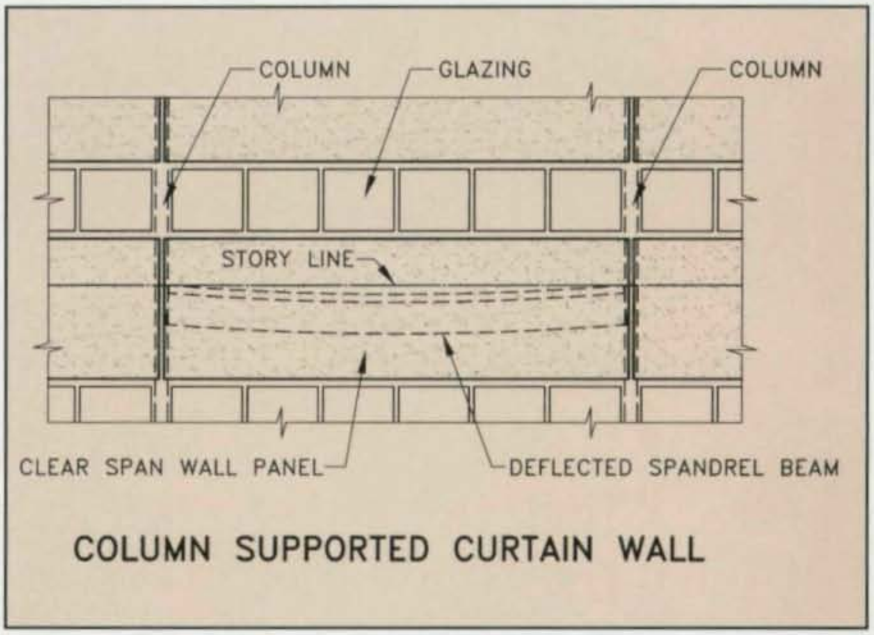
1. Is the curtainwall supported by the spandrel or is it supported by the foundation and the building columns?

2. If the curtainwall is supported by the spandrel, is the load from the curtain wall a major or minor component of the total load?

Not Supported

IN THE CASE WHERE THE CURTAINWALL IS NOT SUPPORTED by the spandrel, the primary concern for deflection is the control of cracking and distress in the joint between the wall finish and the floor finish. Since the wall and floor (or roof) are supported independently, they are loaded and deflect independently. The





range of movement depends on the finishes, but, for example, typical office occupancy finishes can tolerate vertical differential movement on the order of one quarter to one half of an inch. The appropriate load to use in calculating this deflection is 50% of the design live load. This reduction is reasonable provided the occupancy is such that full live load is not expected along the spandrel. For example, extensive files would not likely be placed along a window wall. Lastly, the curtain wall tie back connections must be detailed to accommodate the range of vertical movement at the building perimeter.

Supported

WHEN THE SLAB PERIMETER AND SPANDREL SUPPORT the gravity load of the curtainwall, the concern for deflection shifts from the floor wall joint to the performance and integrity of the curtain wall itself. Additionally, when the curtainwall represents a major portion of the spandrel total load, there is concern for deflections which occur as the pieces of curtainwall are erected. Lastly, the deflected curve of the spandrel prior to setting the curtain wall represents a field condition which should not differ markedly from the elevations portrayed on the construction documents and the curtain wall shop drawings. The magnitude of movements of the building perimeter and spandrel before, during and after the curtain wall installation has a major impact on the detailing of joints, connections and the provisions for field adjustment of the curtainwall system.

In the case of relatively light cladding, i.e. when the load from the curtain wall is less than or equal to 25 percent of the total spandrel load, the following limits apply. The recommended limit on spandrel deflection prior to erecting the curtain wall are span over 480 with a absolute limit of $^3/_8$ in. The loads appropriate to this calculation are those in place prior to cladding. The deflection due to all dead loads including the curtain wall, should be limited to span over 480 with an absolute limit of $^5/_8$ in.

The deflection limits for live load are based on the movement allowed by the detailing of the completed curtainwall.

Although large movements could theoretically be accommodated, common detailing practice would allow for movements in the range of $^1/_4$ in. to $^1/_2$ in. These deflections would be the net allowable movement after accounting for the dead load placed after the curtain wall is completed. In relative terms the live load deflection should be limited to span over 360. The portion of design live load used depends on the expectation of its presence and could vary between 50 and 100 percent.

When the curtainwall weight exceeds 25 percent of the total dead load on the spandrel, different limits apply. In this case, the critical dead load deflection is that due to the dead load in place at the time of cladding plus the dead weight of the curtain wall itself. The deflection due to these loads should be limited to span over 600 with an absolute limit of $^3/_8$ in.

The deflection limit for dead loads imposed after cladding and live loads are the same as for the case of relatively light cladding. They are:

a. Additional dead load: span over 480 with an absolute limit of $^5/_8$ in.

b. Live load: span over 360 with an absolute limit in the range of $^1/_4$ in. to $^1/_2$ in.

It should be noted that these limits are set by the allowable movements in the joints and connections. Thus, for the most part camber is not a means to address these concerns. The only exception would be deflection prior to cladding. However, these loads and deflections are relatively low in magnitude and probably would not justify cambering the beams.

Other Considerations

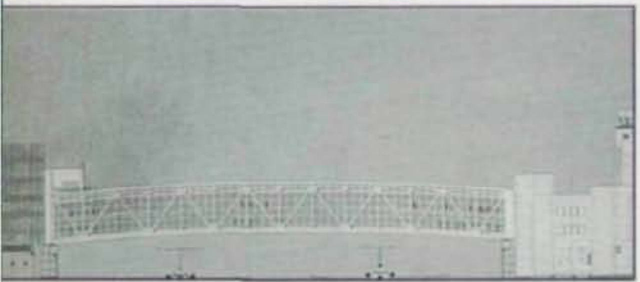
ADDITIONALLY, THERE ARE THREE OTHER important considerations. First, although it is common to design spandrels for a uniform load to account for the curtainwall weight, it should be recognized that in fact the curtainwall weight is applied at discrete points as concentrated loads. When the concentrated loads are closely spaced, the uniform load is reasonably accurate. However, when the curtainwall elements can span horizontally, such as large precast elements, then the concentrated loads can become quite significant and a uniform load design would likely be inaccurate.

Second, it is common for some types of curtainwalls to make gravity load connections on alternate floors, which could invalidate a design applied the load to every floor. Thus the structural construction documents should describe the assumptions made on curtain wall loading and set limits on the magnitude *and location of* curtainwall reactions. If it is advantageous for the curtainwall supplier to depart from these limits, revisions should be proposed so that the structure can be redesigned to accommodate them.

Third, the structural construction documents should list the magnitude of anticipated dead and live load deflections so that the movement can be accounted for in the detailing of the curtain wall.

Since many aspects of the curtain wall detailing take place after the structure is designed, drawn and bid, clear communication among all the parties involved in the structure and the curtain wall is essential for good performance of both systems.

MICHAEL A. WEST, P.E., IS VICE PRESIDENT OF COMPUTERIZED STRUCTURAL DESIGN, INC., MILWAUKEE.



PRECISE MOVEMENT

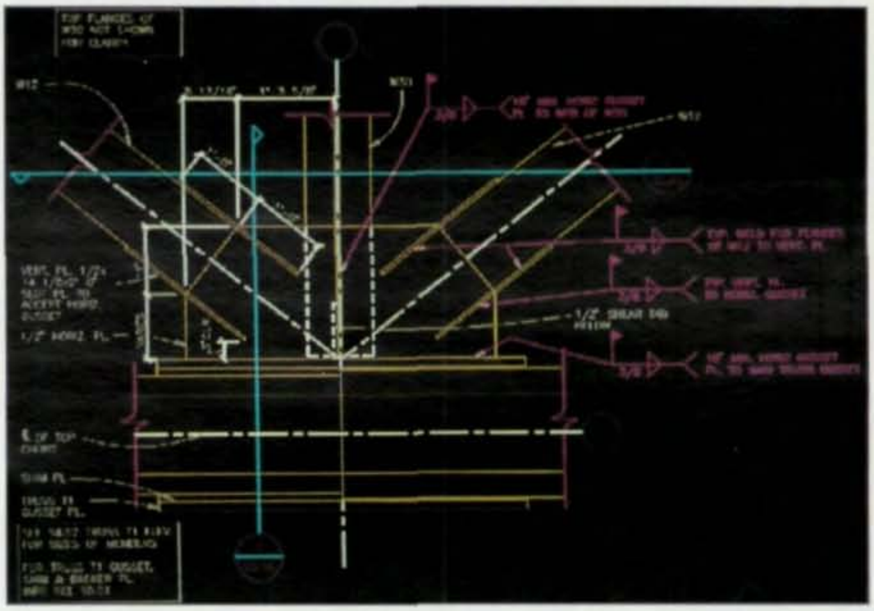
Extremely low fabrication tolerances were required to ensure the success of a 350-ft. long pedestrian bridge at the new Denver International Airport

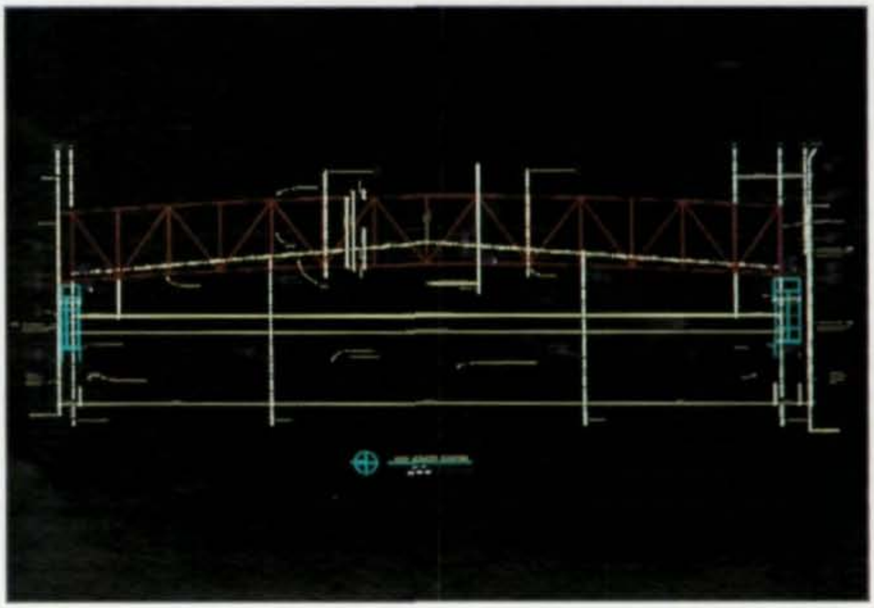
By Geoff Moses

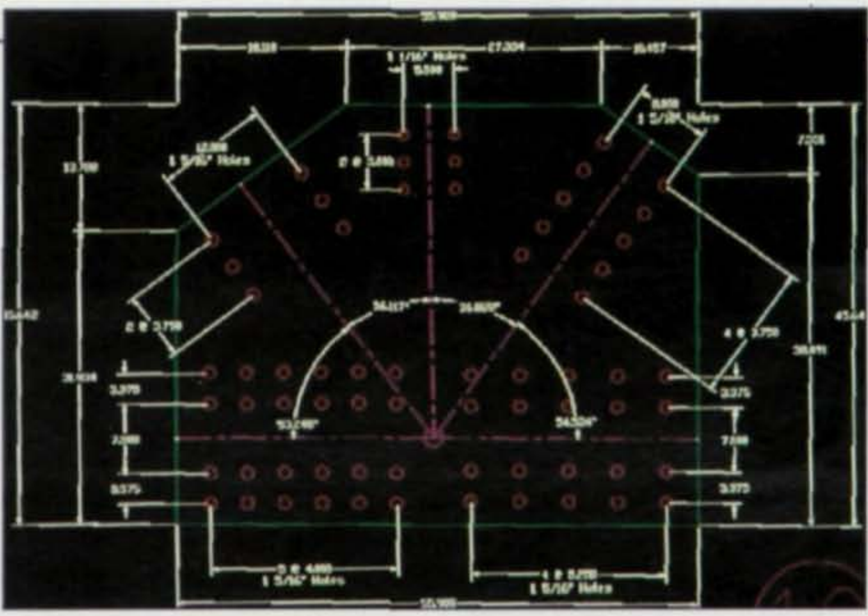
PSST! HEY, BUDDY! YA WANNA BUILD A 1,400-TON Swiss watch? Okay, that's not exactly how the conversation started between the structural engineers at LONCO, Inc. and the steel fabricators at AISC-member Zimmerman Metals. But it could have. Because the success of a Passenger Bridge at the new Denver International Airport depended on the same accuracy for which Swiss watches have long been known.

Engineering is all about solving problems and meeting challenges. Usually, the problems are as straightforward as "how do we get from here to there?" At the Denver Airport, the "here" and "there" were two buildings, designed by different architects and already under construction. The floor elevations did not match, and there was no plan for a connecting bridge, though linking tunnels were already constructed. Plus, the bridge had to pass over traffic consisting not of cars and trucks, but of large jets.

ECONOMIC AND POLITICAL COMPLEXITIES CAUSED constant changes in almost all phases of the design and construction of the Denver International Airport. The change that necessitated the pedestrian bridge was the decision that international arrivals would no longer be delivered to the U.S. customs facilities in the main terminal by planes taxiing directly to gates located there. In the revised configuration, the international gates were placed in Concourse A, where Continental Airlines has its domestic arrivals and departures. The problem was how to keep customs-bound international arrivals separate from the domestic traffic. The underground automated people-mover wasn't designed for the job and it, the concourse and the terminal were all already under







Shown is a typical gusset plate connection and a computer rendering of a gusset plate.

construction.

The task of bridging the conceptual chasm was undertaken by the structural department of LONCO in concert with the fabricators at Zimmerman and general contractor M.A. Mortenson, all of Denver.

"The geometry on this one was essentially dictated from the beginning," said Randy DeLancey, P.E., LONCO principal and project manager. The space envelope was restricted side-to-side by the configuration of the concourse and terminal, and by the need to avoid collapsing the tunnels during construction. Below, the structure had to clear two taxiways by a minimum of 44 ft. while ramping no more than 12% at any place in order to be accessible to carts and wheelchairs. "We looked at several alternative designs. A concrete Vierendeel truss looked promising, but its flat bottom wouldn't clear the taxiways unless it was raised, and then it would need stairs on at least one end. Also, it really had to much mass for seismic lateral load requirements. So steel appeared as the obvious choice. We considered several possibilities, including a cable-tensioned structure (a suspension bridge), but the project management team at D.I.A. wasn't wild about the idea of having the cables anchored inside their buildings."

Instead, the engineers started designing a simple, though enormously heavy, Warren truss structure supported by two massive cast-in-place concrete abutments 350 ft. apart—making it one of the longest free-span pedestrian bridges in the world.

In order to provide the required 44 ft. of vertical clearance, the truss incorporates a

The success of the project was largely dependent on the precision of fabrication. Pictured is the computerized milling of the shim plates and a view of the milled shim plates at the truss member ends.

parabolic curve that results in maximum floor slopes of 7% at the terminal complex and 8.3% at the concourse—a rise of 6 ft. from the former and 9 ft. from the latter—which meets ADA guidelines.

Because planes rarely pass under fixed structures, special training is required for the pilots who must accommodate narrowed wingtip tolerance. This unique feature drove many of the design considerations for the bridge. LONCO and architectural consultant Louis O. Acosta Architects preferred the drama of an external, exposed truss, but airport and FAA officials decided that accumulated ice and snow could present a hazard to aircraft passing below. Instead, the plans were redrawn with a smooth curtainwall of laminated glass on the outside to remove the threat of falling ice. The resultant glass-enclosed structure affords deplaning passengers with a dramatic view of the city and the Rocky Mountains all the way to the Continental Divide.

The structure itself is actually a tube truss comprised of four mutually stiffening trusses. The design began with a preliminary geometry survey and continued through the final phase of determining all movements of the interlocking steel. The design team spent long midnight hours becoming intimately acquainted with the vertical deflection characteristics of the structure under all anticipated loads. The resultant design handles thermal movement through large pot bearings, which rotate under gravity loads to maintain unified bearing pressure on compressible neoprene pads. Other forces are transmitted along the tube in a spiral pattern resembling a double helix, which provides exceptional stability. All told, the project required 1,400 tons of structural steel.

THE "SWISS WATCH" ASPECT OF THE PROJECT AROSE at the juncture between design and execution. The problem was in getting the more than 30 joints in each truss to line up smoothly over the 350-ft. length of the bridge given that the fabricator and erector were working with structural elements of widely varying web and flange widths—all of which may be warped up to $^1/_4$ in. out of alignment due to the steel mill process. Further, many of the members are jumbo shapes up to 42 ft. in length and 655 lbs. per linear ft, making them nearly the largest available from domestic producers.

The complexity of the connections was further increased by the designer's desire to minimize their size and weight by making the connections being bearing-type instead of slip critical. Each member had to be matched to its connecting members by means of gusset plates, backer bars and shims, and each piece had to be bored precisely to within $^1/_{16}$ in. to fit the bolts snugly. And finally, the whole thing had to be assembled in the air, an operation not unlike staging a Busby Berkely chorus number with tap dancing elephants.

It was in teaching the pachyderms their steps that the collaboration between engineer and fabricator became most apparent. J.P. Illes, P.E., senior design engineer on the project, credits longtime Denver fabricator and AISC-member George Zimmerman with achieving almost impossibly tight tolerances in the bolt holes at the truss connections and with a highly innovative solution for achieving the almost impossibly tight tolerances in the bolt holes at the truss connections and with a highly innovative solution for compensating for mill warp and size differential in the ends of individual structural elements.

USUALLY, THE MEMBERS, GUSSET PLATES AND BACKING BARS WOULD BE shop drilled with pilot holes and then field reamed to fit. The bolts would be forced through, where necessary, and loose shims would be used to wrestle the surfaces that don't quite fit into alignment. Zimmerman's idea, however, was to weld shim blanks onto the member's ends, mill them to standardize their widths, machine the backer bars to match exactly, and then bore each piece with standard size holes only $^1/_{16}$ in. larger than the

diameter of the bolts and with a horizontal tolerance of a mere three thousandths of an inch over 40 ft.

Zimmerman credits this remarkable accuracy to his shops very advanced CNC machining operation. "We had never done this before, and I'm not aware of anyone else who has," explained Zimmerman. "LONCO provided us with a precise and complete design, which enabled me to write the program for our computerized milling machinery on my home computer. Our biggest problem was that our working *drawings were more accurate* than what could be field verified (better than one hundredth of a ft.). We had to trust our design completely. We knew that the holes would line up to create the exact shape called for. The erector didn't know it. The contractor didn't know it. Even the designer didn't quite believe it. But we knew it."

The laborious process of boring more than 15,000 holes and making some 400 welds took roughly three months. Marc Langlee, Zimmerman's project manager, reported that their carbide bits drilled through the 2.5-in. plate in only 20 seconds. The welds, however, were another matter. Between 2 ft. and $4\frac{1}{2}$ ft. in length, they presented difficulties both in the surface area involved and in the amount of preheating required. "There's a lot of mass in the larger elements," said Langlee. "We had to constantly fight the 'heat-sink' *effect" of the sheer volume in the* big steel. After getting the elements bored and milled, it took two men a full week to assemble the first third of the truss (approximately 120 ft.). After leveling and lining up all the members, the bolts literally dropped into place.

Obtaining suitable bolts proved to be another difficulty. "You'd think that getting the bolts would be the simplest part," said Zimmerman. But there are approximately 6,000 $1\frac{1}{4}$-in. bolts in the bridge and their availability became so problematic that it threatened to shut down the project. Fortunately, Langlee and Mike

Breider, project superintendent for Mortenson, managed to locate enough bolts.

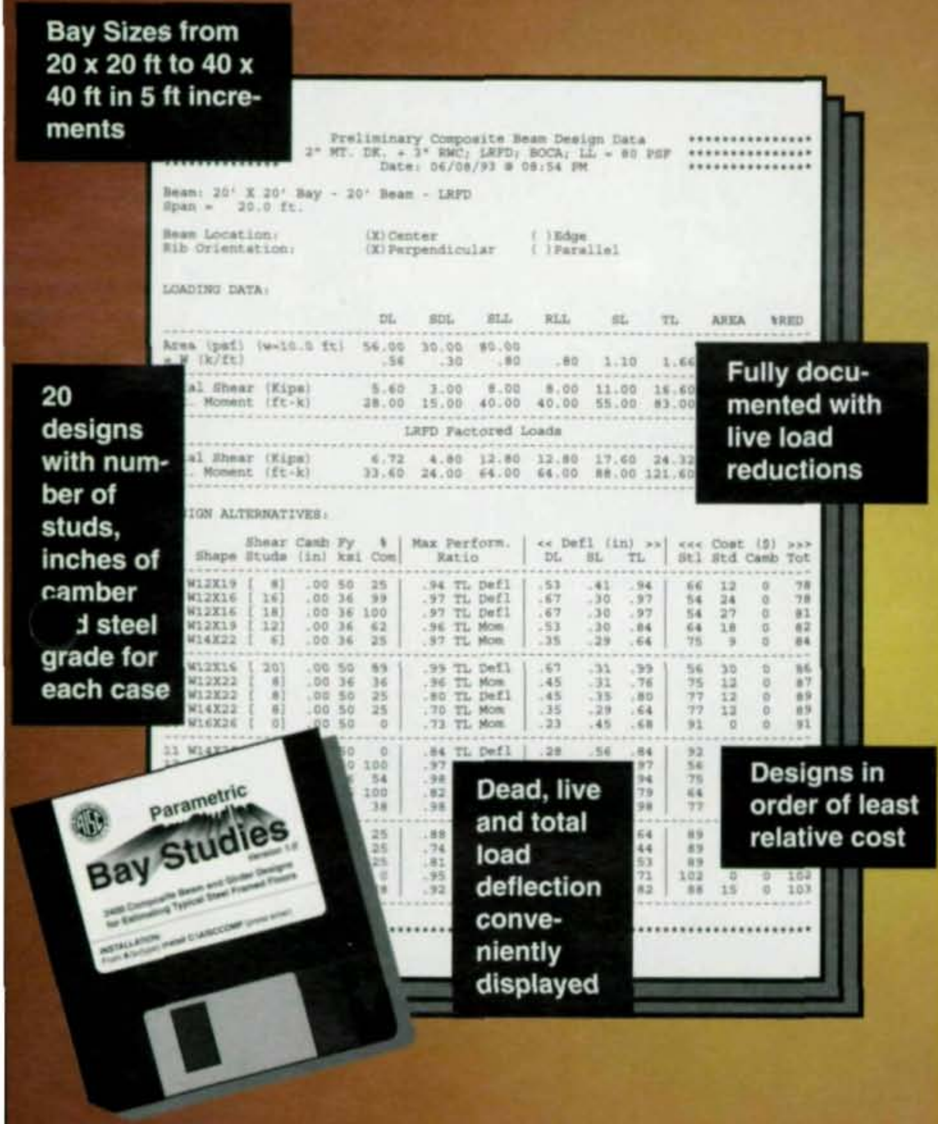
"We knew it would work. The erector (Derr and Gruenwald) was so surprised he wrote us a 'congratulations' letter," Langlee reported. The most important part was making sure everything was level. "The air-splice was tricky, and the arc and camber built into the truss complicated matching the elevations, but as long as the shoring was right, we knew it would work."

When the day arrived for the removal of the shoring, everyone was understandably a bit anxious. For two weeks prior, LONCO and Derr Gruenwald had been going back-and-forth refining their plan. "It was a difficult analysis, explained Dan Bechtold, structural analyst with LONCO. "It involved an iteration of changing support conditions. The analytical model developed from our first order analysis had to be modified to reflect the support of the shoring towers. Then we had to further modify it to reflect the difference in stiffness between the towers and the abutments. Then, we had to simulate the lowering of the towers." The engineer's recommendations were adapted to the erector's field conditions, defined largely by the dimensions in the shim stack. Quarter inch by quarter inch, the towers came down. Every effort was made to lower them simultaneously, so as not to overburden any one of them.

When the towers were clear of the span, a hasty field survey was made—and the results almost caused a panic. It appeared that the structure had come down two or three times as much as anticipated. A more formal re-survey, however, reassured the project team. The measurements were within fractions of an inch of their predicted deflection.

Geoff Moses is with LONCO, Inc., a Denver-based civil and structural engineering firm.

Accurate Steel Costs



Increased competition for projects is forcing engineers to more carefully consider the differences in constructed costs between steel and concrete structures. Furthermore, today's low cost steel bay designs result from the optimum balance of steel costs, shear stud costs and cambering costs. Thus the least steel-weight design may no longer be the least cost design.

To help engineers make accurate cost analyses, AISC Marketing has introduced a new design tool that is based upon today's cost information for steel and composite bay designs. The **Parametric Bay Studies** program is a database that provides relative cost information on more than 2,400 composite beam and girder designs on one IBM-compatible 3.5-inch diskette. The **Parametric Bay Studies** program allows the design engineer to quickly analyze a wide range of bay alternatives based on bay size, composite and non-composite construction, AISC specification (LRFD or ASD), model codes (BOCA, UBC, SBC), and deck profiles to determine the least cost configuration.

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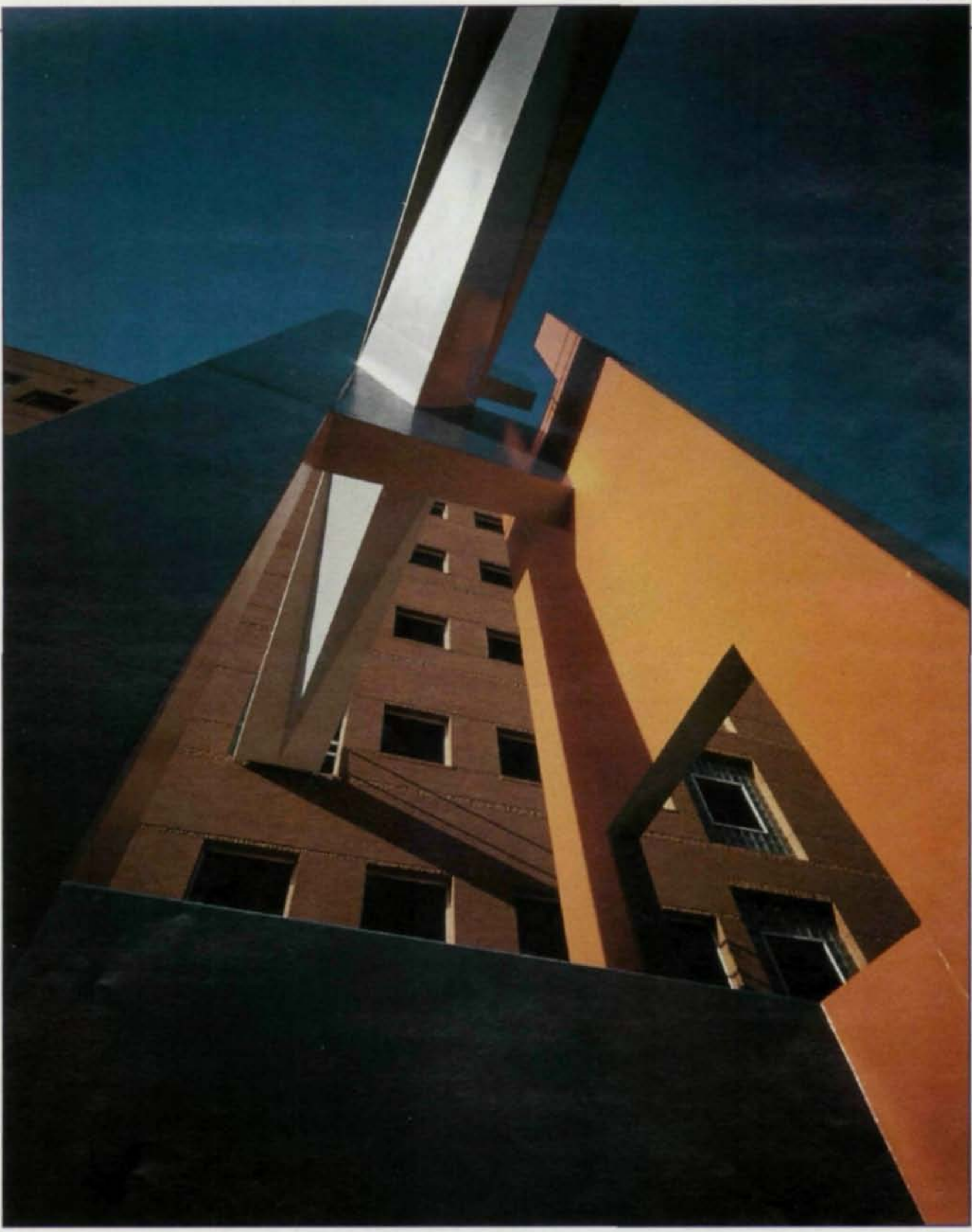


DIFFERENT PATHS

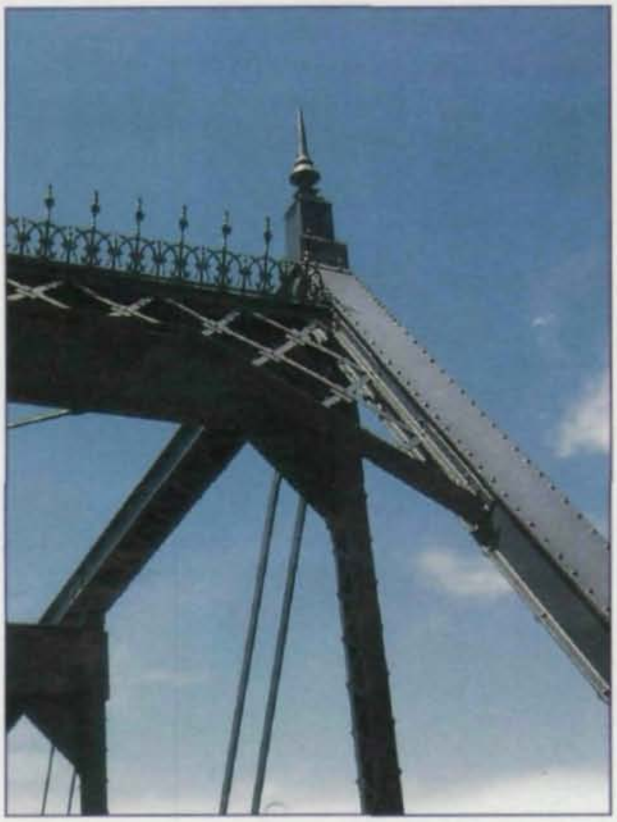
BECAUSE OF THEIR RAPID SPEEDS, passengers in cars rarely have the time to pay more than cursory attention to the many small bridges and overpasses they use constantly. As a result, designers are usually more concerned with a clean appearance than with intricate aesthetic details. One exception to this are pedestrian bridges, where strollers always have the time—and often the inclination—to admire even a small walkway as a potential piece of art.

HOWEVER, THE FORM THAT THIS ART can take varies greatly, from traditional to modern, ornate to simple, and everything in between.

Designer of the Life Sciences D-Wing Bridge at Arizona State University in Tempe was Anderson DeBartolo Pan, Inc., Phoenix and general contractor was McCarthy, also of Phoenix. An elaborate, artistic ramping system was used to bridge the non-aligned levels of the two structures while maintaining wheelchair access.

AT ONE EXTREME IS THE LIFE SCIENCES D-WING BRIDGE at Arizona State University in Tempe. Its purpose was to connect the new and existing Life Sciences Buildings at two separate levels. Complicating the design was that the buildings' floors did not align—and stairs were not permissible to bridge the height discrepancy due to ADA requirements.

The designer's sculptural solu-



Designer of the renovation of the 19th Street Truss Bridge was LONCO Engineers, Inc., Denver, and gneeral contractor was Edward Kramer & Sons, Plain, WI.The lead paint removal was accomplished by L & M Enterprises, Berthoud, CO, which also was the steel erector on the project.

tion was conceived as symbolizing a scale of balance, with the arms of the scale at different levels to accommodate the non-aligning floors with a sloping ramp.

Because the walkway was constructed on a plaza sitting above a double basement, weight was a crucial concern and made the decision to build in steel an easy one. The vertical element was constructed of steel box sections for structural rigidity over its more than 64-ft. height. The horizontal component is a cantilevered steel plate bridge system. A colorist was employed to choose the paint scheme, which featured multiple colors to reinforce the bridge's image as a piece of sculpture and not just utilitarian connection.

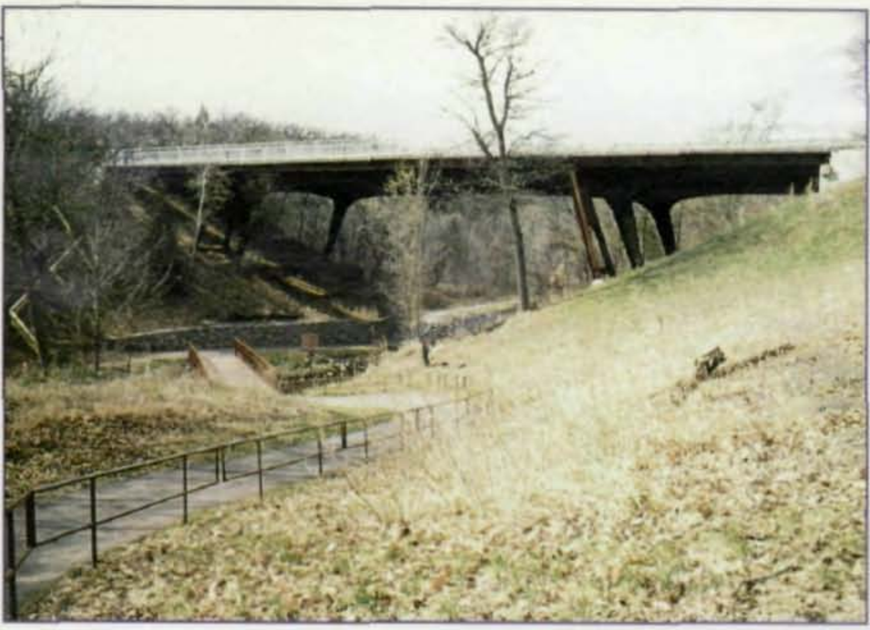
AT THE OPPOSITE END OF THE SPECTRUM is the Historic 19th Street Truss Bridge across the Platte River in Denver. This pin-connected iron Pratt through-truss dates back to 1887, when it was constructed as a major transportation artery for the nearby towns that grew during the Gold Rush of 1858. A century later, it was taken out of service but preservationists fought its demolition. The solution was to realign 19th street adjacent to the historic bridge to accommodate a new structure, while rehabilitating the old bridge for use with the bicycle/pedestrian paths incorporated into the city's Greenway plan.

The new bridge is a low-profile structure that defers to the ornamented truss and uses stone veneer facing to pay tribute to the massive, solid stone ashlar abutments, pier and stepped-stone wingwalls of the original.

Key to the renovation was the removal of its lead-based paint. After considering several more traditional methods, a closed-abrasion, vacuum-blast operation was chosen. In this process, steel shot is propelled against the surface of the metal to be stripped by a head held directly against it. The head also provides suction to collect the small bits of paint dislodged. The steel shot is recycled until it becomes small enough to be collected by the vacuum system. This closed system protects against flying toxins without the need for encapsulating the bridge and the workers. Instead, a small draping at the bottom of the bridge is all that is necessary to protect the river.

Further work on the bridge included replacement of lost or vandalized portions of the bridge's ornamental iron.

OF COURSE, NOT ALL BRIDGES NEED TO MAKE A dramatic splash. Sometimes, the exact opposite



Designer of the 106th Street Pedestrian bridge in Bloomington, MN, was Barr Engineering Co., Minneapolis, and general contractor was Carl Bolander & Sons Co., St. Paul.

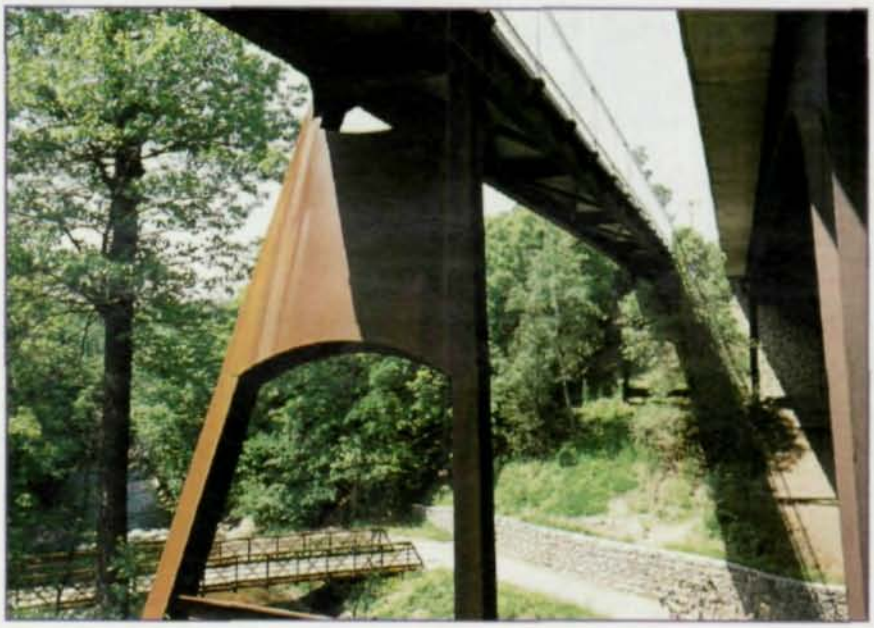
effect is desired. For example, the 106th Street/Nine Mile Creek Pedestrian Bridge in Bloomington, MN, was designed to minimize distraction from an adjacent award-winning roadway bridge.

The pedestrian bridge is part of a larger project to restore and protect the steep valley walls in the area, protect the plant and wildlife habitat and provide an improved access for recreational use along the lower Nine Mile Creek. The new bridge is part of a two-and-a-half-mile-long trail system that includes seven other pedestrian bridges and provides stair-free access throughout an urban park.

The primary design consideration was to minimize disruption to the valley floor and side walls, while creating an aesthetically pleasing bridge that would blend in with both the natural setting and the adjacent roadway bridge. A three-span weathering steel bridge was selected for both aesthetic and cost reasons. The adjacent 20-year-old-plus roadway bridge also is constructed of weathering steel and has had excellent performance.

For the tall (50-ft. high), narrow (8-ft. wide) bridge structure, the project's designers chose inclined bents to resist a large portion of the wind loading.

The bent lateral design includes a bracing system with a steel cover plate and internal diagonal members. The system provides a clean, uncluttered appearance and reduces the potential hazards of people climbing on the structure. In order to eliminate foundation-pile uplift while matching the narrow deck width, the bent legs are battered in addition to being inclined. To simplify field fabrication and erection, the three-dimensional connection between the beams and the bent was accomplished with a bolted connection to curved gusset plates.



WHILE THE APPEARANCE OF PEDESTRIAN BRIDGES is very important, equally important are the views from the bridge. The recently completed Federal Office Building in Oakland features twin 12-story towers rising from a five-story base. And connecting the two towers at the 13th and 14th floors is a pedestrian bridge that provides spectacular views of the surrounding city and the San Francisco Bay.

The bridge is 11-ft.-6-in. wide and spans 85 ft. between the towers. The choice of steel for a framing system was based on a need for a lightweight framing system that would be easy to erect with all of the skeletal steel of the towers in place and that would meet extreme displace-

Design firm for the pedestrian bridges at the Oakland Federal Building was Cygna Consulting Engineers, Oakland, and general contractor was Walsh Construction Co., Sacramento. Steel fabricator and erector was AISC-member Herrick Corp.

ment requirements under seismic and wind conditions. After careful consideration, it was decided to design the bridge to act as a simple span between the two towers with transverse lateral loads resisted at both towers but longitudinal loads resisted only at the south tower.

Two W36 x 230 girders were selected to act as simple span beams between the towers. Full penetration shop welding was used to splice the girder and provide 85-ft. continuous pieces. For the bridge floors, a 3-in. metal deck with 3-1/4-in. lightweight concrete fill spanning between the girders was selected to eliminate the need for form work and to reduce the structures weight. Once the girders are in place the concrete deck stiffens the floor system; however, to provide rigidity to the floor deck system during erection a horizontal truss was created by adding diagonal members between the girders.

The connections to the towers were tricky, since during an earthquake the two towers would experience deflection that are not synchronized. The design had to allow for the worst case deflection scenario, i.e., the two towers moving in opposite directions. Lateral motion and inward or outward motion had to be accommodated simultaneously. After considering several concepts, end connection details were developed based upon a combustion engine's crankshaft, rod, piston head and cylinder assembly.

In a combustion engine, a rod is pinned to a crankshaft at one end and pinned to a piston head at the other to allow for the lateral movement of the crankshaft. This design considers the south tower as the crankshaft, the north tower as the cylinder and the bridge as the rod and piston. The two pins, one at each tower, accommodate the lateral motion of the towers. In an engine, the piston head is free to move in a lubricated cylinder in an in-out motion. Similarly, the pinned end at the south tower slides in a guide channel and allows the in-out motion of the bridge.

WHILE THE OAKLAND BRIDGE HAD A DUAL AESTHETIC PURPOSE, a bridge in Arkansas had a more practical dual purpose. When Arkla Energy Resources began construction of 222 miles of natural gas pipeline from Wiburton, OK, to Glendale, AR, the company discovered that the optimum routing crossed the Cossatot River in Arkansas in an area designated as a "natural area" by the State of Arkansas. The wild and scenic river also is a prime habitat for an endangered fish species, the Leopard Darter. As a result, a conventional trenched crossing was not feasible—and rerouting the pipeline to avoid this area would have cost an additional $30 million.

An economic alternative was an aerial crossing using a three-span, continuous box steel plate girder bridge. The bridge would not only provide a means for the pipeline crossing but would also provide a pedestrian bridge for access to park facilities on both sides of the river—while eliminating the hazard of pedestrian use of a nearby highway bridge.

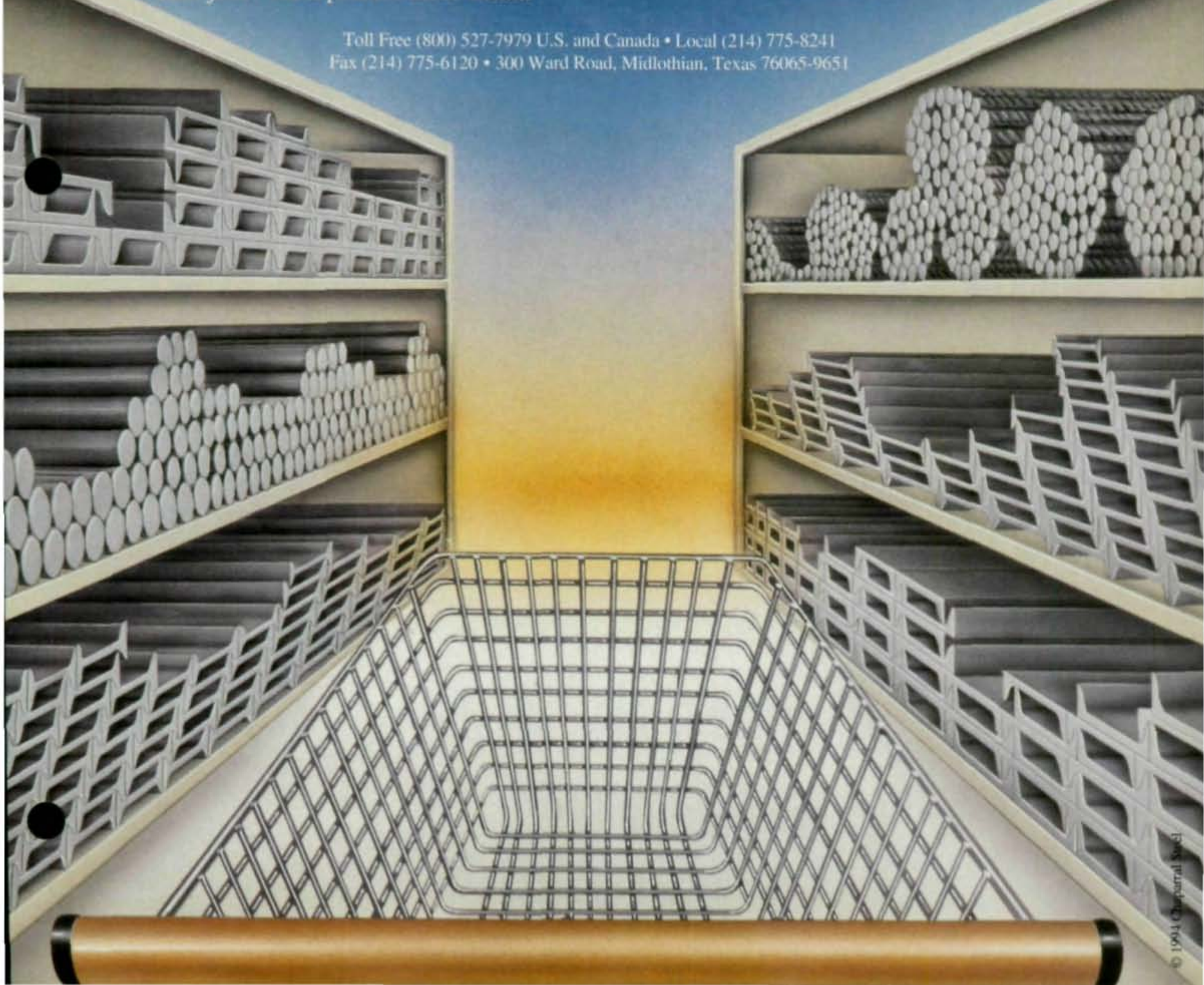
The pipe bridge's design needed to meet the aesthetic requirements of a number of permitting agencies. The design elements included: nearly matching the vertical projection of the adjacent highway bridge; painting the bridge to blend with the existing surroundings as well as

Designer of this combination pipeline/pedestrian bridge was Willbros Butler Engineers, Inc., Tulsa, and general contractor was Michael Curran and Associates, Houston. Fabricator was AISC-Member Nesom's Capitol Steel & Iron Co. and erector was Dick Mooney, Inc.

to match the adjacent bridge; completely enclosing the pipe and hiding it out of sight; and providing complete handicap accessibility. In addition, additional room was provided to accommodate future utility crossings.

To minimize erection time and meet environmental criteria, the bridge was designed to be fabricated and shop painted in four sections. The bridge spans 395 ft. with three spans of 155 ft., 165 ft. and 75 ft. The box section is 9-ft. wide and 7.33-ft. deep. The top deck is 1/4-in. plate stiffened transverse to the longitudinal axis of the bridge. The bottom deck, which is a 1/4-in. plate stiffened parallel to the longitudinal axis of the bridge, is raised one foot above the girder flanges to clear the horizontal bracing and diaphragms. The four sections are 86, 89, 125 and 95 ft. long. The end span sections have cantilevers of 20 ft. with a W18 x 105 beam bolted to the bottom flange to facilitate erection of the 125-ft. set-in-section.

The two sections forming the 155-ft. span and 20-ft. cantilever were designed with full penetration welds of the flanges and web. The intent was to weld the splice with the sections on blocks on the road, then lift the two sections as one; however, the contractor elected to build a false bent and make two lifts and weld the splice in the air.



Company: **AMERICAN PUNCH CO.**
Address: 685 S. Green Road
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Phone: 800/243-1492
Information: The company offers a wide range of low-cost punches, dies, and shear blades for angleline, beamline and portable punches. In addition, the company offers a variety of shear blades, coupling nuts and punch stems.

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Address: 301 Chestnut St.
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Phone: 800/628-3030
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Information: Lenox Band-Ade, a high performance, biodegradable sawing fluid has been reformulated to provide extended shelf and sump life to help solve the problems of souring and organic breakdown. The fluid is an advanced synthetic emulsion with lubricating properties that allow for faster cutting and extended blade life.

Company: **THE CLEVELAND STEEL TOOL CO.**
Address: 474 East 105th St.
Cleveland, OH 44108
Phone: 800/446-4402
Information: The company has published a new four-color catalog. It details products available both in stock and by special order. Included are punches; dies; and tooling for a wide variety of manufacturers.

Company: **COMEQ, INC.**
Address: Box 2193
Baltimore, MD 21203
Phone: 301/325-7900
Fax: 301/325-1025
Information: The ADIRA line of Americor hydraulic shears includes as standard equipment: ergonomic design; 25-in. or 40-in. power front-operated back gauge with manual fine-tuning adjustment; gap in frame for slitting longer plates; rapid blade gap adjustment; short stroke capability to increase operating speed; and short squaring arm and blades suitable for shearing both mild steel and stainless steel. A wide variety of sizes are available.

Company: **COMPUSTEP**
Address: 774 Rye St., Unit 11
Peterborough, Ontario, CANADA K9J 6W9
Phone: 705/745-2961
Fax: 705/745-8130
Information: The company offers a six-page color brochure describing its Quickdrill Stretch family of large area drilling/tapping machines. More than just a product brochure, it highlights areas for productivity improvement and drilling alternatives. In addition to describing the company's product line, the brochure provides information on CNC programming and advice on productivity improvements.

Company: **GRIPHOIST INC.**
Address: 392 University Ave.
Westwood, MA 02090-0005
Phone: 800/421-0246
Information: A new newsletter contains new product information as well as a listing of available literature from this manufacturer of hoists, fall protection products, scaffolding and load indicators. New this year is a stirrup adapter that allows manual hoists to be used on a powered scaffold stirrup.

Company: **HARRINGTON HOISTS**
Address: 401 West End Ave.,
Manheim, PA 17545
Phone: 717/665-2000
Fax: 717/665-2861
Information: "How to Get the Most Out of Your Hoist" is the title of a new, illustrative brochure from this leading manufacturer of hoists and cranes. The free,12-page brochure gives detailed advice on how to select, install, inspect, maintain, and safely operate a manual or electric chain hoist.

Company: **HILMAN, INC.**
Address: 2604 Atlantic Ave.
Wall, NJ 07719
Phone: 908/449-9296
Fax: 908/223-8072
Information: A new, six-page product catalog is now available from this manufacturer of high capacity machinery roller dollies. The catalog includes photos, full descriptions and charts of technical and dimensional data. Hilman's chain action rollers are designed to move heavy machinery and equipment from 500 lbs. to well over 1,000 tons. Also available is a series of four-page, four-color brochures featuring applications of Hilman rollers.

Company: **HILTI INC.**
Address: 5400 South 122nd East Ave.
Tulsa, OK 74146
Phone: 800/879-8000
Information: Recently introduced is the TE 74 Combihammer, which is designed for medium to heavy duty drilling and chipping. Weighing only 18 lbs., the combihammer features an electronic variable speed switch and two-level hammering power regulation for full operator control and precise chiseling. The powerful and versatile tool has hammering speeds of 2,700 and 1,960 blows per minute and single impact energy of 5.2 and 2.6 foot pounds. Additionally, it features a service indicator to alert operators to when the tool needs servicing to help avoid unexpected downtime. Also available is a new line of drill bits.

Company: **HOUGEN MFG. INC.**
Address: G-5072 Corunna Road
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Phone: 810/732-5840
Fax: 810/732-3553
Information: Rotabroach portable magnetic drills are ideal for bridge repair and other projects where on-site hole making is required. Models 10904 and 10908 offer the portability of a hand-held drill and the stability of a drill press. A small ($^{3}/_{18}$ in. x $6^{7}/_{16}$ in.) but powerful magnet rated at 1,800 lb. dead lift and an efficient 115 volt motor design combine to provide a hole production range of $^{7}/_{16}$-in. to $1^{3}/_{8}$-in. diameters in materials up to 2-in. thick. Tests show that a 1-in.-diameter hole through 1-in.-thick mild steel can be drilled in less than 40 seconds.

Company: **INSCO SAW DIVISION**
Address: 320 International Circle
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Phone: 803/873-7850
Information: A wide variety of segmental saw blades, bandsaw blades and an ICAS Software Program for high-speed cutting are available. ICAS selects the proper blade, correct feeds and speeds to maximize beam cutting efficiency.

Company: **JANCY ENGINEERING CO.**
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Davenport, IA 52804
Phone: 319/391-1300
Fax: 319/391-2323
Information: Company manufacturers a variety of products under the Slugger name including—
•Industrial belt grinders designed for continuous heavy duty grinding operations. These new grinders allow the operator to safely hold material and grind a part accurately without vibration.
•Pipe notching machines for inexpensive and productive notching of all types of pipe and tubing.
•Portable drilling machine weighing 24 lbs. with a compact 16-in. overall height. It's capable of drilling $1^{1}/_{16}$-in.-diameter holes in 2-in.-thick carbon steel.
•Portable magnetic bases drilling system weighing 42 lbs. and capable of producing $2^{1}/_{16}$-in.-diameter holes in 2-in.-thick steel.

Company: **KALTENBACH, INC.**
Address: 6775 Inwood Dr.
Columbus, IN 47201
Phone: 812/342-4471
Information: The HDM-1000 and HDM-1400 structural saws are designed for use in a "tandem system" with the company's structural CNC drill. Unique design includes a traveling saw arm and fixed datum fence to include miter cutting, optifeed, and vertical clamp.

Company: **MI-JACK**
Address: 3111 East 167th St.
Hazelcrest, IL 60429
Phone: 708/596-5200
Information: The company's TRAVELIFT Crane is a self propelled rubber tired gantry crane able to pick and carry its load anywhere. Capacities range from 15 to 325 tons and dimensions are customized to each application. Existing storage can be increased up to 500%; independently controlled hooks combined with optimal spreader bar eliminate product damage; and lifting capacity can be doubled with a two-magnet system.

Company: **NATIONAL CRANE**
Address: 11200 North 148th St.
Waverly, NE 68462
Phone: 402/786-6300
Fax: 402/786-6379
Information: Newly introduced is the Series 1100 telescoping crane with a rated capacity of 56,000 lbs. and a vertical reach of 104 ft. under hydraulic power. It features state-of-the-art Anti-Two-Block and Load Moment Indicator systems, along with behind-the-cab, stand-up dual operator controls.

Company: **PANGBORN CORP.**
Address: P.O. Box 380
Hagerstown, MD 21741-0380
Phone: 301/739-3500
Information: The company offers a wide range of blast cleaning systems for descaling plate, rolled shapes, weldments and fabrications, for both pre- and post-blast descaling. Scale models and visual aids will be used to depict a wide variety of blast cleaning systems used in surface preparation of materials for burning, welding and applications of coatings.

Company: **PEDDINGHAUS CORP.**
Address: 300 N. Washington Ave.
Bradley, IL 60915
Phone: 815/937-3800
Information: Newly introduced this year is a new line of horizontal bandsaws designed specifically for processing structural steel sections. These saws feature a twin column guidance system that totally encloses both sides of the cutting head to achieve the maximum rigidity when cutting structural steel sections. This guillotine approach includes angling the blade at a fixed six degree angle relative to the material to reduce the cutting load as the blade traverses through the web of steel beams and channel. Wide flange sections up to W36x328 and column sections up to W14x730 can be processed efficiently.

Company: **PULLMAX INC.**
Address: 1201 Lunt Ave.
Elk Grove Village, IL 60007

Phone: 800/255-0502
Fax: 708/228-5650
Information: The company's X-93 production plate beveler helps fabricators achieve high-quality weld preparation. It bevels any angle between 25 and 55 degrees and is up to 12 times faster than flame cutting steel plate up to 2-in. thick. The plate beveler produces consistent edges without heat effect and distortion and is safer than flame cutting.

Company: SHEPARD NILES
Address: N. Genessee St.
Montour Falls, NY 14865
Phone: 607/535-7111
Fax: 607/535-7323
Information: The company designs, manufactures and installs complete material handling systems designed to meet customized needs. Systems include automated/integrated monorail systems, overhead cranes, hoists, steel rod/coil handling systems, and AS/RS machines to service teleplatforms.

Company: SHUTTLELIFT
Address: 49 E. Yew St.
Sturgeon Bay, WI 54235
Phone: 414/743-8650
Fax: 414/743-1522
Information: This company manufacturers a large line of versatile industrial mobile hoists. Available in 14 standard models with capacities from 15 to 400 tons, models are available with either single or double ISL traversing hooks or SL fixed (four to six) hooks. Newly introduced are the Carrydeck 3330-E and 333-ELB mobile cranes with capacities from 15 to 500 tons.

Company: SLINGMAX INTERNATIONAL
Address: P.O. Box 2423
Aston, PA 19014-2423
Phone: 610/485-8500
Fax: 610/494-5835
Information: Twin Path slings have similar lifting characteristics to wire rope but weigh less and therefore require fewer riggers to handle them. On a recent project requiring a maximum lift of 755 tons, the synthetic slings weighed 25,000 lbs. less than the equivalent steel wire rope slings.

Company: TRAMAC
Address: 7 Emery Ave.
Randolph, NJ 07869
Phone: 800/526-3837
Fax: 201/362-5128
Information: The company's new PFH Demolition Shear is a combination tool that does not require changing of accessories in order to adapt to different materials—including

steel, wood or reinforced concrete. The tool offers distortion-free jaws, mechanical swivel, trunnion mounted power cylinder, and reversible cutting blades. In addition, unlike many conventional shears, the PFH series does not require the removal of the excavator's dipstick.

Company: TRUMPF POWER TOOLS
Address: Farmington Industrial Park
Farmington, CT 06032
Phone: 203/677-9741
Fax: 203/678-1704
Information: Newly introduced is the N1000 two-speed, lightweight nibbler with the capacity to cut thick sheets and nibble over 90 degree bends. The pneumatic model weighs only 30 lbs., while the electric model weighs 34 lbs. The portable tools are easily adjustable with minimum set-up times and fast tool change.

Company: W.A. WHITNEY CO.
Address: 650 Race St.
Rockford, IL 61105
Phone: 815/964-6771
Fax: 815/964-3175
Information: The company has introduced several new products this year, including the TRUECut Oxy-Plasma System. The system uses a new oxygen plasma cutting process that allows for a much longer consumable life than previously available air or oxygen plasma cutting processes. In addition, clean-up is reduced or eliminated since the process produces clean parts with little or no dross. The 3700 ATC//TRUECut combination offers contoured feedrates 50% to 100% faster than laser systems, lower operating costs, a 40-ton hydraulic punching capability with high hole size capacity and the ability to form countersinks, louvres, number stamps and countermarks, and high quality finished parts.

Company: VERNON TOOL CO.
Address: 503 Jones Road
Oceanside, CA 92054
Phone: 619/433-5860
Fax: 619/757-2233
Information: A new two-axis CNC pipe cutting control provides computer precision at an affordable price. Compared with manual methods, the new MPM-2 delivers increased cutting speed and enhanced fit-up accuracy with lower labor costs. The versatile machine can provide a wide variety of shapes, including: straight and beveled cuts; miters; T, K, and Y saddles; multiple intersections; centerline offsets; and circular and regular holes.

Help Wanted

Progressive and rapidly expanding Structural Steel Fabricator in Midwest is seeking a Structural Engineer. Candidate must have a minimum of five year's experience as a Registered Professional Engineer with a proficiency in the design of steel structures and be familiar with the AISC Code of Standard Practice. CAD experience a plus. Company offers an excellent salary and benefit package. Please submit your resume with salary history and requirements to: **Construction Manager, P.O. Box 1066, Salina, KS 67402-1066**.

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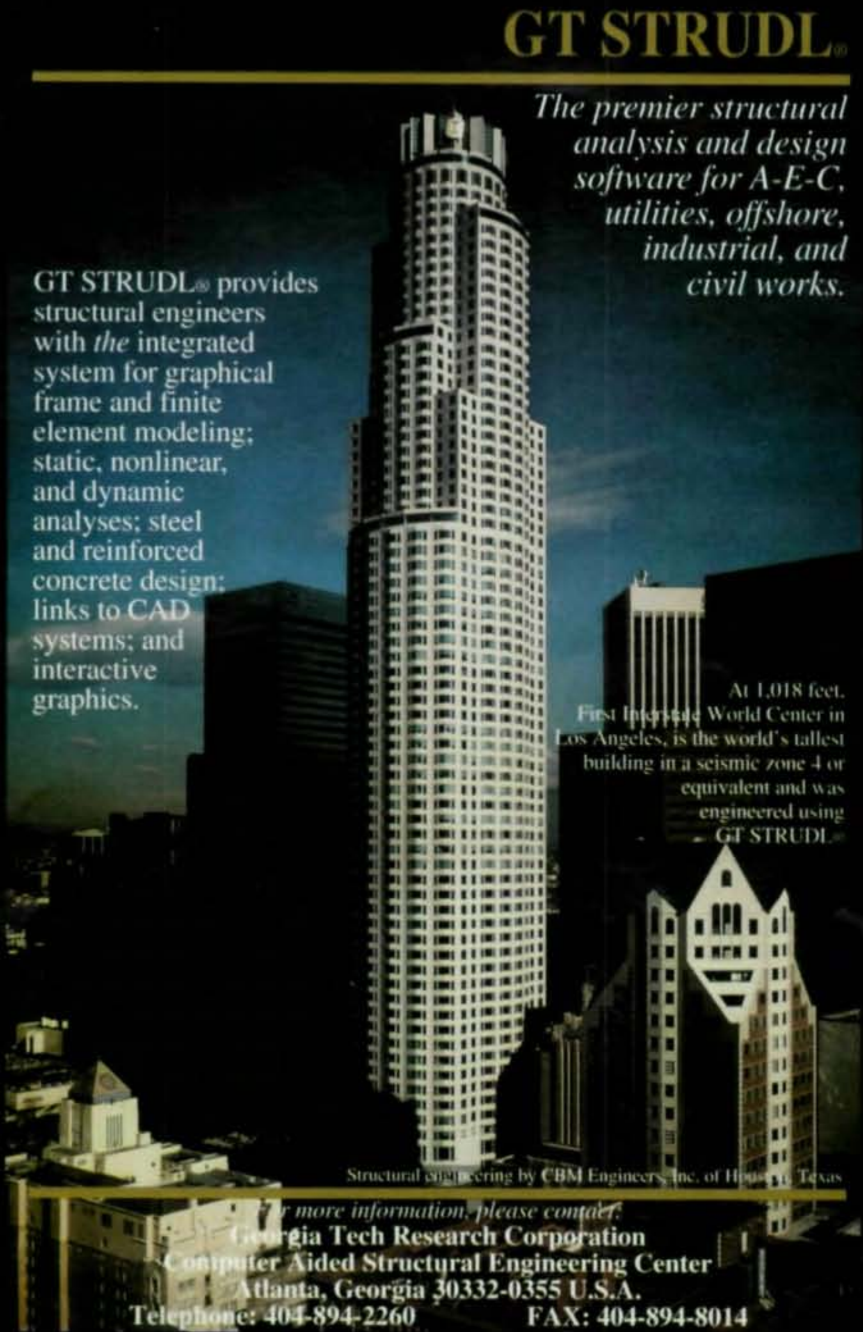
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