MODERN STEEL CONSTRUCTION

July 1992

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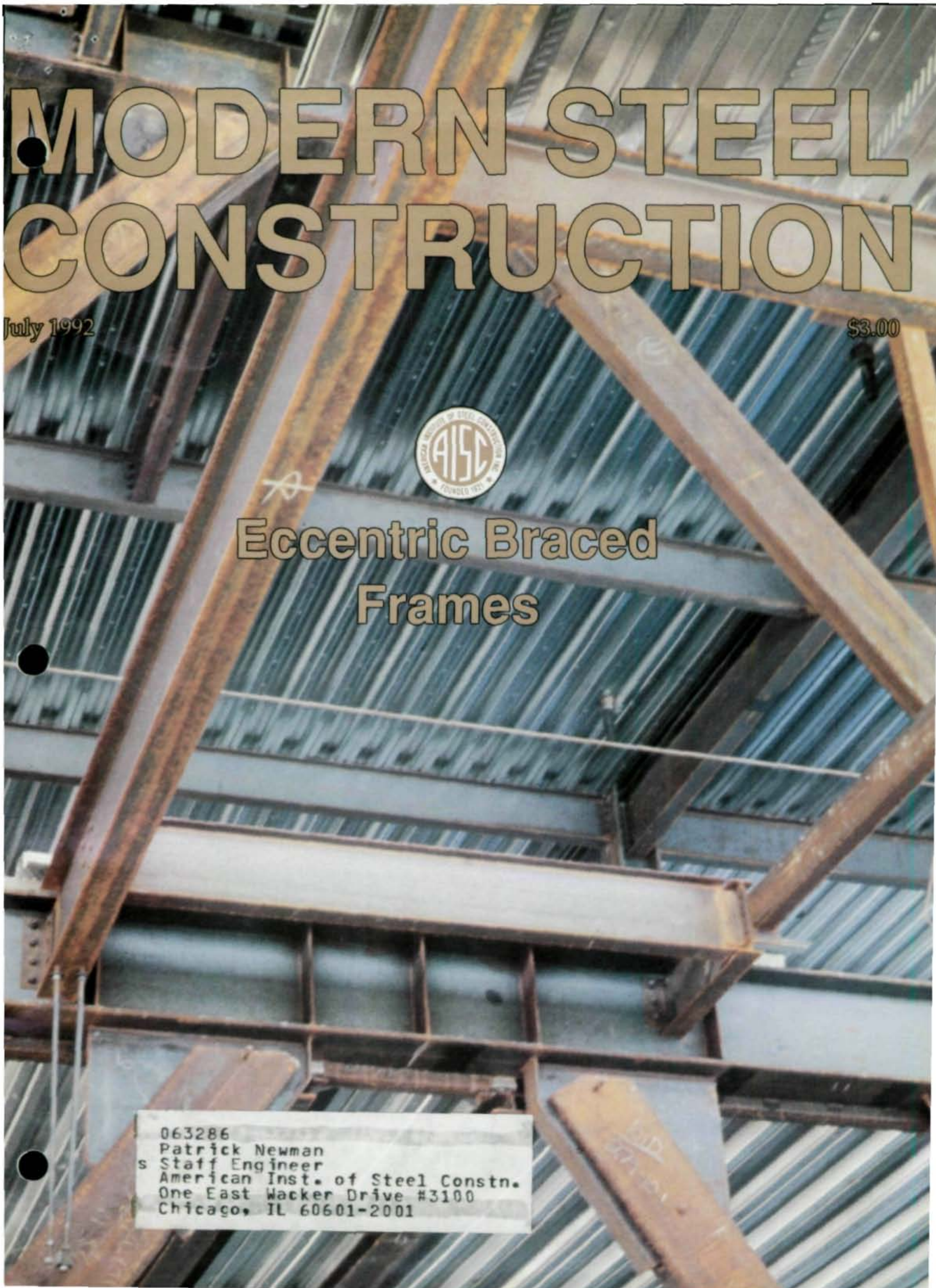


Eccentric Braced Frames

UNITED STEEL DECK, INC.

ROOF DECK DATA BASE

DECK DESIGN DATA SHEET

No. 17

Attribute	Type B Deck (B, BI, BA, BIA)				Type F Deck			Type N Deck (NS, NI, NSA, NIA)			
gage	22	20	18	16	22	20	18	22	20	18	16
thickness	.0295	.0358	.0474	.0598	.0295	.0358	.0474	.0295	.0358	.0474	.0598
weight, psf	1.7	2.1	2.8	3.5	1.6	2.0	2.6	2.1	2.5	3.3	4.1
I_p, in.^{4} (1)	0.17	0.24	0.31	0.40	0.13	0.17	0.24	0.64	0.82	1.19	1.62
I_n, in.^{4}	0.20	0.24	0.32	0.40	0.15	0.19	0.25	0.85	1.04	1.38	1.75
S_p, in.^{3}	0.19	0.25	0.34	0.44	0.13	0.16	0.22	0.37	0.49	0.68	0.88
S_n, in.^{3}	0.20	0.26	0.36	0.45	0.14	0.17	0.23	0.42	0.54	0.74	0.93
Ext.R(2), lbs.	450	620	1010	1860	440	610	1000	320	450	760	1410
Ext.R(3), lbs.	540	730	1160	2100	540	720	1140	390	530	870	1590
Int.R(4), lbs.	1270	1830	3120	4670	1250	1800	3070	940	1370	2370	3800
Int.R(5), lbs.	1320	1880	3200	4750	1320	1880	3190	1090	1580	2700	4020
V(6), lbs.	1920	2300	3000	3780	1970	2360	3120	2350	3390	4960	6180
Max.1 span(7)	5'10"	6'8"	8'0"	9'1"	5'2"	5'11"	7'0"	11'5"	13'0"	15'8"	18'3"
Max.2 span(8)	6'11"	7'10"	9'5"	10'9"	6"1"	7'0"	8'4"	13'5"	15'3"	18'5"	21'6"
Max. Cant.(9)	1'11"	2'4"	2'10"	3'3"	1'2"	1'5"	1'10"	3'6"	4'0"	4'10"	5'5"
FMS span(10)	6'0"	6'6"	7'5"		4"11"	5'5"	6'3"				

NOTES

(1) I_p, I_n, S_p, and S_n are the section properties per foot of width. These values were calculated using the American Iron and Steel Institute Specifications. The subscripts denote positive or negative bending.

(2) Allowable end reaction per foot of deck width -- 2" bearing.

(3) Allowable end reaction per foot of deck width -- 3" bearing.

(4) Allowable interior reaction per foot of deck width -- 4" bearing.

(5) Allowable interior reaction per foot of deck width -- 5" bearing.

(6) Allowable vertical shear per foot of width -- do not confuse this with horizontal shear strength provided by the diaphragm.

(7) Maximum span recommended for roof construction based on SDI criteria -- single span.

(8) Maximum span recommended for roof construction based on SDI criteria -- 2 or more spans.

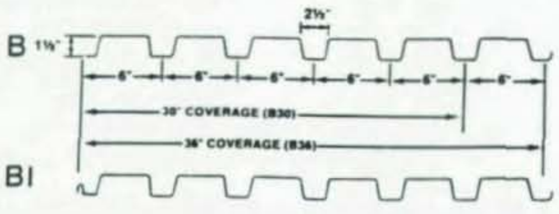
(9) Maximum recommended cantilever span based on SDI criteria; these spans are sensitive to the length of the adjacent span as they are controlled by deflection. Call if you need a more precise calculation.

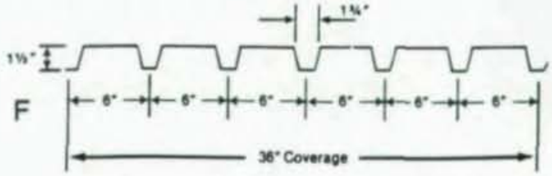
(10) Maximum spans for Factory Mutual Class I construction. Factory Mutual will allow these spans to be extended by 10% (maximum) when the insulation is mechanically fastened to the deck by screws and plates. Whenever this extension is used, sidelap fastening must occur at 18" (maximum) rather than the normal 36". Refer to the Factory Mutual System Approval Guide.

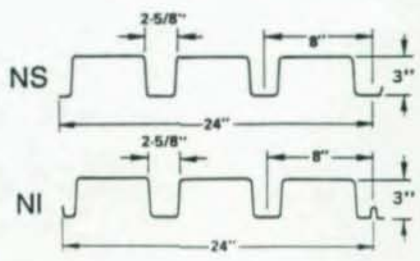
(12) B is generically known as "wide rib" deck; F is "intermediate" rib, and the 3" deep N deck is "deep rib".

(13) The deck type B means flat side lap; BI is the "interlocking" side lap; BA and BIA means the decks are acoustic. F deck is only available with the flat sidelap. NS is flat sidelap; NI is "interlocking" and NSA and NIA are acoustic decks. Better sidelap connections are obtained by screwing or welding through the flat sidelaps and therefore this is the recommended type.

(14) Information not provided on this chart may be obtained by calling our office in Summit, NJ.







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MODERN STEEL CONSTRUCTION

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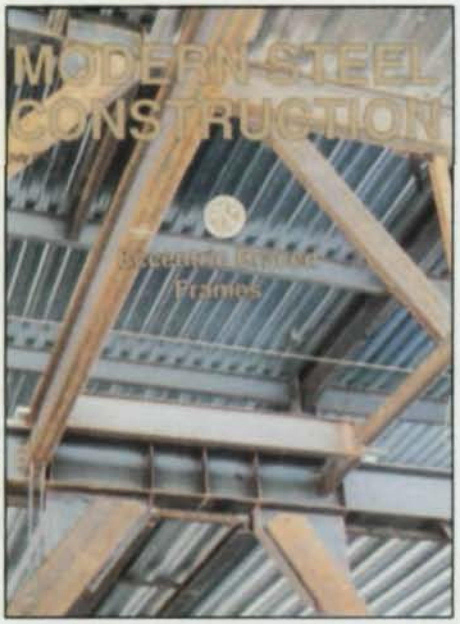

The use of eccentric braced frames and the 1990 "Seismic Provisions for Structural Steel Buildings—Load and Resistance Factor Design" allowed the designers of a research facility in Boston to accommodate the desired bay size and ceiling heights in a seismic zone.

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You Say Tomato...

On the East Coast it's an inverted "V", in the Midwest it's a "K", and on the West Coast it's a Chevron. But regardless of the terminology, engineers throughout the country have long depended on braced frames for stiffness.

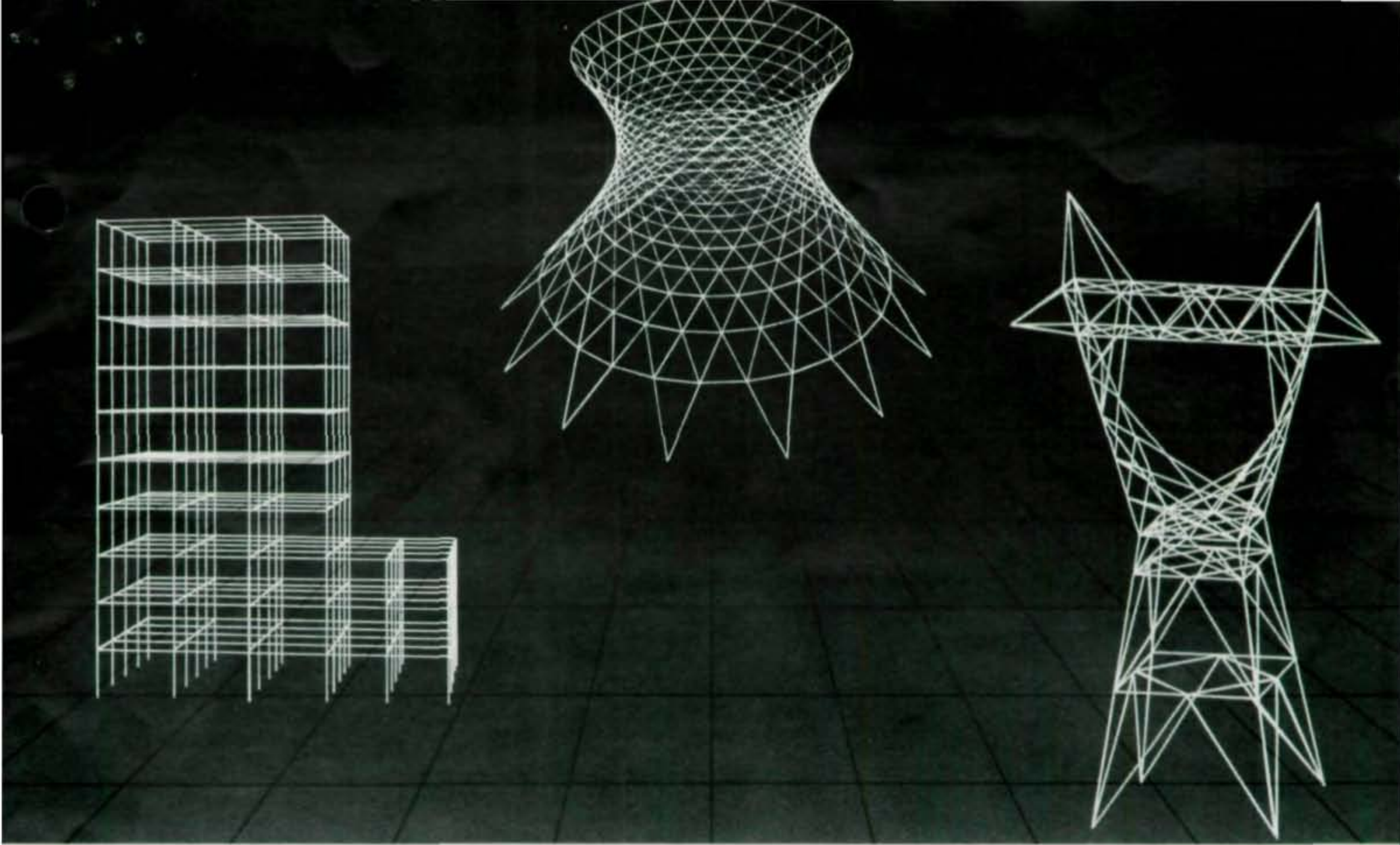
Now, some cutting-edge engineers are working with a variation on the theme to even further improve the seismic performance of structural steel buildings. Steel, due to its unique combination of strength and ductility has long been recognized as the material of choice for seismic areas. Still, designers were always faced with the dilemna that increasing stiffness tended to reduce ductility, and vice versa. Fortunately, there is a solution.

Instead of a traditional Concentric Braced Frame, some designers—especially in California—are increasingly relying on Eccentric Braced Frames (EBF). As the name implies, with a EBF the bracing members do not meet at the same point, but instead are slightly offset. This separation, or link, builds additional ductility into the system. As Jason Louie, S.E., points out in his article beginning on 20, "the ultimate desire is to have the link yield first to dissipate energy in any potential overload situation." And as an added benefit, moving the bracing element away from a corner simplifies connections.

The vast majority of EBFs built to date have been in California, though this type of design is slowly making its way east (see the article on a Boston project beginning on page 14). One reason for the slow movement is that an awareness of seismic risks is relatively recent outside of California. But another reason is a basic resistance to ideas emanating from the West Coast.

But regardless of accents or terminology, it's important to take note of advances in engineering, whatever their source.

As more of the country becomes aware of the need for seismic design, California's experience with earthquakes will be an important lesson. In addition to reporting on EBFs, this issue also presents an innovative seismic retrofit project (page 24) and some information on upgrading bridges to withstand earthquakes (page 32). **SM**



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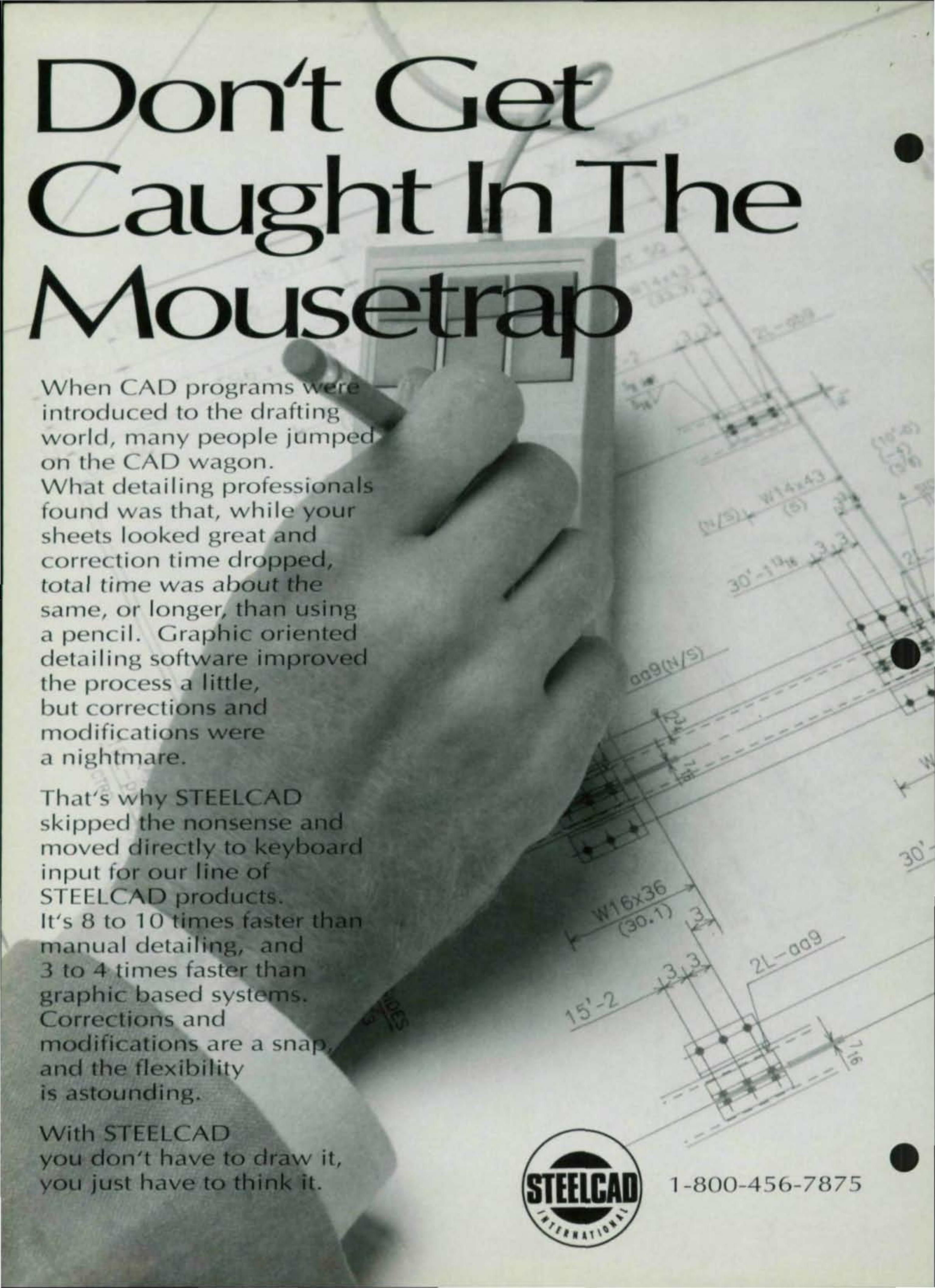
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Steel Interchange

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

Steel Interchange
Modern Steel Construction
1 East Wacker Dr., Suite 3100
Chicago, IL 60601

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

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

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

The following comments and responses to questions from previous Steel Interchange columns, as well as to other articles in this and other AISC publications, have been received:

What is the effect of designing with LRFD on deflection and vibration?

With the increasing popularity of LRFD, I expect that deflection and vibration related concerns will become a large part of claims of structural engineers.

Conclusions from deflection studies (such as those in MSC April, 1992 *Mega Mall Creates New Shopping Experience*, p. 20) may be helpful in keeping structural engineers out of court, where the project size is not adequate to justify such studies.

Discussions of the deflection, vibration concerns may also reduce the hesitancy on the part of engineers to change from ASD to LRFD.

Roger W. McGarrigle, P.E.
Van Domelen/Looijenga/McGarrigle/Knauf
Portland, OR

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

In response to the question regarding a "wind" connection for the top of a column, I offer the detail shown in Figure 1. This is a slight variation or the "semi-rigid" sketches shown in the April Steel Interchange.

Donald J. Shurilla, P.E.
F & M Associates, Inc.
Allentown, PA

There are many different possibilities concerning a good "wind" connection at the top of a column depending on shop and field preferences. My favorite is shown in Figure 2.

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

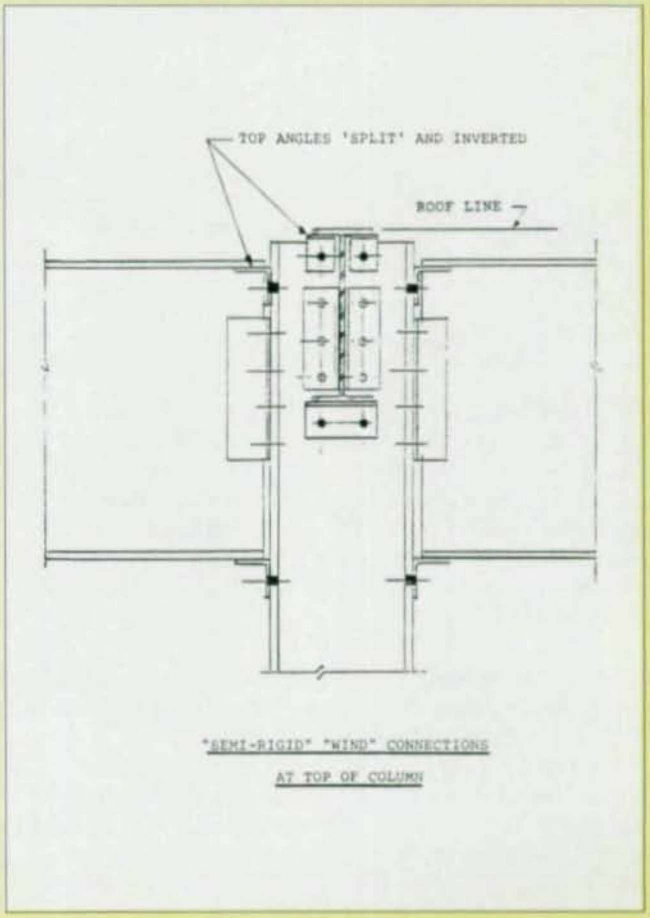

Figure 1: Semi-Rigid Wind Connection

Steel Interchange

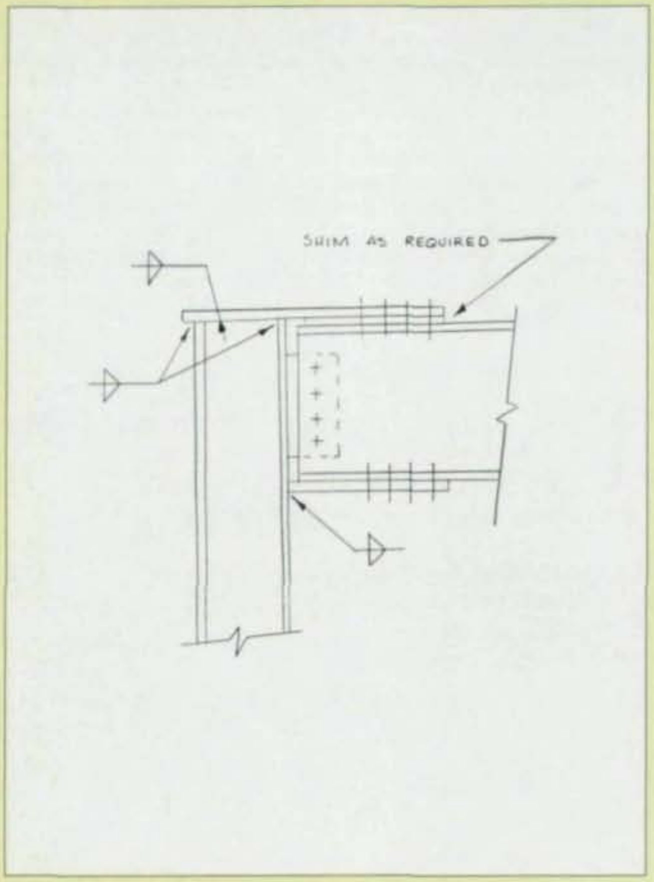

Figure 2: Wind connection at top of column

New Questions

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

1. The following oddity exists when comparing 0.4Fy versus 0.3Fu shear stress values. The ratio of Fy to Fu for A36 and A572 Gr. 50 steel is not proportional, reflecting the lesser ductility of the higher strength steel. For applications based on Fv = 0.4Fy, the allowable shear stress for A572 Gr. 50 steel is 39% greater than A36 steel; however, for applications based on Fv = 0.3Fu, the allowable stress is only 12% greater for the A572 Gr. 50 steel.

For A36 steel there is an increase in going from $0.4F_y$ to $0.3F_u$. For A572 Gr. 50 steel there is a decrease.

Type of Steel	$0.4F_y$	$0.3F_u$
A36	14.4 ksi	17.4 ksi
A572 Gr. 50	20.0 ksi	19.5 ksi

Three questions arise from this paradox:

a)When a single round hole penetration is required in a beam web, is it proper to use $F_v = 0.4F_y$ or $F_v = 0.3F_u$ when calculating the beam shear capacity?

b)Would a row of bolt holes behave differently than one large round hole which resulted in the same net area?

c)Does the presence or absence of bolts in holes affect the shear capacity of the member?

David T. Ricker, P.E.
Payson, AZ

2. The AISC design procedure for end-plate moment connections is for static loading only. (See LRFD manual, 1st Edition, p. 5-143 and ASD Manual, 9th Edition, p. 4-116.) Why is this restriction made? What is the definition of static loading? Can this connection be used for a utility bridge that has wind loading? Can it be used on a frame that supports a crane runway?

Barry K Shriver, P.E.
Piedmont Olsen Hensley
Greenville, SC

3. How should I connect wide flange beams to all four faces of a structural tube column in such a way as to transfer wind moments as well as dead and live load reactions?

John W. Kush, P.E.
Zurheide-Herrmann, Inc.
St. Louis, MO

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Metric In Construction

A new newsletter, "Metric In Construction", is being produced by the Construction Metrication Council of the National Institute of Building Sciences.

The first issue reports of a variety of topics, ranging from the Federal mandate toward metrication to other countries' experiences with metric conversion. According to the newsletter, design firms in Britain, Australia, South Africa and Canada all found that it took less than a week for staff to begin thinking and producing in metric when those countries converted from the inch-pound system during the past two decades. "The architecture/engineering community preferred metric dimensioning since it was less prone to error and easier to use than feet and inches," the report concluded.

Metric also offered a chance to reduce the many product sizes and shapes that had accumulated over the years but were no longer useful.

The newsletter also includes a listing of large federal metric projects and publication resources.

Publications on metric conversion are available from both NIBS and AISC.

NIBS offers a 34-page booklet, "Metric Guide for Federal Construction", which includes: an introduction to metric; a primer on metric usage for the a/e/c community; guidance on metric management and training; and a complete list of metric construction references. To receive a copy, send $15 to Metric Guide, NIBS Publications Dept., 1201 L Street, NW, Suite 400, Washington, DC 20005 or call (202) 289-7800.

AISC offers two publications: "Metric Properties of Structural Shapes with Dimensions According to ASTM A6M" and "Metric Conversion: Load and Resistance Factor Design Specification for Structural Steel Buildings." The former is a 92-page metric version of Part 1 of the *Manual of Steel Construction*, while the latter is a 159-page metric version of the 1986 LRFD Specification. Both cost $10.

For information, contact AISC at (312) 670-5411.

Research Funding

AISC has joined with the National Science Foundation and AISI to sponsor structural steel research. The focus will be on: applications of high performance steel, innovative types of connections, fabrication and erection techniques; innovative applications of steel in buildings, deep foundations, bridges and other structures; and technology transfer tools.

Suggested topics include: superstructure and substructure interaction of steel bridges; steel-framed multi-family residential construction systems; sandwich panels; steel building structures and steel pile foundations, including partially restrained connections low-to-moderate seismic effects; and fabrication-oriented projects.

Proposals are due July 15, 1992 and should be sent to NSF, 1800 G Street, NW, Washington, DC 20550. For more information, contact: AISC (312) 670-2400; AISI (202) 452-7196; or NSF (202) 357-9542.

People In The News

Timothy J. Gates has been named president of AISC-member Southern Ohio Fabricators, Inc. and J.J. Kling will move from president to Chairman of the Board of Directors of the Cincinnati-based structural steel fabricator and erector located. Gates, formerly vice president—finance, joined the company in 1984. The company has annual revenues of $45 million. In addition to its high-profile construction projects within a 300-mile radius of its plant, it fabricates industrial plate assemblies for industry nationwide.

Quality Management

The American Consulting Engineers Council's new video-based course, "Quality for Design Firms", features Bill Hayden, a well-known authority on total quality management. The video and accompanying 186-page workbook are a full how-to course. Cost is $345 for ACEC members and $495 for non-members. For more information, contact: American Consulting Engineers Council, 1015 Fifteenth St., NW, Washington, DC 20005.

The American Society of Civil Engineers has more than 10 publications available on quality management, including: "Four Propositions for Quality Management of Design Organizations;" "Construction Quality Management;" "Quality Management and the Civil Engineer;" and "Meeting the Quality Management issue on Highway Construction." For more information or a complete listing of QM publications, contact: ASCE Marketing, 345 East 47th St., New York, NY 10017 (800) 548-ASCE.

Clarifications

Re: **The International Facility at Chicago O'Hare International Airport.** Michael S. Fletcher, P.E., should have been listed as a joint author of "O'Hare Airport Landmark" (page 12, June 1992). Fletcher is an associate and director of structural engineering at Perkins & Will in Chicago.

Re: **Shortspan Bridges.** J.M. Montgomery, Jr., P.E., C.D. Gorman, P.E., and R.P. Alpago, P.E., all from Bethlehem Steel Corp., should have been listed as co-authors of "Shortspan Bridge Design In The 1990s" (page 32, June 1992). The other authors listed did not work on that paper but did present information on shortspan bridges at the National Steel Construction Conference.

We apologize for the errors and ommissions.

Calendar

July 7-8. **Welding Structural Design two-day seminar,** Houston. Designed to provide engineers and welding inspectors a greater understanding of weld mechanics and welded engineering structures. Contact: AWS, 550 N.W. LeJeune Road, P.O. Box 351040, Miami, FL 33135 (800) 443-9353.

July 16. **Southeast Steel Bridge Forum,** Columbia, SC. Contact: Camille Rubeiz, Steel Bridge Forum, c/o AISI, 1101 17th St., N.W., Suite 1300, Washington, DC 20036 (202) 452-7190.

August 18-19. **Welding Structural Design** two-day seminar, Miami (See July 7-8).

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

September 22-24. **Fracture & Fatigue Control in Structures** short course, Lawrence, KS. Contact: The University of Kansas, Division of Continuing Education, Attn: Management Programs, Continuing Education Building, Lawrence, KS 66045-2608 (913) 864-3968.

Eccentric Braced Frames Head East

An eccentrically braced building in Boston is one of the first uses of the new AISC LRFD Seismic Provisions

By John R. Boekelman, P.E., and
Bruce E. Randall, P.E.

For both its owner and designer, the Boston University Medical Campus Center for Advanced Biomedical Research in Boston represents a series of "firsts".

For Boston University, the building is an important first step in a joint development between its medical campus and University Hospital to create a medical research center in Boston's South End.

For Cannon, architects and structural engineers on the project, the research center presented a unique opportunity to be one of the first designers to put into practice the standards set forth in the 1990 "Seismic Provisions for Structural Steel Buildings—Load and Resistance Factor Design."

Challenging Program

The university's program for the 10-story research facility placed a premium on adaptability of the laboratory spaces and specified a 16' floor-to-floor height to accommodate equipment and mechanical services. Working with a laboratory consultant, the design team determined that the ideal lab size would have a 22' x 33'-4" bay.

The program also called for numerous full-height grouted masonry partitions on the upper levels designated for animal care. The added weight of these heavy interior partitions, combined with a bi-level mechanical penthouse and a brick exterior that required masonry back-up, placed significant mass at the top of the structure.

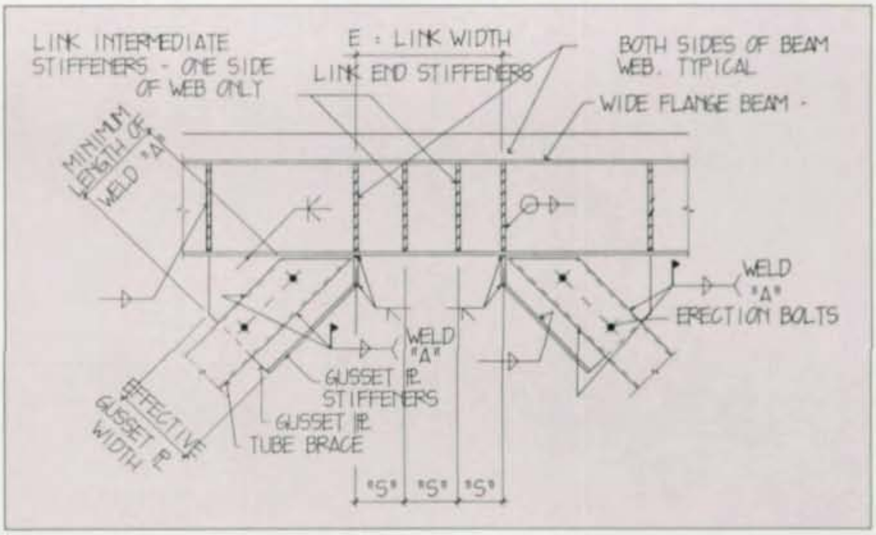
An additional consideration was that the site is underlaid with an organic layer of soils up to 25' thick and deep clay deposits. Competent soils are more than 100' below grade, requiring a deep foundation. Also, the deep, soft soils could potentially amplify bedrock ground motion, requiring a soil seismic load factor of 1.5 in accordance with the Commonwealth of Massachusetts Building Code.

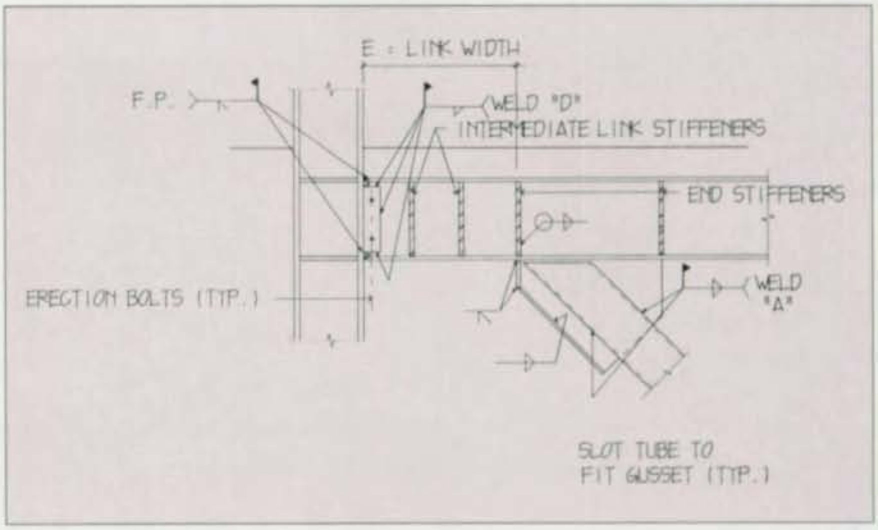
The program requirements for this building made it difficult to determine an appropriate lateral resistance system. The tall floor-to-floor height: increased the wind "sail" area per floor; increased the overturning per level; reduced any given column's allowable axial capacity; and increased the column's flexibility. The additional mass at the top of the building, the soil amplification factor, and the floor-to-floor heights all contributed to boosting the seismic base shear and overturning moment well above the normal level for a 10-story building. Adequate drift control became a major design criteria and several alternate lateral load systems were analyzed.

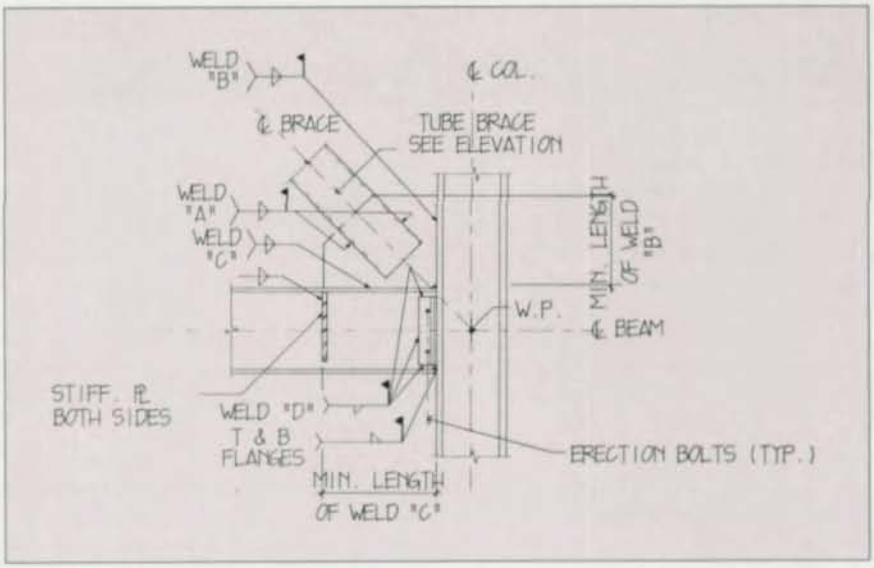
In the broad face (transverse) direction, moment frames were not efficient at controlling drift, especially given the tall floor-to-floor heights and relatively long, 33' spans in open plan. The required girder depths would have taken away from the desired interstitial spaces and made a strong column/weak girder system difficult to achieve.

Conventional concentrically braced frames would have controlled drift, but their lack of ductility is penalized in designs governed by seismic forces. Concentrically braced frames have no mechanism to deform in a duc-

In eccentric braced frames, ductility is substantially increased by connecting each bracing member to the beam a short distance from the beam-to-column connection or from another beam-to-brace connection. Photos by Lucy Chen







tile manner when overloaded; the frame fails by buckling of the compression diagonal, a brittle failure. The higher design load factors on this project would have required that the lateral load system be designed for seismic loads well in excess of wind loads, a potentially large cost inefficiency.

Instead, the designers looked at eccentrically braced frames (EBFs). Since their introduction in the late 1970s, EBFs have shown to be a highly desirable lateral load resisting system, combining the ductility of special moment frames with the stiffness of concentrically braced frames. For this project, EBFs could control drift with a minimum of material, and without reliance on the stiffness of long, slender columns. The ductility of the system allowed code level seismic forces to nearly equal wind forces. The resulting system has exceptional ability to absorb energy during seismic overload, provides damping, and increases the building dynamic period to limit seismic forces.

Code Problems

Unfortunately, EBFs had not previously been used in the Northeast. In fact, due to a lack of recognition of the system's ductility and performance by most model codes and a lack of a set of coherent design guidelines, use of the system was not widespread outside of California. While Massachusetts has been a leader in incorporating modern seismic design principles into its code, neither the Massachusetts Code nor BOCA had appropriate EBF "K" factors (though UBC does).

Fortunately, Massachusetts does recognize LRFD as an appropriate

A Chevron bace configuration was selected for the main EBFs in the Boston University Advanced Biomedical Research Center. This geometry moved the highly stressed link area away from the beam/column connection and reduced reliance on properly made full penetration field welds for good performance in the inelastic range.

steel design method, and the 1990 AISC LRFD seismic provisions were published at about the same time as the building was being designed. The provisions specify a K factor of 0.67 for EBFs and outlines a series of guidelines for use in EBF design. This was important because a proper K factor, or building response factor, is necessary to make EBF systems economically competitive. As a result of this inclusion, Massachusetts allowed the building to be designed with an eccentrically braced frame.

Lateral Resistance System

In plan, two Chevron-type EBFs were placed symmetrically in the transverse direction. Moment frames were introduced along the narrow face perimeter of the building to provide redundancy as a dual system and help control drift. To reduce the amount of uplift at the base of the columns in the EBF, one story Vierendeel outrigger frames were introduced at the second floor. In the longitudinal direction, the columns are spaced more closely (22′ centers) and the number of bays available made a perimeter ductile moment frame a reasonable system.

During design development of the exterior facade, the second floor slab was recessed behind the plane of the exterior columns. A perimeter moment frame at this level was no longer possible and moving the frame to the interior column lines was ruled out due to two-story interior spaces and headroom requirements. Instead, in the longitudinal direction, single diagonal EBFs were created adjacent to the stair shafts (see Plan). These EBFs were carried up to the fourth floor to engage two levels of diaphragm for transferring the shear from exterior to interior. The columns common to frames in both directions were boxed and the connections carefully detailed.

A Chevron brace configuration was selected for the main EBFs. This geometry moved the highly stressed link area away from the beam/column connection and reduced reliance on properly made full penetration field welds for good performance in the inelastic range. The other advantages of this configuration for the 27′-8" center bay are symmetry and appropriately steep braces. If the brace/beam angle gets to be too shallow—less than 35 degrees, for example—then the axial forces to be resisted by the beam segment outside the link become very large under link yielding and are difficult to control.

The remainder of the building frame is conventional composite slab and beam construction, with the typical beams spaced at 11′ centers. Slabs are 3¼" lightweight concrete on 3" composite metal deck with headed shear studs to create the composite action. Depending on whether stress or stiffness controlled design, either A36 or A572 Grade 50 steel was used for both columns and beams. All were designed according to the LRFD Specification.

The ETABS program from Computers & Structures, Inc. was used for 3D analysis.

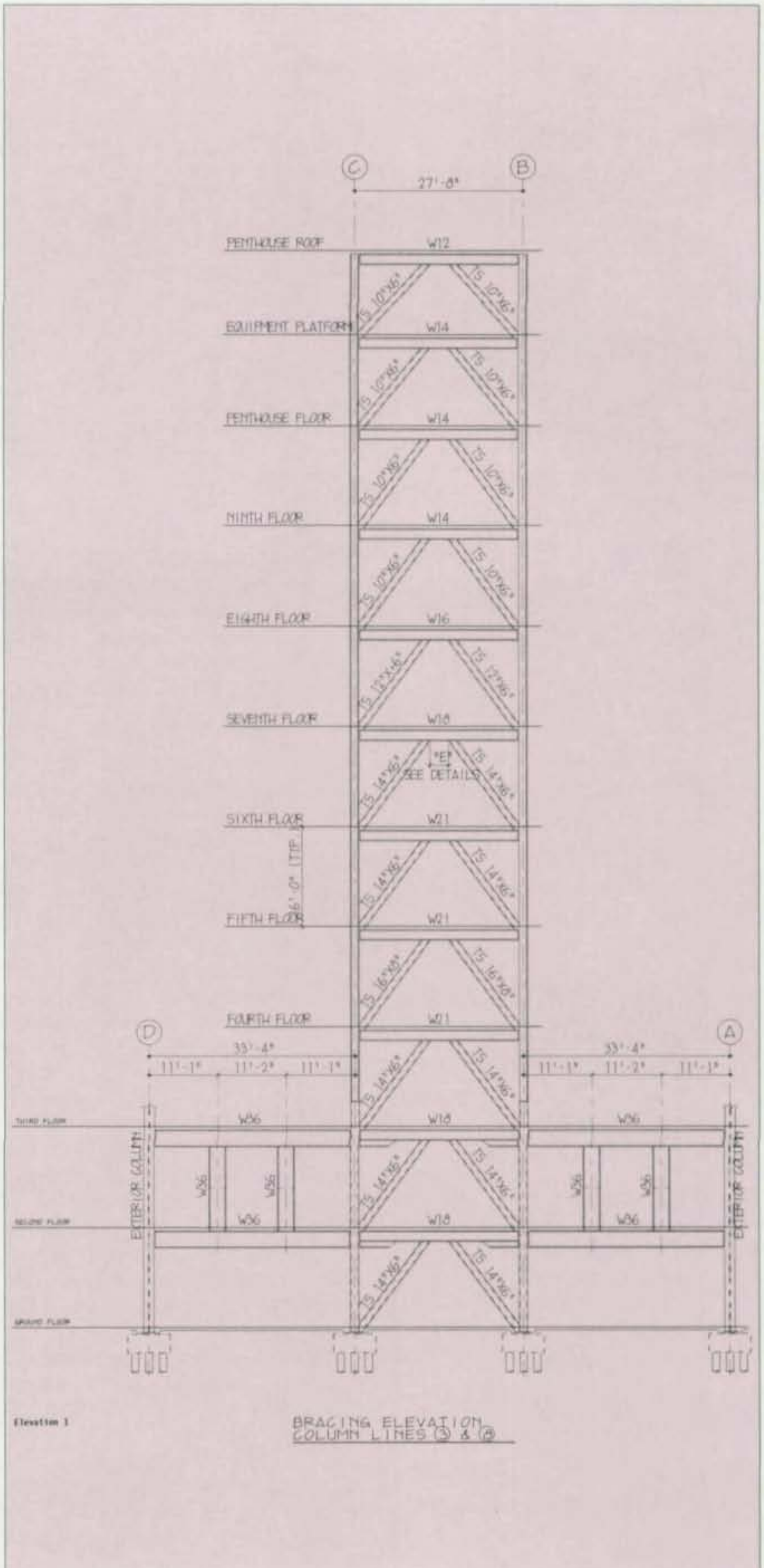
Designing EBFs

Chapter 9 of the "Seismic Provisions for Structural Steel Buildings—Load and Resistance Factor Design" provides the most comprehensive code requirements for EBFs published to date, and includes an extensive commentary to aid in understanding the principles involved. An eccentric brace is created when one end of each brace is deliberately offset along the beam from the normal common intersection point at the column to create a "link" (see detail). The link is a very ductile zone which is expected to yield under overload. The link properties are included in the frame analysis and sized for both allowable stress and building drift.

Under normal wind and seismic conditions, eccentric braced frames perform in much the same way as concentric braced frames, with all members expected to remain elastic at these force levels.

However, while code level wind forces attempt to approximate the maximum expected wind load during the life of a building, the actual forces generated by the maximum probable earthquake for any given site may be far greater than the code level earthquake forces. Since earthquakes are an infrequent event—at least in the eastern U.S.—eccentrically braced buildings are not designed to remain elastic under this maximum load, but instead are expected to remain stable and survive without collapse. During overload the link is proportioned to be the element which will yield.

Well-designed links will deform in *shear*, which research has shown is an excellent means of absorbing



energy while retaining strength and stability. The links are stabilized by stiffener plates to prevent premature buckling and by perpendicular braces at each end to restrain out-of-plane warpage.

The ensure that the links will be the element to yield, the braces, the portion of the beam outside the link, the columns, and the connections of the frame are all designed using the strain hardened capacity of the link rather than code lateral forces. In other words, once the links have been designed for proper elastic behavior under code level forces, the design criteria for the other members are determined using a load factor multiplied by the plastic capacity of the link as the design input force. This capacity design approach is to ensure that the link will be able to achieve its full inelastic capacity while the rest of the frame remains elastic, regardless of the magnitude or distribution of the dynamic forces.

Lessons Learned

Experience on this project indicates that the frame should be proportioned to keep the link width at or slightly less than the ratio 1.6Mp/Vy. This ratio is specified in the code as the limit whereby link yielding is primarily in shear rather than in bending, as shear has been shown to be the superior mechanism of inelastic behavior.

The brace and the beam outside the link should be selected as column-like sections. The beam should be selected based on the plastic shear capacity of the web of the beam exceeding the factored shear resulting from code level forces. The beam outside the link is designed as a beam-column, resisting a percentage of the moment induced by the link beam and the horizontal axial component of the diagonal brace.

The LRFD provisions specify that the brace connection to the link beam be designed for the nominal capacity of the brace member. This creates a rigid connection that has stiffness at the link beam. This stiffness causes the brace member to resist a percentage of the link beam moment as well as the axial force from the strain hardened capacity of the link.

Project experience indicates that approximately 80% of the link beam moment is transmitted to the beam outside the link and the remaining 20% is resisted by the diagonal brace. These values can be used when initially selecting the members.

Designing for the strain hardened capacity of the links has the most dramatic effect on the frame columns. At present, the code requires that columns be designed for the *sum* of all the links above the level under consideration reaching strain hardened capacity simultaneously, which would correspond to building vibration in the fundamental mode only. While space prohibits a full discussion of the probability of this mode of failure, it can be briefly stated that the forces accumulated at the bottom of a multi-story building get to be very large.

The sections selected for the link beams must be chosen with care and not conservatively designed, especially at the top floors. Otherwise, the accumulated forces on the columns will become too great; resisting the amount of uplift at the foundations especially could become impractical. For this project, widening the base of the frames to reduce the uplift at the base of the main EBF columns was desirable. Additional bays of EBFs could not be introduced at the lower levels for architectural reasons. The one-story-high Vierendeel frames provided the means to shed some of the uplift from the EBF columns to the exterior columns.

Fabrication Modifications

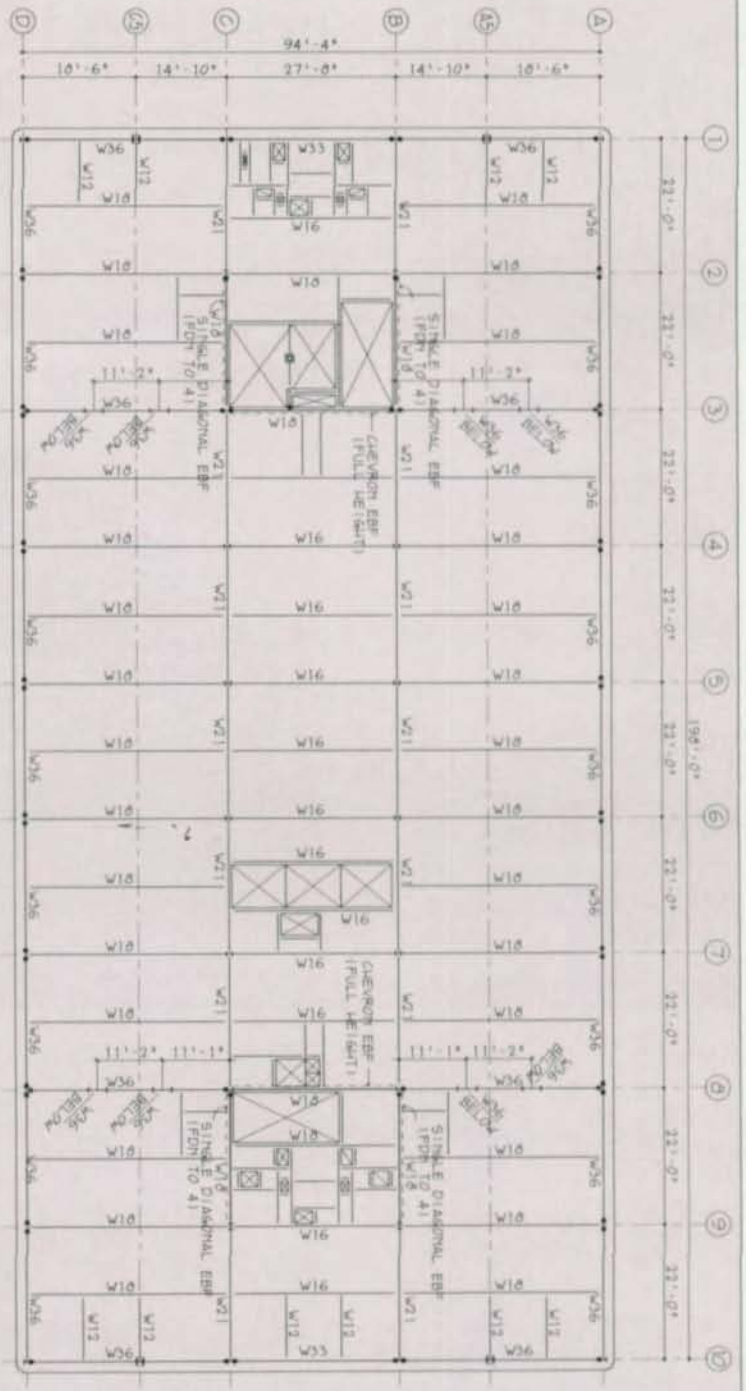
All of the connections within the EBFs were designed and detailed by Cannon Design. The steel fabricator then suggested modifications to aid in fabrication and erection. These were reviewed and incorporated where possible. Close control of the connections was considered important to the frame behavior; this approach was used in lieu of listing performance criteria and having the fabricator design the connections.

Correct detailing of the frame is essential to proper inelastic behavior. The code lists requirements for stiffener size and spacing within the link, and defines the lateral support required at each end of the link for both the top and bottom flange. In this case, the end stiffeners were used as single shear plates to connect full depth perpendicular beams for lateral stability.

Connection of the brace to the beam at the link is another key detail. The code recommended detail is to cut the brace parallel and as close as possible to the bottom flange of the beam, and to provide stiffeners on the bottom edge of the gusset plate to prevent buckling. These requirements complicated field erection, as either the gusset prevented swinging the brace into position, or the gusset would have to be field welded. The fabricator solved the problem by field bolting each brace to the gusset plate of the link beam above and then lifting the entire assembly into place.

With the publication of the LRFD Seismic Provisions, and now with the publication of the 1992 supplement to BOCA, eccentrically braced frames are a defined structural system across the U.S. As engineers become more familiar with the advantages of this system, its use should become more widespread in buildings where seismic loads govern lateral system design and where drift is a primary concern. To purchase a copy of the LRFD Seismic Provisions, send $5 + $4 postage and handling (IL, NY, and CA residents please add sales tax; foreign orders are $5 + $5 shipping) to: AISC, Inc., P.O. Box 806276, Chicago, IL 60680-4120 or call (312) 670-2400 ext. 433. Specify publication S339L.

John R. Boekelman, P.E., and Bruce E. Randall, P.E., both are senior associates with Cannon Design, an architectural/engineering firm headquartered in Buffalo. ■



THIRD FLOOR FRAMING PLAN

Combined Systems Enhance Seismic Performance

Utilizing both Eccentric Braced Frames and Special Moment Resisting Frames creates a structure with both ductility and stiffness

Each of the five four-story buildings of the Apple R&D Center consists of two wings with two Eccentrically Braced Frames in each wing and Special Moment Resisting Frames located at the building's perimeter.

By Jason J.C. Louie, S.E.

While many residents of the earthquake-prone San Francisco Bay area give only occasional thought to "The Big One", for structural engineers it is a constant concern.

Fortunately, design engineers have developed a variety of methods to cope with the threat of earthquakes. Among the most effective is combining Special Moment Resisting Frames (SMRF) with Eccentric Braced Frames (EBF), such as was done on Apple Computer's R&D Center in Cupertino, CA.

The R&D Center consists of five, four-story steel framed office building and one two-story commons building featuring a cafeteria, library and auditorium. As is often the case with low-rise office complexes, the buildings were designed in a cluster arrangement and feature an extensively landscaped campus. Because the computer and manufacturing equipment in the buildings is so valuable, it was essential that the structure *and its contents* be able to survive an earthquake.

In the past, it would be common to design the structure with a Concentric Braced Frame (CBF). This type of system is designed to restrict a structure's lateral movement, thereby limiting damage to non-structural elements. However, CBF's have little ductility and only a limited ability to absorb cyclic seismic energy. Also, CBF's are generally associated with high overturning forces, where one column may tend to go into tension, or lift off, requiring piles, deep foundations or large footings (mass) to hold them down.

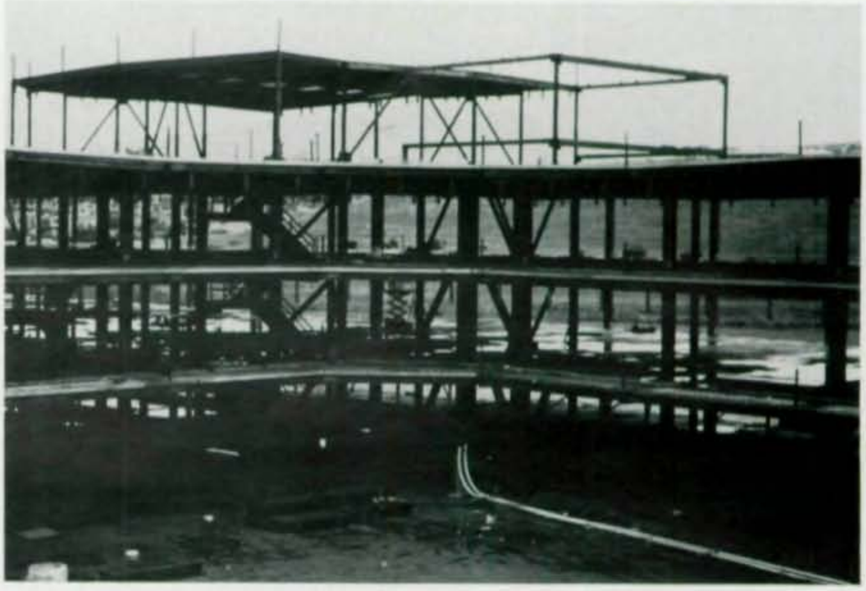
Today, structural engineers are increasingly depending on alternate systems in seismic areas.

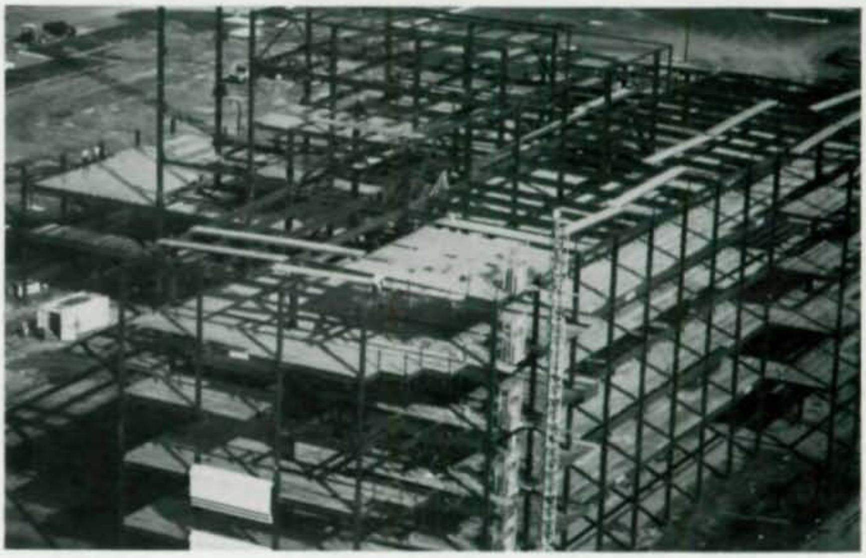
Martin, Middlebrook & Louie, the Apple project's structural engineer, has been designing combined SMRF/EBF systems since 1985. "As building technology increases, our designs have become more streamlined and sophisticated," noted Ron Middlebrook, S.E., the firm's managing principal. "And with refined technology and application, the buildings get better seismically, while construction costs, not to mention maintenance/damage repair costs, can actually decrease."

SMRF's have long been recognized by the Uniform Building Code (UBC) as an excellent lateral load resisting systems for seismic areas due to the system's very high ductility. And for low-rise buildings, an added advantage is that the system is less likely to require special treatment for overturning because dead loads are sufficient to offset the tendency for tension forces on foundations.

EBF's, like the more familiar CBF's, consist of columns and beams with angled bracing. However, the placement of the bracing in EBF's joins floor and roof beams a slight distance away (eccentrically) from the column/beam joint, forming a girder link. A certain amount of ductility or reserve is achieved through bending in the girder link induced by the force from the original angled brace when excited by an earthquake or other lateral loads.

EBF's must be carefully designed for the correct link dimensions (amount of eccentricity) for the desired combination of stiffness (drift control) and redundancy, and carefully detailed with the appropriate number of web stiffeners and a positive brace at the bottom flange of the girder at the link point. The ultimate desire is to have the link yield first to dissipate energy in any potential overload situation. An added advantage of EBF's is that by moving the connection from the corner where a beam meets a column, it simplifies the corner detail for the fabricator and erector.





Combining SMRF's with EBF's proved especially advantageous given the Bishop Ranch's 16' floor-to-floor heights. Had only SMRF's were used, more steel would have been required to control building drift.

Ductile Design

Ductile steel design—as in SMRF's and EBF's—allows, through displacement, the absorption of greater amounts of energy with less distress. The idea is to have the ultimate "failure" occur as plastic hinges in the girders outside the joints—and to keep the columns and joints stable (as per the strong column/weak beam theory).

EBF's exhibit more ductility than CBF's and more stiffness or resistance to drift than SMRF's. Therefore, while code penalties for CBF's for seismic resistance are 50% over those for SMRF's, the penalty for EBF's is only 20% more than for SMRF's. Also, EBF's are generally associated with moderate overturning forces (greater than SMRF's but less than CBF's), which means that tension forces are more easily dealt with than with a CBF.

A combined SMRF/EBF has a variety of advantages over an SMRF alone. Combined SMRF/EBF layouts provide superior ductility and redundancy with no seismic force penalty. The combined system also decreases overturning effects as well as minimizing building drift—which theoretically results in less damage to contents and non-structural components.

Current UBC seismic provisions are primarily written to maximize life-safety and not necessarily to minimize damage. Combined SMRF/EBF systems generally result in dispersion of lateral load resisting elements throughout the building floor plate, resulting in shorter spans and greater drift control for floor and roof diaphragms—again, less movement and better redundancy. If one frame is overstressed, another will pick up the excess.

> Combining EBF's with SMRF's proved faster to erect and lower in cost, while also allowing greater flexibility in architectural design

Combining Systems

Often the best combination is comprised of SMRF's on the perimeter and judiciously placed EBF's in the core or other interior locations, as was done with the $82 million, 1,175,000-sq.-ft. Apple R&D Complex.

Each of the five four-story buildings consists of two wings with two Eccentrically Braced Frames in each wing and Special Moment Resisting Frames located at the building's perimeter. An additional advantage of this configuration is that it allowed the project's architect, Helmuth, Obata & Kassabaum, greater flexibility when placing windows and doors: There was no tell-tale diagonal angling seen through exterior glazing. ETABS and SAP 90, both from Computers & Structures, Inc., were used for frame analysis.

The beams on the project are A36 steel and range in size from W30 to W36, while the columns are A572 Grade 50 and range in size from W14 to W18. The braces are standard and built-up W10s. A variety of connections were utilized, including complete penetration welded flanges, bolted, and welded web. Fabricators on the project included AISC-member PDM-Strocal, Inc. Contractor was Rudolph & Sletten.

Variation On A Theme

Martin, Middlebrook & Louie has used slightly different configurations of combined SMRF/EBF systems on a number of projects, such as Sunset Development Co.'s award-winning Bishop Ranch Complex in San Ramon, CA. Designed by architects Hoover Associates, Palo Alto, CA, the several million sq. ft. complex accommodates several large corporate clients in their own facilities as well as dozens of buildings owned by the developer.

Bishop Ranch 3, 5, & 7 are three prototypical, three-story, steel-framed office buildings utilizing the combined system. Each building, due to a large, 240,000-sq.-ft. footprint, is separated into three parts by seismic joints. The main central portion of each building is hexagonal in plan with large central courtyards. The hexagons are further divided into two L-shaped SMRF ringed shapes joined by floor and roof diaphragms at their ends. The rectangular wings also have perimeter SMRF's. In addition, each of the three floor plan elements has a U-shaped (in plan) EBF. The braced frames were sized to provide stiffness to control drift, while the Special Moment Resisting Frames were designed to resist the seismic forces.

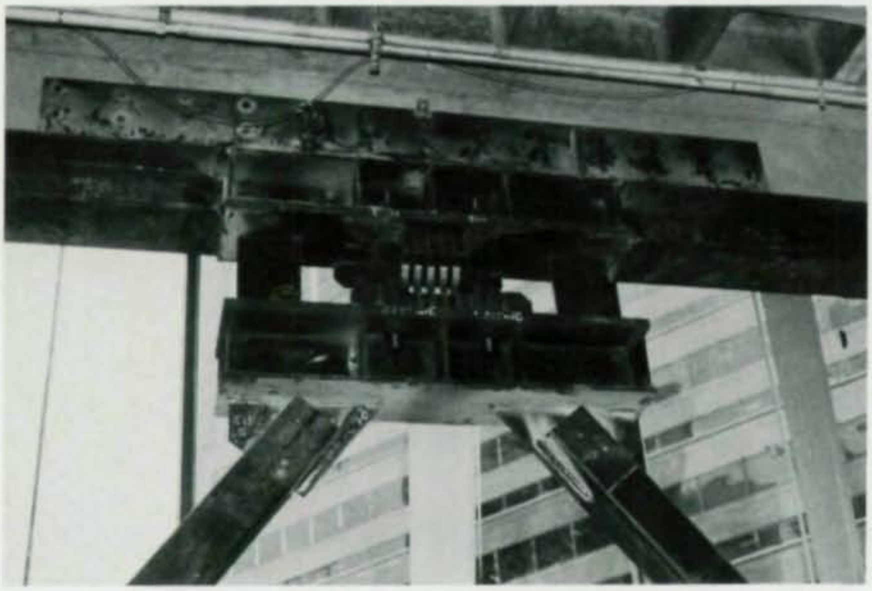
The combined system proved especially advantageous given the project's 16' floor-to-floor heights. If only SMRF's were used, more steel would have been required to control building drift.

A36 steel was used for the W24 beams, while A572 Grade 50 steel was used for the W12 and W18 columns. The braces are 8"x 8", 10"x 10", and 12"x 12" tubes. Tubes are a more efficient compression member than are wide flange shapes, and therefore using tubes for the bracing reduced the amount of steel on the project. Fabricator was AISC-member The Herrick Corp. and contractor was Sunset Development Corp.

The combination frame proved faster to erect and lower in cost than either concentric braces or ordinary moment resisting frames. And, of course, the combined system allowed for greater flexibility in architectural design.

Jason J.C. Louie, S.E., is a principal with Martin, Middlebrook & Louie, Consulting Structural Engineers, in San Francisco, CA. ■

Damping System Aids Seismic Retrofit



The photos above show an Added Damping and Stiffness (ADAS) element during fabrication and after installation in the Wells Fargo Building in San Francisco. The unit includes five 1.5"-thick 50 ksi steel plates clamped together with A490 bolts.

The first commercial application of an Added Damping and Stiffness element in the U.S. proved successful in retrofitting a bank building

By Eduardo Fierro, P.E., Cynthia L. Perry, P.E., and Thomas R. Varner, S.E.

Many structures in the San Francisco Bay Area were badly damaged by the 1989 Loma Prieta earthquake and the results are still in evidence in communities throughout the region. Lots remain vacant where buildings were demolished and plywood continues to cover windows that were destroyed. Many building owners and tenants still are weighing the relative merits and financial implications of demolition, cosmetic repairs, or voluntary seismic upgrades.

Following the Loma Prieta earthquake, Wells Fargo Bank retained Dames & Moore to evaluate the seismic exposure of many of its buildings. In addition, the firm was hired to evaluate the seismic vulnerability of a prominent Market Street building into which the bank planned to relocate a major branch office.

The study indicated that the 14,000-sq.-ft., two-story, non-ductile concrete building could be vulnerable to damage and possible collapse in a major earthquake, such as the 8+ magnitude earthquake that hit San Francisco in 1906.

Earthquake Damage

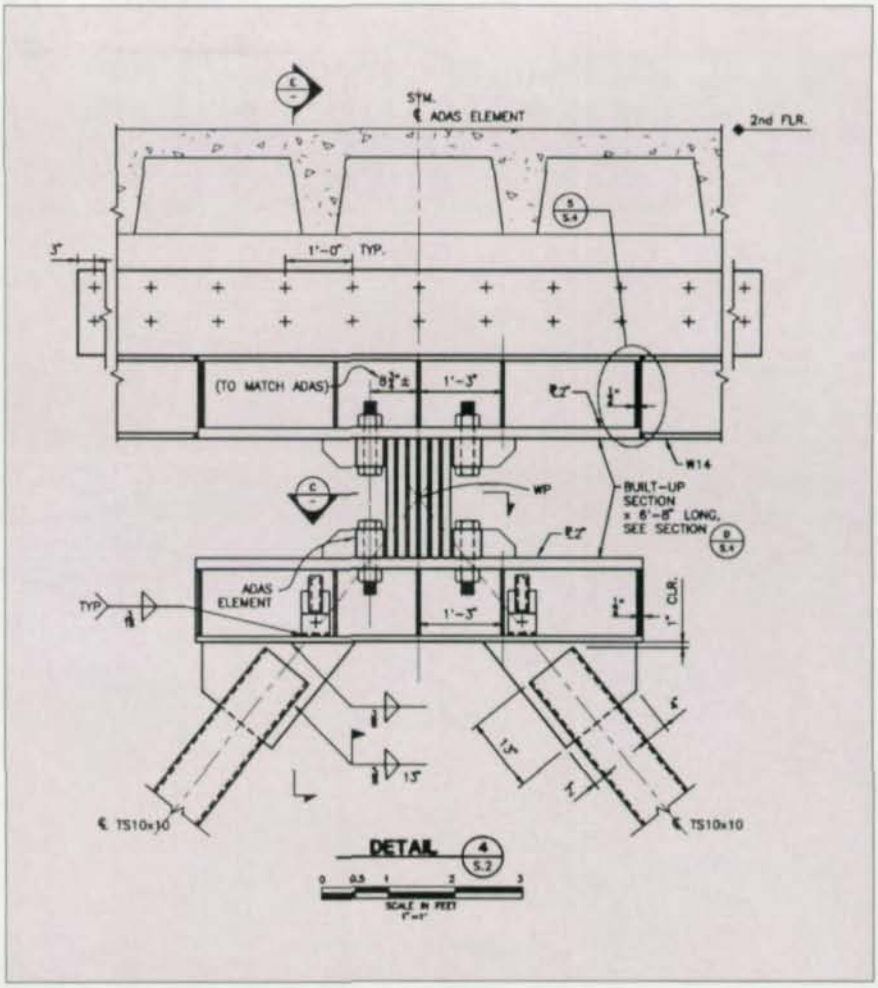
The building was constructed in 1967 on top of an underground parking garage that occupies an entire city block. The lightweight concrete-frame building measures 81' x 81' in plan and consists of three frames in each direction located at 27' intervals with four corner columns omitted. The floor and roof slabs are each 18.5" deep waffle slabs with integral interior beams and deeper spandrel beams at the perimeter. A concrete shear wall runs along the west side of the building from the street level to the second floor. The typical interior column is 24" x 24", reinforced with 16 #11 Grade 40 reinforcing bars with #3 ties spaced at 18" on center.

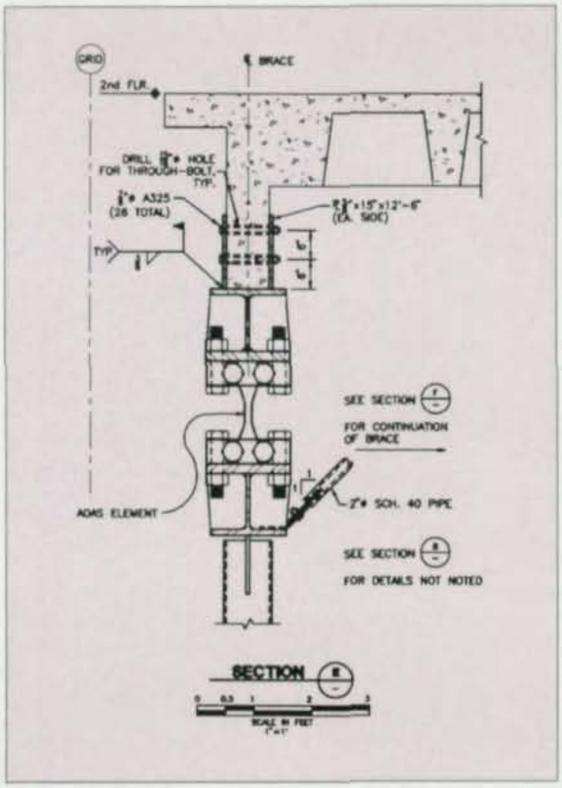
Although it appears the concrete frames were originally designed to resist seismic loading in both directions, the presence of the shear wall and partial mezzanine along the west side of the building has a profound effect on the dynamic characteristics of the structure. The structure had a significant torsional problem as a result of the increased stiffness of this shear wall and the fact that two of the interior columns were restrained against frame action by the mezzanine floor.

And, in fact, the building suffered both structural and non-structural damage during the Loma Prieta earthquake. Predictably, the two "short" columns showed classical shear cracking between the mezzanine level and the second floor. Also, a 22'-high plate glass window along the east side of the building shattered as a result of the structure's torsional deformations. A dynamic response spectrum analyses of the building simulating the Loma Prieta earthquake showed a similar pattern of damage.

Performance Standard

Both the bank and Dames & Moore were concerned that the Uniform Building Code's (UBC) "life safety" standard, which protects against building collapse in an





The ADAS units were attached to 10"x10" tube steel chevron braces.

AFE FROM EARTHQUAKES MPANIES THE SHAKES.

That was essential because the building, which was constructed for Evans & Sutherland Computer Corporation, is located within a mile of the Wasatch Fault in Salt Lake City. What's more, Evans & Sutherland is a leading designer of special-purpose digital computers, software systems and display devices — products extremely vulnerable to damage from seismic tremors.

To plan for maximum protection, Vulcraft was asked to join with the architects and engineers at the design stage of the project. Already, they'd decided to use a "base isolation" system, the most advanced buffering method available. But using our steel joists and joist girders was also an important decision. The joists and joist girders are much lighter in weight than wide flange beams, so the entire building required less steel, lighter columns and less foundation. And this not only lightened the load for the base isolators, it saved appreciably on building costs.

Throughout construction, Vulcraft remained constantly involved, tailoring our delivery of materials to the exact erection schedule and meeting deadlines without fail. What's more, our joists and joist girders helped the steel erectors meet their deadlines. That's because our products are fast and easy to erect — a fact that saves time and money on virtually any job where they're used.

The design team chose a "base isolation" system to accommodate seismic loads. They also specified Vulcraft steel joists and joist girders for the building.

So whether you need Vulcraft's help to protect your building from earthquakes or you want to stay out of the hole when it comes to construction costs, contact any of the plants listed below. Or see Sweet's 05100/VUL.

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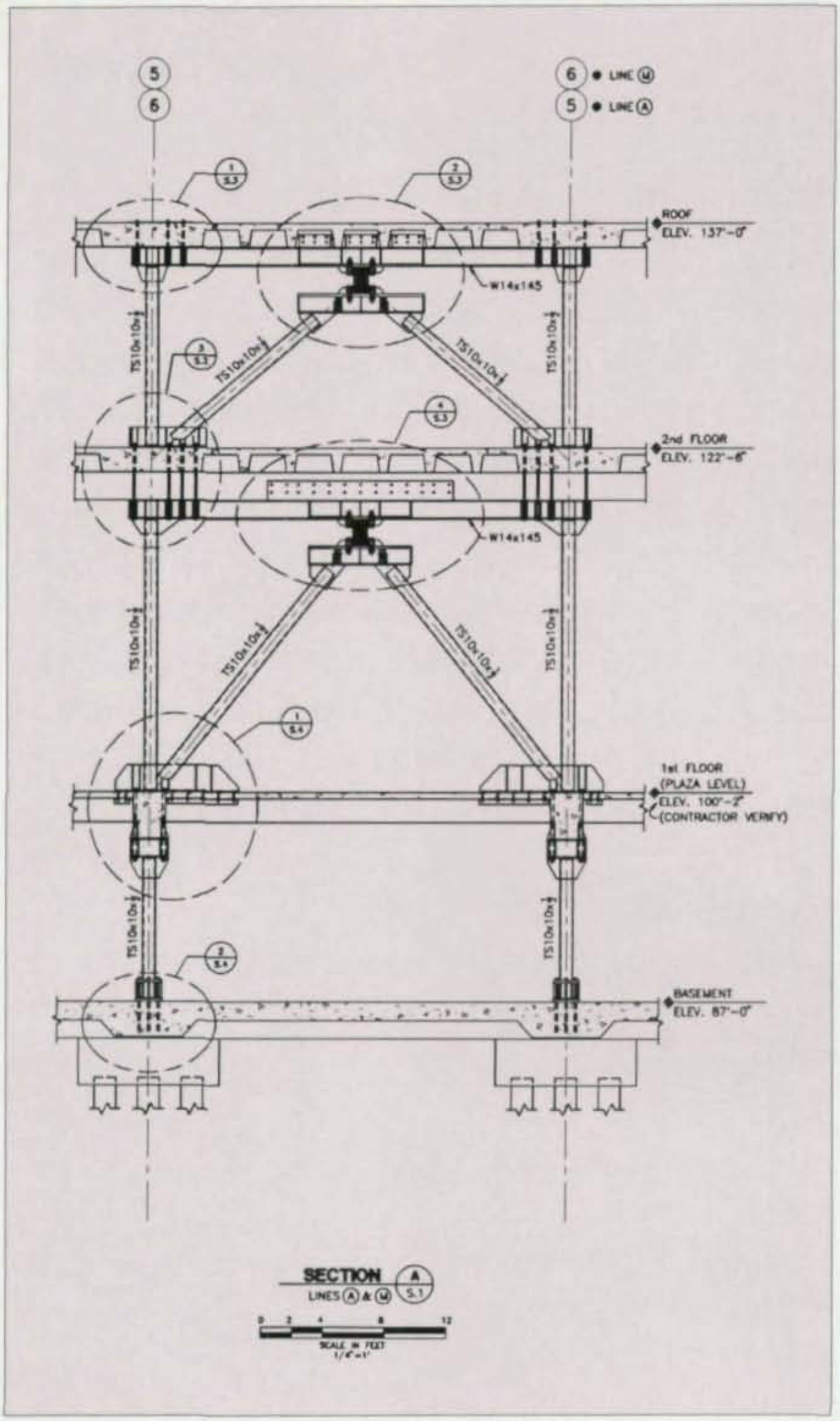
P.O. Box 637, Brigham City, UT 84302 801/734-9433; P.O. Box F-2, Florence, SC 29502 803/662-0381; P.O. Box 169, Fort Payne, AL 35967 205/845-2460; P.O. Box 186, Grapeland, TX 75844 409/687-4665; P.O. Box 59, Norfolk, NE 68701 402/644-8500; P.O. Box 1000, St. Joe, IN 46785 219/337-5411. Architect: Erlich-Rominger Architects AIA; Structural Engineer: Reaveley Engineers & Associates, Inc.; General Contractor: The Bettilyon Corporation; Steel Fabricator: D&H Steel Supply, Inc.; Steel Erector: Steel Deck Erectors, Inc.



(Section details are shown on the previous page)

earthquake but allows construction that could result in a structure being damaged beyond repair, was insufficient for the bank's needs. According to Thomas Bender, vice president and division manager of retail facilities for Wells Fargo, "the bank wanted to be able to repair the building, if necessary, and resume operations at this prominently located Market Street branch without any significant inconveniences to our downtown customers."

Unfortunately, typical retrofit alternatives, such as adding new shear walls or steel braces, were not practical since they would have interfered with the below-grade parking structure.

Instead, Dames & Moore designed a system which employs force-limiting energy dissipation devices in conjunction with steel chevron braces located at the perimeter of the building. In addition, the existing shear wall was retained and upgraded. This scheme significantly reduced the torsional response of the building and also alleviated the need for costly foundation work.

High-Tech Energy Dissipation

The energy dissipation devices used for this project, known as Added Damping and Stiffness (ADAS) elements, were developed jointly by Roger E. Scholl, Ph.D., of CounterQuake Corp., Redwood City, CA, and William H. White, Ph.D., of the Bechtel Corp., San Francisco. White and Bechtel jointly hold a patent for the devices, which were testing in 1988 at the University of California (Berkeley) Earthquake Engineering Research Center under the direction of 1990 AISC Higgins Award Winner Vitelmo V. Bertero.

Each of the seven ADAS devices consists of a series of 50 ksi steel plates cut in an hour-glass shape that bends in double-curvature flexure when subjected to lateral loading. The hour-glass shape was designed to match the moment diagram and provide uniform plastification over the height of the steel plates. These devices exhibited predictable, post-yield behavior during testing, with stable hysteretic loops

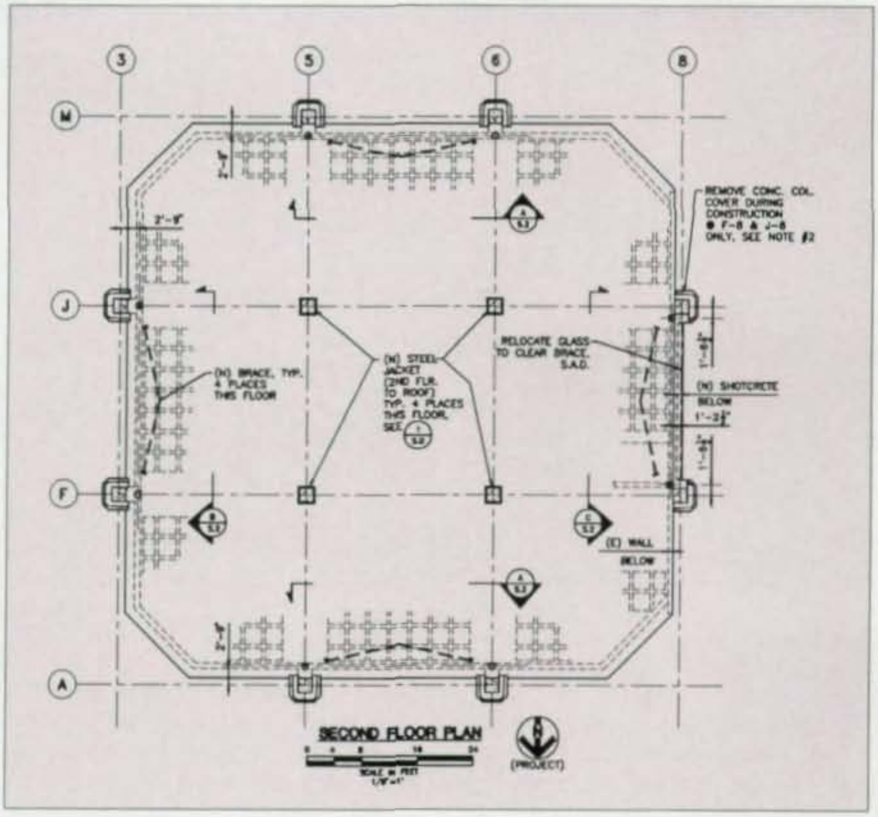
for many cycles of the load reversal, resulting in significant energy dissipation.

According to Scholl, these devices have been used for the seismic retrofit of three concrete structures in Mexico City, but the Wells Fargo project is the first commercial building in the U.S. to use them.

The devices are custom designed for each project. For Wells Fargo, they consisted of five 1½" steel plates with an overall height of 19" and a total weight of 1,300 lbs. The plates were clamped together and bolted to the structure using 2½" diameter A490 bolts each torqued to a pretension force in excess of 200 kips. The design yield force for the devices was 150 kips, dictated by considerations which included the seismic capacity of existing elements at the parking level and the need to limit story drifts, and thus the seismic demands, on the non-ductile concrete columns that form the vertical load-carrying system for the building.

The chevron braces consist of 10" x 10" tubes. Tubes were utilized instead of wide flange shapes both for aesthetics and because tubes have some capacity out of plane. In addition, 2" pipe tubes were used as lateral braces to prevent torquing by the weighty ADAS device.

Dames & Moore used the SAP90



Both the design team and Wells Fargo's project manager agreed that any attempts to camouflage the structural elements would be more obtrusive than the braces themselves, so the structural elements were left exposed.

program from Computers & Structures, Inc., to perform linear analysis and for non-linear analysis DRAIN-2DX, which is distributed by NISEE for the Earthquake Engineering Research Center at the University of California (Berkeley). These analyses revealed that the behavior of the structure was stable and that the maximum deformations of the structure were within acceptable limits consistent with the bank's desire for "damage control." They also revealed, however, that some elements of the existing building would be overstressed when subjected to large earthquake forces. As a result, several additional strengthening procedures were undertaken, including: adding steel jackets around the two interior columns restrained by the mezzanine floor and around all four interior columns at the second floor; the upgrade of the existing shear wall along the west side of the building; and the strengthening of the two concrete columns in the basement directly below the west wall.

Architectural Treatment

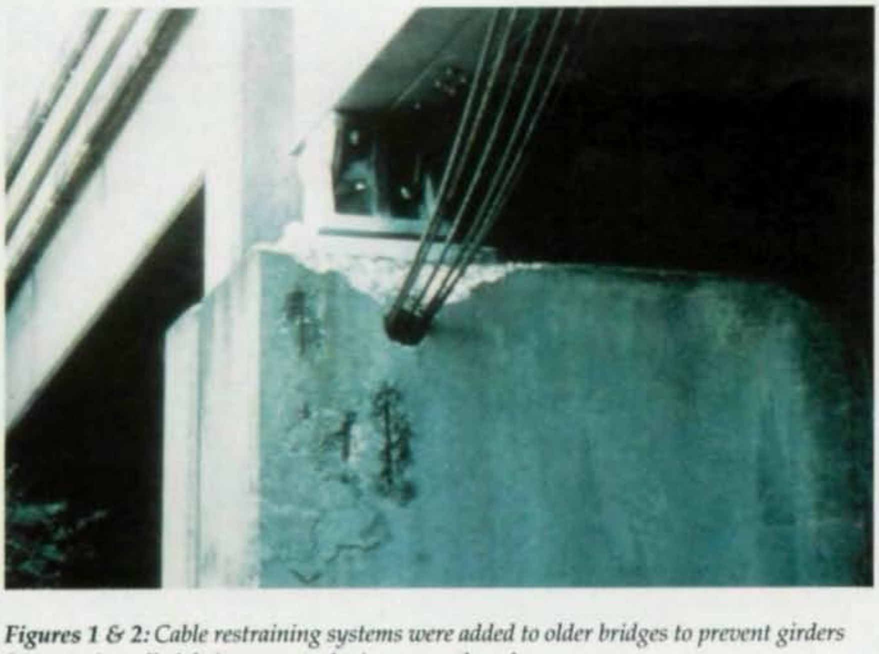
One of the important architectural issues for the project involved the visual impact of the seismic upgrades on the bank lobby. The combination of TS 10 x 10 chevron braces and columns, ADAS elements, and the connection hardware are all highly visible due to their location directly in front of full-height plate glass windows. Both the design team and Wells Fargo's project manager agreed that any attempts to camouflage the structural elements would be more obtrusive than the braces themselves, so the structural elements were left exposed. Architect on the project was Whisler-Patri and contractor was Dinwiddie Construction Co., both of San Francisco.

> The project team agreed that any attempts to camouflage the structural elements would be more obtrusive than the braces themselves

Eduardo Fierro, P.E., Cynthia L. Perry, P.E., and Thomas R. Varner, S.E., all are associates at Dames & Moore, an environmental and consulting engineering firm headquartered in Los Angeles. All three are part of the Earthquake Engineering Group at Dames & Moore and work in the firm's Oakland office. ■

Sharing California's Seismic Lessons

As municipalities outside of California become more earthquake-conscious, their highway officials can learn important retrofit lessons from Caltrans



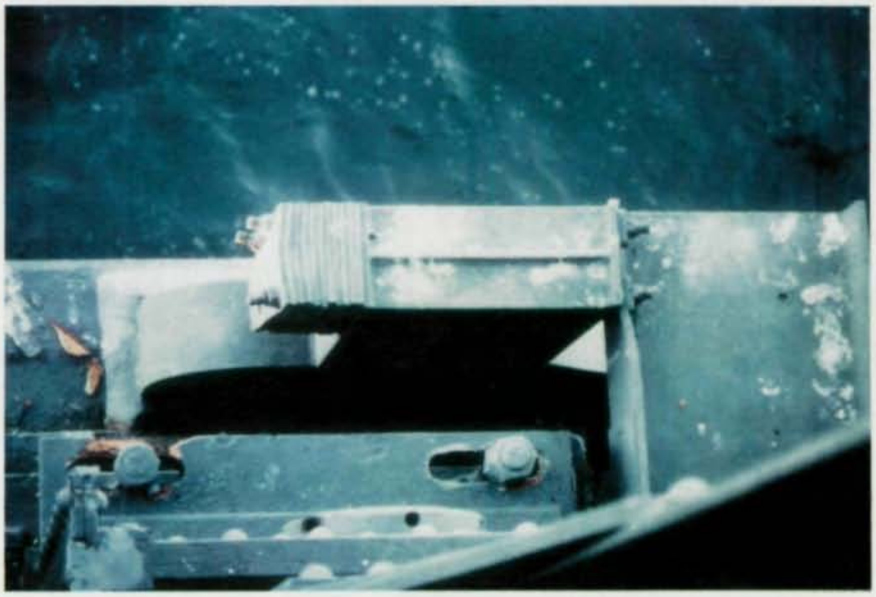
Figures 1 & 2: Cable restraining systems were added to older bridges to prevent girders from coming off of their supports during an earthquake.

By James E. Roberts, P.E.

While structural steel bridges have historically performed well during moderate earthquakes in California, most were designed prior to the development of modern seismic codes in the early 1970s. As a result, most were designed to withstand seismic forces of 0.06 g or 6% of the contributing dead load acting laterally at the deck level, as compared with current seismic requirements that are typically eight to 10 times as great in most California locations and even greater in severe seismic zones.

However, even with such low seismic design forces, most damage to steel bridges during earthquakes have been limited to the substructure and to the superstructure joints and connection details. Examination of damage to structural steel bridges after recent earthquakes demonstrates the need for continuous bridges or for seismic retrofit details to tie superstructure joints together, as well as tieing together the superstructures to the supporting substructures. For example, the most severe damage to a structural steel bridge during the 1971 San Fernando earthquake occurred when the girders came off their supports and fell because they were not tied down. In contrast, performance of the same type of bridge in the 1989 Loma Prieta earthquake was significantly improved as a result of the addition of cable restraining systems to tie the girders to the substructure (see Figures 1

Figures 3, 3A, & 4: *Figure 3 shows the damaged truss end post/lower chord bearing of the Bay Bridge after the Loma Prieta earthquake. Figure 3A shows a damaged backup bumper system on the same bridge. Figure 4 shows a typical seismic retrofit collar on an elevated viaduct.*



and 2).

While structural steel is inherently a ductile material, it must be designed to resist the highest expected seismic forces or retrofitted to resist those forces. Additionally, the large relative displacements resulting from ductile columns going plastic during a seismic event must be considered in the initial design or retrofit scheme. The analysis of 11 major bay; and river crossings in California is now underway and retrofit designs will be initiated in 1992 and contract work will begin in 1993.

Performance And Damage

San Francisco-Oakland Bay Bridge

The San Francisco-Oakland Bay Bridge was designed in the early 1930s for a seismic force of 7.5% to 10% g. It is a tall column supported structure providing navigation clearance for capital ships. This height is both a benefit—because the steel towers are flexible—and a problem—because they are all of different heights, which causes non-uniform movements. It has been determined that the damage at Pier E-9 during the Loma Prieta earthquake was caused by the differential movement of these varying height towers. Figure 3 shows the damaged truss end post/lower chord bearing where 40 1"-diameter anchor bolts failed in shear and the east truss moved

Figures 5 & 6: Figure 5 shows the north end of the main girder span of the Van Duzen River bridge after the April 25-26, 1992 tremors. Note that the restrainer cables on the left side of the girder is slack, while the cables on the right side are taut, indicating a movement to the left. Figure 6 shows a buckling of the lateral bracing system on the same bridge.

7" away from the west truss during the 10 to 15 second shaking period.

At other locations on this bridge, evidence of joint movement as low as 5" and up 12" was observed. Investigators have calculated that a force of more than 2 million lbs. was required to shear all the anchor bolts simultaneously and break the friction force between the bearings and the top of the steel tower.

During the seismic event the 50' closure spans on the upper deck came off their sliding bearings and dropped onto the lower deck. Despite the publicity at the time, this was the only major damage to this 5½-mile-long structural steel bridge. Damage was limited because Caltrans had completed the first phase of a billion dollar multiphase seismic retrofitting program for older bridges on the State Highway System. The retrofitting consisted of restrainer cables at deck joints (similar to those shown in Figure 1) and backup bumper systems constructed around the sliding bearings on the easterly ends of all the truss spans. These bumper systems are intended to keep the sliding bearings of truss spans from dropping off the pier cap. During the earthquake, these systems performed well as evidenced by minor damage to the bumpers—damage that was repaired by replacement of individual elements of the bumper system (see Figure 3A).

Elevated Viaducts

Several structural steel viaducts were in the area close to the Cypress Street viaduct and the Bay Bridge. The damage was typically limited to columns and bearings.

Figure 4 shows an example of a typical seismic retrofit collar that had been installed on all the columns of the West Grand Avenue Viaduct that connects the Port of Oakland with the Bay Bridge near the Toll Plaza. While this structure suffered considerable damage to non-ductile concrete columns, superstructure damage was limited due to these retrofitted collars. Most of the damage was repaired while the superstructure was supported on falsework and traffic continued to use the structure. This was also the case on the viaducts in the interchange at the east end of the Bay Bridge, where Figures 1 and 2 originate. Contracts are now underway to reinforce the footings and construct steel jackets around the columns for improved ductility and seismic performance.

Eel River and Van Duzen River Bridges

On April 25-26, 1992, three consecutive tremors hit the Humboldt County area south of Eureka, CA, with Richter Magnitude Scales of 6.9, 6.5, and 6.0 between 11 a.m. on the 25th and 4:20 a.m. on the 26th. This area had previously been heavily instrumented by the California Division of Mines and geology, Strong Motion Instrumentation Program (SMIP), so excellent data on ground acceleration and structure response was obtained. Major damage consisted primarily of landslides and houses sliding off foundations; no State Highways were closed. Bridge damage was limited to spalling of deck joints caused by deck segments banging together, some minor cracking of abutment walls, and buckling of vertical and horizontal members of the wind bracing systems.

There are a large number of structure steel plate girder bridges and one through-truss over the Eel River near the epicenter of the first shock, a 6.9 Richter Scale event with a north-south orientation—the same orientation as most of these bridges. Bridge inspection engineers reported minor damage after that first event, and only minor additional damage after the next two tremors—nothing serious enough to warrant the closing of any bridges. This success story is a testimonial to the superstructure retrofit program completed in 1989.

Figure 5 shows the north end of the main girder span of the Van Duzen River bridge after the tremors. The entire superstructure moved northward about 1½" on the bearings. Note that the restrainer cables on the north end of

Figures 7 & 8: Figure 7 is another view of the buckling of the lateral bracing on the Van Duzen River bridge after the April 25-26, 1992 tremors. This buckling was evident on several river crossings after the seismic event. Figure 8 shows the installation of a structural steel jacket on an older, non-ductile concrete column.

the girders are slack and those on the south end of the shorter span are tight, indicating a movement to the north. This is typical of all the large River crossings in the affected area. Figures 6 and 7 show the only damage to the bridge, a buckling of the lateral bracing systems. This also was typical of all the girder spans on the several bridges that were inspected after the tremors.

While this buckling is evidence of tremendous movements during the seismic events, there is little evidence of any other damage to these bridges. An instrumented bridge approximately five miles south of this bridge recorded an acceleration at the deck level of 1.25 g. The only other damage to bridges in the area was settlement of approach embankments ranging from 4" to 6". This is typical of earthquake damage in California and is readily repaired by state maintenance crews within hours of the events.

The Eel River Bridge at Robinson's Ferry, about 3 miles south of the Van Duzen River Bridge, had been retrofitted in 1988 with base isolation bearings on its two 300'-through-truss spans. By reducing the base shear forces that could be transmitted from the superstructure, this retrofit detail precluded the necessity of upgrading the older piers. These bearings performed well and damage, as expected, was limited to spalling at the joints. This was the first test of a base isolation system on a bridge in California and demonstrates its validity as a solution for older, hard to repair piers. Caltrans plans to systematically replace existing rocker bearings with neoprene pads and base isolation systems as it implements later phases of the bridge seismic retrofit program.

Steel Jackets For Concrete Columns

While hinge and joint restrainers performed well in several moderate earthquakes, shear failure of columns on the I-605/I-5 grade separation bridge during the moderate Whittier Earthquake of October 1, 1987, reemphasized the inadequacies of pre-1971 column designs. Even though there was no collapse, the extensive damage resulted in plans for basic research into practical methods of retrofitting bridge columns on the existing pre-1971 bridges. The research program was initiated in early 1987 and is currently being conducted at the University of California at San Diego. Funding levels for implementation were increased four-fold after the Whittier earthquake. After the Loma Prieta earthquake the research funding was increased by a factor of 16 and several additional university research facilities were engaged to assist in this massive program.

One possible retrofit method for increasing concrete column ductility is the installation of confining jackets. These external jackets are primarily structural steel but Caltrans has tested and will begin using fiberglass wrapped blankets and possibly prestressing strand if construction details for wrapping around small diameter members can be developed.

For the jackets to work, on single column supported bents the columns and footings must both be retrofitted to provide resistance to overturning from lateral forces. Most footings on the older structures must be rebuilt with the addition of top reinforcing steel mats and additional piles or soil anchors to provide the required additional overturning resistance.

In many cases, "super bents" must be constructed in areas that are accessible and out of the way of traffic. These bents are designed to take most of the lateral forces and preclude or reduce the need for retrofit on columns near traffic lanes. Figure 8 shows the installation of a structural steel jacket on an older non-ductile concrete column in the Los Angeles area. Light erection equipment and off-site fabrication make this retrofit detail a very competitive retrofit solution.

James E. Roberts, P.E., is Chief, Division of Structures, for the California Department of Transportation.

Seismic

Crucial for any analysis of structural steel bridges is agreement on basic seismic design principals and nomenclature. What follows is a brief review of the terminology and design philosophy used by Caltrans.

Earthquake Magnitude Scale (Richter Scale) vs. Peak Ground Acceleration

The Richter Magnitude Scale is a measure of the total energy released in a particular earthquake. Its significance is that it is a function of the length, width and permanent displacement of rupture on the fault. Seismologists have some limited capability to estimate the potential ruptures and energy releases or magnitudes for faults. For practical purposes, most engineers design for the maximum credible event effect at a site. In general, the higher magnitude earthquake will generate a larger peak acceleration, a longer duration shaking period, and cause damage over a more widespread area. Most faults in the U.S. are capable of generating moderate earthquakes and they typically shake for about 15 to 25 seconds even though their duration varies. The major fault offshore from Northern California to Alaska is a "subduction fault" where the adjacent tectonic plates have overlapped and there is a potential for very large earthquake energy releases and a long duration of shaking—resulting in

Design Considerations

great damage and soil liquefaction. Some experts predict this level of damage in the Midwest from a large earthquake on the New Madrid fault zone in Missouri, even though that fault is not on a subduction zone.

Peak Ground Acceleration is an important factor in seismic design because it is a measure of the rock acceleration below the structure. Peak Ground Acceleration data can be obtained from ground shaking maps, which are prepared by compiling seismogenic faults, estimating their maximum credible earthquake magnitudes, and then relating the magnitudes to ground shaking parameters, such as peak acceleration. Peak acceleration dissipates with distance from the fault rupture and sometimes is also reduced by energy absorption of the soils overlying the bedrock. Maps of predicted peak rock acceleration for the U.S. are included in the AASHTO Seismic Design Specifications.

Acceleration at Ground Surface

On most harder soils and alluviums the incoming seismic energy imparted by the bedrock acceleration is absorbed and the acceleration at the surface usually is reduced. The reduction is dependent partly on the depth and type of material over the bedrock. ARS curves [a combination of expected peak acceleration on rock (A), the associated bedrock response spectra (R), and the soil amplification ratio (S)] must be developed for various typical site conditions. Deep, soft soils tend to amplify the accelerations and are of critical importance to tall, slender columns and longer period structures. Structures built on harder soils must be designed to withstand high frequency vibrations with small amplitudes while those built on deep, soft soils and mud tend to be subjected to lower frequency vibration with larger amplitudes. This latter case was interpreted to be a major cause of damage to many buildings and some bridges in the Loma Prieta earthquake in 1989.

The terms "Near Field" and "Far Field" effects are used by seismologists and geotechnical engineers in describing nearby and distant earthquake ground shaking effects on structures. Even distant but large earthquakes can be damaging, as was the case in the 1985 Mexico City earthquake. This means that one must consider all the distant and nearby faults and design for the worst effects that will cause damage to the structure. For example, in some parts of California the designer must consider the effects from perhaps five or six faults (nearby and distant) to determine the appropriate design earthquake. In the April, 1992, Humboldt County earthquakes, more bridge damage was caused by the last two of the three tremors, even though they were offshore and of lower Richter Magnitude Scale.

Performance Criteria

An agency or designer must have a performance criteria established. What do you want the structure to do in an earthquake? The current California Performance Criteria are No Collapse, No Major Damage, No Secondary Injuries or Fatalities Because Emergency Equipment Cannot Get Through, Major Important Structures and Routes Must Remain Operational. These criteria are generally attainable, though the last one can be expensive if structures are expected to withstand severe earthquakes such as a 45 second shake for large earthquakes on a major fault.

Seismic Design Principles

Continuity is extremely important and is the easiest and cheapest insurance obtainable. If structures are not continuous and monolithic, they must *be tied together at deck joints,* supports and abutments. This will prevent them from pulling apart and collapsing during an earthquake. Performance of seismically designed and seismically retrofitted structural steel bridges in the most recent California earthquakes prove that adherence to this principle will provide safe bridges.

Ductility in the substructure elements is the second key design consideration. It is important to note that when you design for ductility you accept some damage during an earthquake, but collapse or major damage is unacceptable. The secret to a good seismic design is to balance acceptable damage levels with the economics of preventing or limiting the damage. Properly designed ductile steel structures will perform well during earthquakes as long as the design has accounted for the movements and controlled them or provided for movement at abutments and hinge joints.

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Computers And Steel Design

William McGuire, P.E., is the recipient of the T.R. Higgins Lectureship Award and a professor emeritus of structural engineering at Cornell University. This article is condensed from his lecture. The entire text will be printed in a future issue of Engineering Journal. Also, McGuire will present his paper six times in the next 12 months, including St. Louis on October 14, Kansas City on October 15, and San Antonio on October 16. Also scheduled are fall presentations in Chicago and New York.

A solid education in structural design is needed to avoid the misuse of computer-aided engineering

Computer-aided structural engineering is no longer an idea that has to be sold. Its widespread use in planning, analysis, element design, detailing, and the control of fabrication shows broad acceptance. Nevertheless, this is still a critical time for the medium. Further development is needed, and the professional magazines that are full of ads describing the wonderful things that can be done on the computer too often contain disturbing accounts of its serious misuse.

State-of-the-art computer programs are very powerful.

- Today's commercial programs are capable of giving designers voluminous structurally significant graphical information on two- and three-dimensional frames and continua.
- Three-dimensional linear elastic analysis of frames and continua is handled thoroughly.
- Commercial inelastic analysis programs exist but those that include second as well as first order effects still require computer power beyond the reach of most structural engineers, and even these are limited in their treatment of significant behavioral phenomena.
- Commercial programs that integrate analysis and design in a coordinated interactive graphics package exist, but may still be too expensive for small firms.
- The continuing revolutions in hardware and underlying software that have made the advances of the last 15 years possible have the side effect of complicating the tasks of developing and assimilating applications software. The needed fusion of workstation and personal computer technologies is coming, but too slowly. And the variety of operating and graphics systems is an obstacle to program dissemination.

> Some have predicted that a catastrophic failure attributable to computer misuse is only a matter of time

The power of these systems makes critical their proper use, which requires understanding their capabilities and limitations, attention to detail in their implementation, and, above all, sound structural engineering knowledge from beginning to end—from development of the structural model to interpretation of the results.

While these seem like trite precepts, there is ample evidence they are often violated. In a recent *ENR* article (October 28, 1991), leaders in the development and application of computer-aided engineering expressed alarm over the incidence of misuse. They gave numerous examples and some predicted that a catastrophic failure attributable to computer misuse is only a matter of time.

I share their concern, but neither I nor any of them would say the remedy is to ban the computer. Computer misuse has its special characteristics and dangers, but it is just one manifestation of the broader problem that engineers have always faced: Coping with side effects of advancing technology. I suspect each of us has his own gallery of structural horrors. In mine the most frightening examples are abuses of the principles of good weld design and practice. And as with other structural problems, the answer for computer aided engineering is not to halt the advance of technology.

It seems to me that the better computer-aided engineering systems are indeed good and that the fault is in the users and not in the computer. How then do we approach the 21st Century—a time in which technological advances will far surpass anything we have seen so far—with some assurance that structural engineers will partake fully of the advantages of this technology while retaining control of it? I look toward R&D, improved education, further evolution of design specifications, and increased professional responsibility.

Research And Development

Though there is much good research work being done, there are some disturbing trends. Notably, while the sheer volume of university research in computer-aided engineering makes R&D activity look healthier than ever before, this is misleading. Much of the material is beyond the limits of helpfulness—at one end it is too close to theoretical mechanics to have any near-time application in design engineering and at the other end too small a modification of things done before to be of interest.

Another disturbing trend is the reduction in R&D spending. In its latest report, the National Science Board said overall spending on research by the Federal Government, industry, universities and private sponsors slowed during the second half of the 1980s and began to fall in 1989. This is happening at a time when similar investments in Japan and Germany are rising rapidly.

With respect to research in steel structures, and particularly in computer-aided steel design, the picture is every bit as gloomy. In the National Science Foundation, the major supporter of this type of university research, funding for all individual project research related to steel is less than $1.5 million per year.

As technological tools—such as computers—advance, it is essential that theory advances to take advantage of these tools.

Education

That a proper education in structures is the essential prerequisite for the intelligent use of a computerized analysis or design system can't be overemphasized. Other safeguards may be needed, but no design engineer without adequate knowledge of structural analysis and behavior should be permitted to sit down unattended at a computer.

Unfortunately, what constitutes "proper education" and "adequate knowledge" is debatable. During the past decade, there has been a renewed emphasis on undergraduate education in many universities. More professors are spending more time with undergraduates inside and outside the classroom. While this is to be applauded, consideration also must be given to what is being taught, and not just to how it is being taught.

But some topics, such as connections, stability, structural analysis, non-linearity and torsion, can not receive adequate coverage in four undergraduate years. For example, knowledge of the principles of contemporary methods of numerical analysis—matrix and finite element methods in particular—is essential to understanding of computerized analysis and interpretation of results, such as the significance of a reported stress.

> No design engineer without adequate knowledge of structural analysis and behavior should be permitted to sit down unattended at a computer

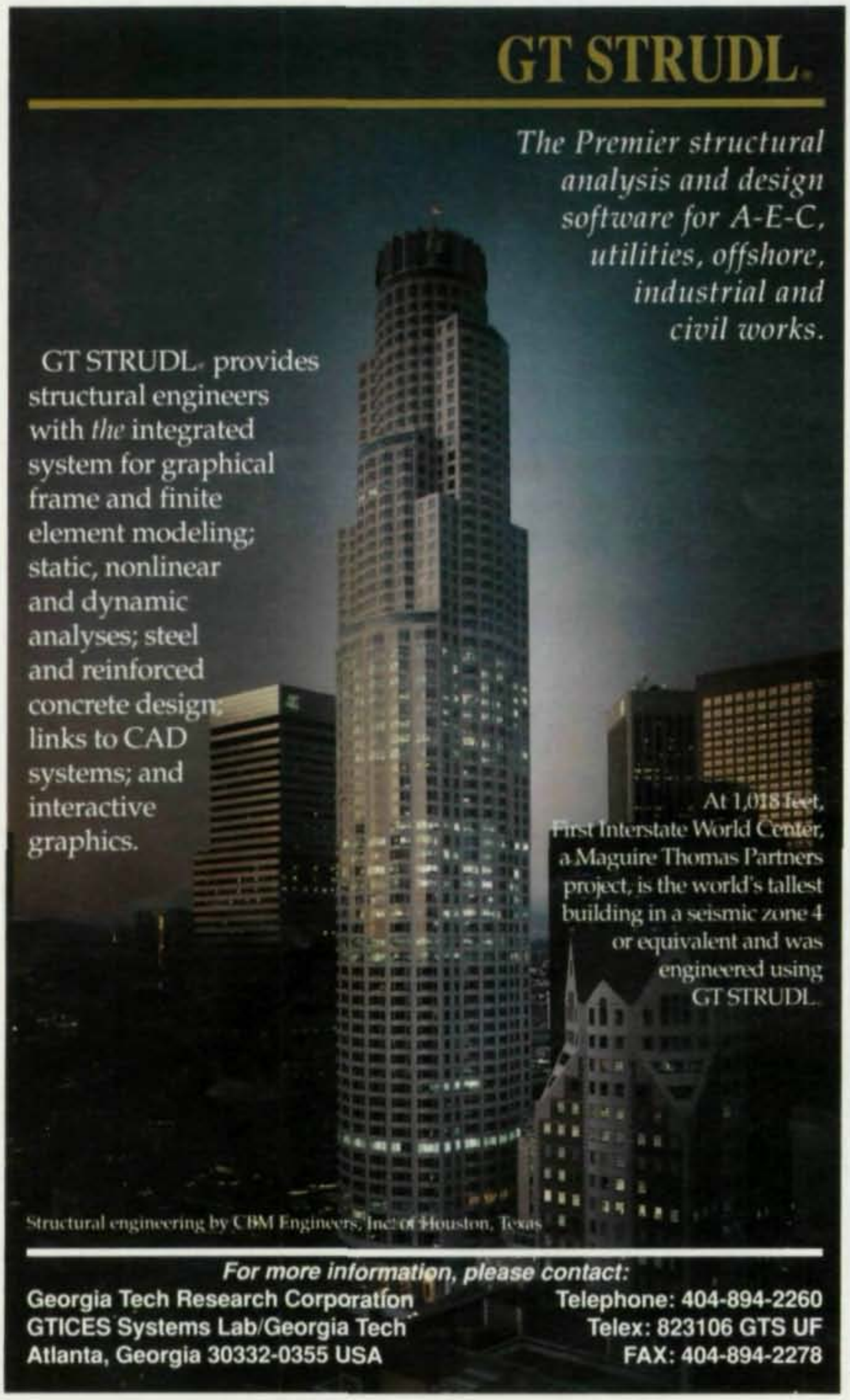
I don't advocate scrapping or modifying present undergraduate programs. But taking advantage and staying in control of our increasingly complex technology requires more. The education of a structural engineer may start in the first four years, but there are topics that are now basic to everyday practice that can only be treated adequately in professional and graduate programs. Extension of basic engineering education to this higher level would not cure all the problems of computer misuse or, for that matter, poor weld design. However, it would help, and at the same time the general quality of design would improve.

Design Specifications

According to Hardy Cross (*Engineers and Ivory Towers*, McGraw-Hill, 1952), "As the size and complexity of projects increased, the time came when there was more work to do than men to do it or time in which to think out problems. It became desirable and even necessary to set up a series of routine procedures for analysis and design. This meant the development of a series of formulas and rules and standards which could be followed within limits by men trained in that vocation."

He noted that there appeared, in effect, an intellectual "assembly line" without which it would be impossible for engineers to turn out the needed volume of work. But, he added, "much of the most important work cannot be done by using fixed rules, standardized formulas or rigid methods."

Standards—and I include steel design specifications—are not everything, and they must be changed over time. Today, computer-aided engineering is aiding us to move away from relying on "K factors" for estimating effects of member interaction and towards practical nonlinear methods. Right now, second order elastic analyses programs that eliminate the need to calculate B1 and B2 factors and their associated effective length coefficients are available. I wish more engineers would use them. But there are other places where effective lengths are still the best, or only, practical expedient for routine design, though future research may change this.



Professional Responsibility

Everything I've said so far that relates to human problems in the use of technology also involves professional responsibility: the idea that, regardless of an engineer's place in the scheme of things, he has the responsibility to apply sound technology intelligently and conscientiously, and to stand behind the results.

This notion includes technical competency, which means knowing how to do the job one takes on at any stage in an individual's career.

In this present business and legal climate, this concept of professional responsibility may sound as idealistic and impossible to live up to as a Victorian code of conduct. Well, it always has been. Even in simpler times every engineer has on occasion been wrong. But that is not the point here.

It is simply that awareness of one's professional responsibility is all important. It means asking at the start of a job: "Do I know how to do it, will I take the time to do it decently, and if I fail, will I be ready to accept the consequences." Such introspection should not be an exercise in timidity, but it may be all that is needed to avoid heading into trouble. As it applies to the latest technology, it can make the difference between using a sophisticated computer system without proper preparation, taking the time to understand it and learn how to use it properly, or falling back on less advanced but sound methods that you know well. ■

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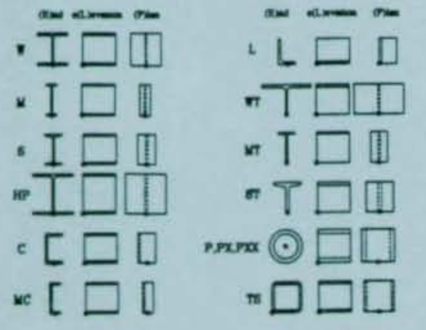

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Available on 3" or 5" disk. ***$495.00***

M-Strudl

M-Strudl, developed and supported by C.A.S.T., is a general purpose structural analysis/design program. The program is capable of solving 5,400 3D joints, 16,000 members, and 8,000 plate/shell elements for both static and dynamic (RSA and/or time history) problems with almost an unlimited number of load cases and mode shapes on computers using either a DOS or OS2 operating system. It also is available on UNIX operating systems, where the program's capabilities are even further enhanced. M-Strudl features a number of special commands; for example, an extensive EDIT command enables the user to duplicate frames (vertical or horizontal) to different locations, while a DISTRIBUTED LOAD command enables the user to specify area load. A Windows version will be available late in 1992.

The cost of the program is $340.

For more information, contact: Ta Chang at (510) 226-8857; fax (510) 226-7328.

FRAME mac

By taking advantage of the superior graphics of the Macintosh computer, Compuneering's FRAME mac program achieves accurate results with minimal user effort. The program also allows users to quickly consider a wide range of alternatives. For example, by combining FRAME mac and MacShapes, one user cut his outside consultant fees by $3,500 on one project by doing the work himself in only 30 minutes.

The cost of the program is $595.

For more information, contact: Dorete Sivkin at (416) 738-4601; fax (416) 738-5207.

COSMOS/M

Structural Research & Analysis Corp.'s COSMOS/M is a full function, easy-to-use finite element analysis (FEA) software program. The Basic System offers the user a comprehensive statics and dynamics analysis capability as well as easy to use pre- and post-processor. Additionally, the program offers a Steel Design Module (SDM) that performs checking of structural steel members modeled and analyzed by COSMOS/M. As a result, users can utilize the program's AISC ASD code checking capability while designing their structures. In addition to its DOS versions, the company has recently introduced a Macintosh version.

For more information, contact: Structural Research & Analysis Corp., 1661 Lincoln Blvd., Suite 200, Santa Monica, CA 90404 (213) 452-2158; fax (213) 399-6421.

MERLIN DASH

Version 4.5 of MERLIN DASH (Design Analysis of Straight Highway Bridge Systems) is now available from OPTI-MATE. The program is fast with an extremely user-friendly menu driven input. Output is complete, tabular and well organized. The program will consider welded plate and rolled beam sections, non-composite or composite, continuous up to 10 spans. Automatic live loading includes HS and multiple user defined vehicles (which may be stored in a truck file). AASHTO destruction and impact factors are computed but may be overridden. A Code Check is performed for both LFD and WSD.

Also available is DESCUS On The PC For Curved Bridges. These programs are identical to the DESCUS programs used for years by many state DOTs and consultants for the analysis and design of curved "I" and "Box" girder bridge systems—except they run on 386/486-based PCs rather than on mainframes accessed through timesharing. Live loading is automatic, structures may be skewed, bifurcated, non-composite or composite, and continuous up to 8 spans. An AASHTO Code Check is performed at each design point indicating compliance with specifications.

For more information, contact: Ollie Weber, OPTI-MATE, Inc., P.O. Box 9097, Dept. A1, Bethlehem, PA 18018 (215) 867-4077.

RISA-2D

The popular RISA-2D program provides a fast, truly interactive environment for the solution of a wide range of structural design problems. RISA-2D can easily handle frames, trusses, shear walls, continuous beams and much more. Statics and dynamics are included, with full steel design (including member selection). Customized spreadsheet editing combined with powerful graphics to simplify use.

A free demo disk with documentation is available.For more information, contact: RISA Technologies, 26212 Dimension Dr., Suite 200, Lake Forest, CA 92630 (800) 332-7472.

LARSA

Innovative Analysis Corp.'s LARSA—Advanced Structural and Earthquake Engineering software is now available for use on Macintosh computers, in addition to the already existing versions for 386/486 DOS, 286 DOS and VAX VMS based computers. The new Macintosh version uses a 3D graphical interface with pull down and pop-up menus and dialogue boxes. The graphical point and click environment simplifies the use of the program's most advanced features. LARSA analysis options include linear & non-linear static, eigenvalue, response spectra and time-history analysis. The program supports AISC codes for steel.

For more information, contact: Innovative Analysis Inc., 330 West 42nd St., New York, NY 10036 (800) LARSA-85.

GT STRUDL

Version 91.01 of GT STRUDL is now available on the following systems: DEC VAX under VMS, IBM RISC System/6000 under AIX, Intergraph Clipper workstations under CLIX, SUN SPARC and SunOS, and PCs under DOS. Enhancements include: GT MODELER, a menu-driven, interactive graphical modeling environment; API code checking; fatigue analysis and platform launch simulation; composite modal damping; and elastic buckling analysis. In addition, interfaces between GT STRUDL and several CAD systems are available: CADAM AEC 3D Structures; Frameworks from the AEC Group; GDS from McDonnell Douglas; MicasPlus and I/FEM from Intergraph; Patran from PDA; PDMS from CadCentre; and AutoCAD from Autodesk.

For more information, contact: Michael T. Lee, Software Marketing Manager, GTICES Systems Laboratory, at (404) 89402260; fax (404) 894-2278.

MicasPlus

Intergraph's MicasPlus consists of a tightly integrated set of modules (Design, Analysis, ModelDraft, and InterSect) and offers modeling, analysis, postprocessing, design and drawing production tools for structural engineering projects. Automating the entire structural engineering workflow and blending graphics and database management, MicasPlus is suited for commercial and residential building, light and heavy industry, transportation and utility projects. The ModelDraft module provides 3D structural layout, drawing extraction, and revision tracking. The Analysis module is a finite-element modeling and analysis application where engineers can layout 3D frame and finite element models and analyze them. Through integration with ModelDraft, the Analysis module can use previously defined geometry to define the analytical model, eliminating redundant data entry. By directly evaluating results from the Analysis module, the Design module automatically optimizes structural member sizing based on user-controlled parameters. Aiding the process, the InterSect module contains a comprehensive library of standard structural steel cross-sections from American and international steel tables. The InterSect module is available as part of MicasPlus or separately.

For more information, contact: Laurie Harris, Intergraph Corp., Huntsville, AL 35894-0001 (205) 730-2000; fax (205) 730-2461.

ASG Architectural & Structural

System ASG is a series of software programs for all aspects of the construction industry. ASG Core is the foundation for the system. Its a valuable utility and can be used to customize an AutoCAD environment, as well as to run a wide range of ASG applications, including: ASG Architectural, which is designed to enhance AutoCAD productivity in architectural design and drafting; ASG/Vertex Detailer, which enables the user to assemble building details from more than 25,000 components, rather than drawing them by hand; and ASG Structural, which quickly produces accurate structural plans and details. Included in ASG Structural are structural frames, rolled steel shapes, miscellaneous steel members, and automatic weld notes, as well as metal decking and other structural materials.

For more information, contact: ASG, 4000 Bridgeway, Suite 309, Sausalito, CA 94965.

TPCFM1

Keck Tech's TPCFM1 is an engineering program that can compute the flexural and/or torsional cross-sectional properties of virtually any shape in 10 minutes or less, from start to finish. It breaks a cross-section into many short line elements within a Cartesian coordinate system. Line elements of zero thickness can represent holes or punch-outs in the cross-section, or be connecting lines such as between bolts. Bolts or screws themselves are treated as square line elements whose thicknesses are equal to their lengths. Output includes: location of the center of gravity and the shear center within the Cartesian coordinate system; the moments of inertia, section moduli and radii of gyration about the X- and Y-axes; and the moments of inertia and radii of gyration about the principal axes, plus location of these axes from the X-axes; the product of inertia; and the polar moments of inertia about the center of gravity and shear center.

For more information, contact: Keck Tech, Inc., 286½ East State St., Salem, OH 44460 (216) 332-1132; fax (216) 332-0556.

Composite Steel Girder Design

In addition to existing capabilities for designing and rating multi-span girders, Version 5.0 of MDX's AASHTO Composite Steel Girder Design program includes interactive graphical output for viewing and plotting stresses and deflections. MDX licenses both load factor and working stress versions of its DOS-based bridge girder design programs. Qualified bridge design firms may receive a free four month trial.

For more information, contact: MDX at (314) 446-3221; fax (314) 446-3278.

SAP90 & ETABS

Computers and Structures, Inc., has released new versions of both SAP90 and ETABS. SAP90 is based on modern equation and eign/ritz solving techniques and new finite element formulations. The program now includes P-Delta effects for either static or dynamic analysis. The ETABS System is a series of large capacity programs specifically developed for three-dimensional analysis and design of building structures. ETABS can analyze moment frame, braced frame or shear wall buildings, or combinations of these. Dead, live, wind, static seismic and/or dynamic earthquake load analysis (including time history) are all possible.

STEELER V5.3, a post-processor program for ETABS, now supports LRFD, as well ASD, UBC and the Canadian CAN/CSA code. In addition, the program provides the capability to perform automated stress checks of Eccentrically Braced Frames. The design post-processor SAPSTL V5.4 for SAP90 now offers an option for LRFD. Sway and non-sway load conditions are differentiated and their moments separately magnified.

Also, Version 2.0 of AutoFLOOR, the AutoCAD-based program for automated drafting, analysis, and design of steel floor framing systems, is now available. This release allows all design options to be performed without leaving AutoCAD. Composite and non-composite design options are available for ASD and LRFD. A new feature called "Design-One" permits beams to be designed interactively on a member-by-member basis, with the capability to review loads, forces, stresses and vibrations.

For more information, contact: Computers and Structures, Inc., 1995 University Ave., Berkeley, CA 94704 (510) 845-2177; fax (510) 845-4096.

STAAD-III/ISDS

Research Engineer's STAAD-III/ISDS is an integrated software system for structural analysis, design and drafting first introduced in 1978. Analysis facilities include 2D/3D static/dynamic/seismic/P-Delta analysis, frame/plate/shell elements and all possible loading support conditions. Extensive load generation facilities are available including moving loads (AASHTO and user provided), UBC seismic loads, wind loads, and floor loads. Interactive model generation and elaborate verification capabilities, such as plotting of structure geometry, deflected shapes, bending moment/shear force diagrams and stress contours, are available. STAAD-III/ISDS is fully integrated with AutoSTAAD, an AutoCAD-based structural drafting and model generation software. Structural drafting facilities include generation of framing plans, sections, elevations, and foundation plans. Hardware platforms supported include 286/386/486 PCs, workstations (SUN, HP/Apollo, DEC, IBM, Silicon Graphics, Intergraph) and mainframes (IBM, Prime, DEC).

For more information, contact: Research Engineers, Inc., 1570 N. Batavia St., Orange, CA 92667 (714) 974-2500; fax (714) 974-4771.

Structural Optimization Design and Analysis

A Windows version of Waterloo Engineering Software's SODA (Structural Optimization Design and Analysis) software, which is designed automate the steel design process, is now available. The engineer can start with a sketch, and simply ask the software to find the least-weight solution. The software runs all the necessary reiterations, sparing the engineering from doing all the manual calculations. At the same time, it checks its work against all applicable building codes, including ASD, LRFD and CAN3-S16.1-M89. The software also can display 3D views of the design.

For more information, contact: Waterloo Engineering Software at (519) 885-2450.

RAMSTEEL

Ram Analysis' RAMSTEEL is a specialized structural software program that automates the gravity analysis and design of steel buildings. With a powerful, yet easy-to-use Windows interactive graphical modeler, this PC-based software allows the user to graphically model buildings, floor-by-floor, creating a database of floor loads, slab properties, and member locations. The program calculates tributary loads and live load reduction factors, and selects the optimum beam sizes of either composite or non-composite beam design. Automation of the gravity design provides substantial time savings and increased quality over traditional methods. The program is in strict compliance with AISC, BOCA, SBC and UBC and works with either ASD or LRFD.

For more information, contact: Ram Analysis, 55 Independence Circle, Suite 201, Chico, CA 95926 (916) 895-1402; (916) 895-3544.

Composite Beam Design & Column Design

HESCO offers reliable design programs featuring friendly data entry and comprehensive design. Software includes not only an LRFD composite beam design program and an LRFD steel column design program, but also an ASD composite beam design program. Each program displays default data on the screen for the user to accept or change. The composite beam design programs are applicable both in new building design and in the design of cover plates at bottom flanges in existing buildings when the loads are greater than the capacity of existing beams. The program will handle: uniformly distributed loads; up to 30 concentrated loads; up to 20 partial uniform or varying loads; end moments due to cantilevers; shored and unshored construction; cost of each design; required number of shear studs between maximum and zero moment and stud layout when concentrated loads are preset; camber for unshored construction; final deflection due to total load with long-term effect; and vibration behavior analysis.

For more information, contact: HESCO, 13839 SW Freeway #128, Sugar Land, TX 77478 (713) 545-9820.

ANSYS

Swanson Analysis Systems Inc.'s ANSYS software is an FEA program to help integrate design and analysis. It's design optimization capability enables the user to automatically define a design based on geometric, material, and performance parameter limits.

For more information, contact: Swanson Analysis Systems Inc., Johnson Road, P.O. Box 65, Houston, PA 15342-0065 (412) 746-3304; fax (412) 746-9494.

Softdesk

Softdesk is the world's leading supplier of application software for Autodesk. The Softdesk Structural Engineering Family, includes: Plans & Elevations, which creates accurate construction details with steel, wood, concrete and masonry; and the Steel Detailer, which provides the tools to create shop fabrication erection and bolt setting drawings. In addition, The Modeler provides an environment for creating frame analysis models inside AutoCAD and supports a direct link to industry-standard frame analysis programs.

For more information, contact: Softdesk, 7 Liberty Hill Road, Henniker, NH 03242 (603) 428-3199.

CADVisions

CADVisions has been designated an authorized Softdesk Training Center. Training classes are available at the company's facility in Dallas or at the client's worksite.

For more information, contact: CADVisions, 2200 N. Lamar Suite 104, Dallas, TX 75202 (214) 720-2023.

Design Data

Design Data's SDS/2 Engineering, Analysis and Design package is designed to work with its SDS/2 Detailing, SDS/2 Estimating and SDS/2 Production Control programs. The total package allows engineers, fabricators and detailers the opportunity to communicate more effectively from the start of a project through its completion.

For more information, contact: Design Data, 1033 "O" Street, Suite 324, Lincoln, NE 68508 (800) 443-0782.

AutoDesign and EXPERTISE

During automated structural design, AutoDesign automatically calculates the sizes of all members of the structure, while at the same time satisfying specifications throughout the structure, all within an AutoCAD environment. A unique AISC module is available for the design of steel structures.

EXPERTISE performs seismic hazard analysis and develops peak ground accelerations, site specific response spectra and UBC equivalent static forces. The user is asked to input the zip code and the latitude of the specific site. The software then develops recommendations for the types of seismic analyses for structures and sites.

For more information, contact: Structural Analysis Technologies, Inc., 4677 Old Ironsides Dr., Suite #250, Santa Clara, CA 95054 (408) 496-1120; (408) 496-1196.

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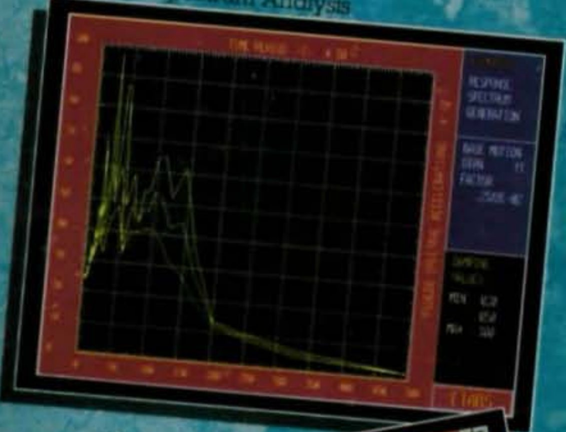
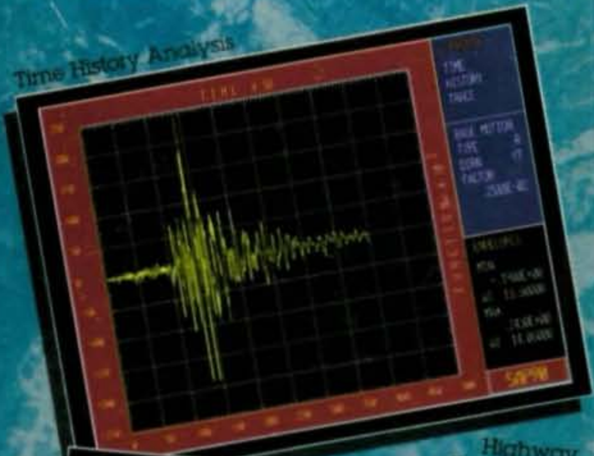
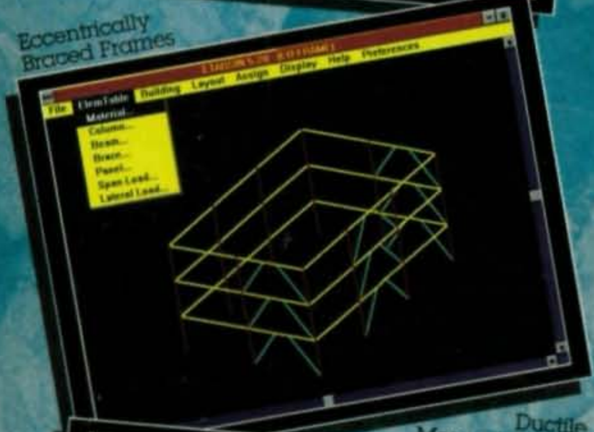


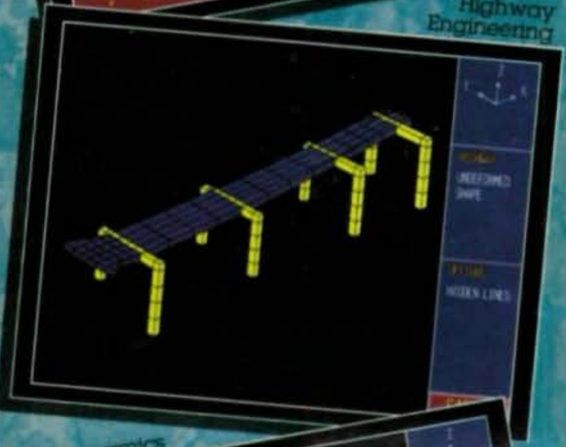
Time History Analysis



Eccentrically Braced Frames



Highway Engineering



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