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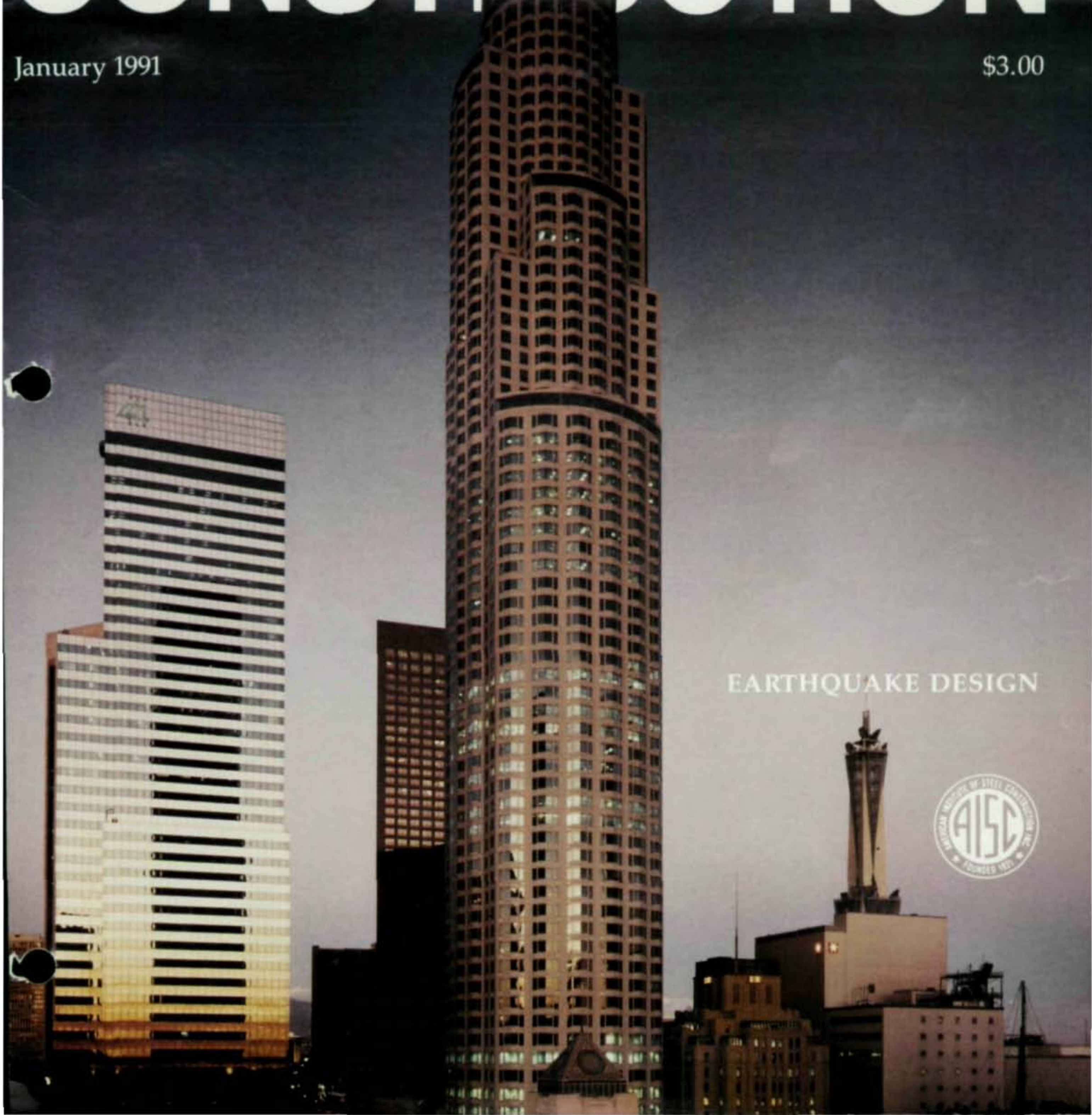
MODERN STEEL CONSTRUCTION

January 1991 $3.00



EARTHQUAKE DESIGN

UNITED STEEL DECK, INC.

DECK DESIGN DATA SHEET
No. 15

DIAPHRAGM INFORMATION

TYPICAL FLOOR DECK WITH CONCRETE DIAPHRAGMS

GAGE	SPAN	WELD PATTERN	SHEAR plf
28	2.0′	30/4	1780
26	4.0′	36/3	1670
24	6.0′	36/3	1700
22	6.0′	36/3	1760
22	8.0′	36/4	1710
20	8.0′	36/4	1750
20	10.0′	36/4	1750
18	8.0′	36/4	1820
18	10.0′	36/4	1820
18	12.0′	36/4	1770
16	10.0′	36/4	1890
16	12.0′	36/4	1830

NOTES

1. THE G′ (STIFFNESS) VALUE OF 2450 KIPS/INCH CAN BE USED FOR ALL COMBINATIONS IN THE TABLE.
2. SIDELAPS ARE WELDED OR SCREWED (28 TO 24 GAGE) AT A MAXIMUM OF 36″ ON SPANS GREATER THAN 5′; i.e. A 6′ SPAN WOULD HAVE ONE SIDELAP ATTACHMENT, AN 8′ SPAN WOULD HAVE TWO.
3. STRENGTH VALUES ARE BASED ON 2.5″ COVER OF NORMAL WEIGHT CONCRETE (f′c = 3 ksi) OVER THE RIBS; FOR LIGHT WEIGHT (STRUCTURAL) CONCRETE MULTIPLY THE TABLE VALUES BY 0.7. DECK DEPTHS UP TO 3″ ARE COVERED.
4. IT MAY BE NECESSARY TO INCREASE THE NUMBER OR STRENGTH OF THE PERIMETER CONNECTIONS TO UTILIZE THE STRENGTH SHOWN IN THE TABLE.

WELD PATTERNS

TYPICAL ROOF DECK DIAPHRAGMS

B DECK WITH ⅝″ WELDS TO STEEL AND #10 SIDELAP SCREWS.

GAGE	SPAN	SIDELAP SCREWS	36/3 PATTERN		36/4 PATTERN		36/7 PATTERN	
			q	G′	q	G′	q	G′
22	6.0′	1	165	7	180	13	240	51
20	6.5′	2	220	12	240	21	310	71
18	7.0′	2	270	22	295	38	380	95

"q" IS POUNDS PER FT. G′ IS IN KIPS PER INCH.

STEEL DECK INSTITUTE *DIAPHRAGM DESIGN MANUAL 2ND EDITION* IS THE BASIS FOR BOTH FLOOR AND ROOF DECK TABLES.

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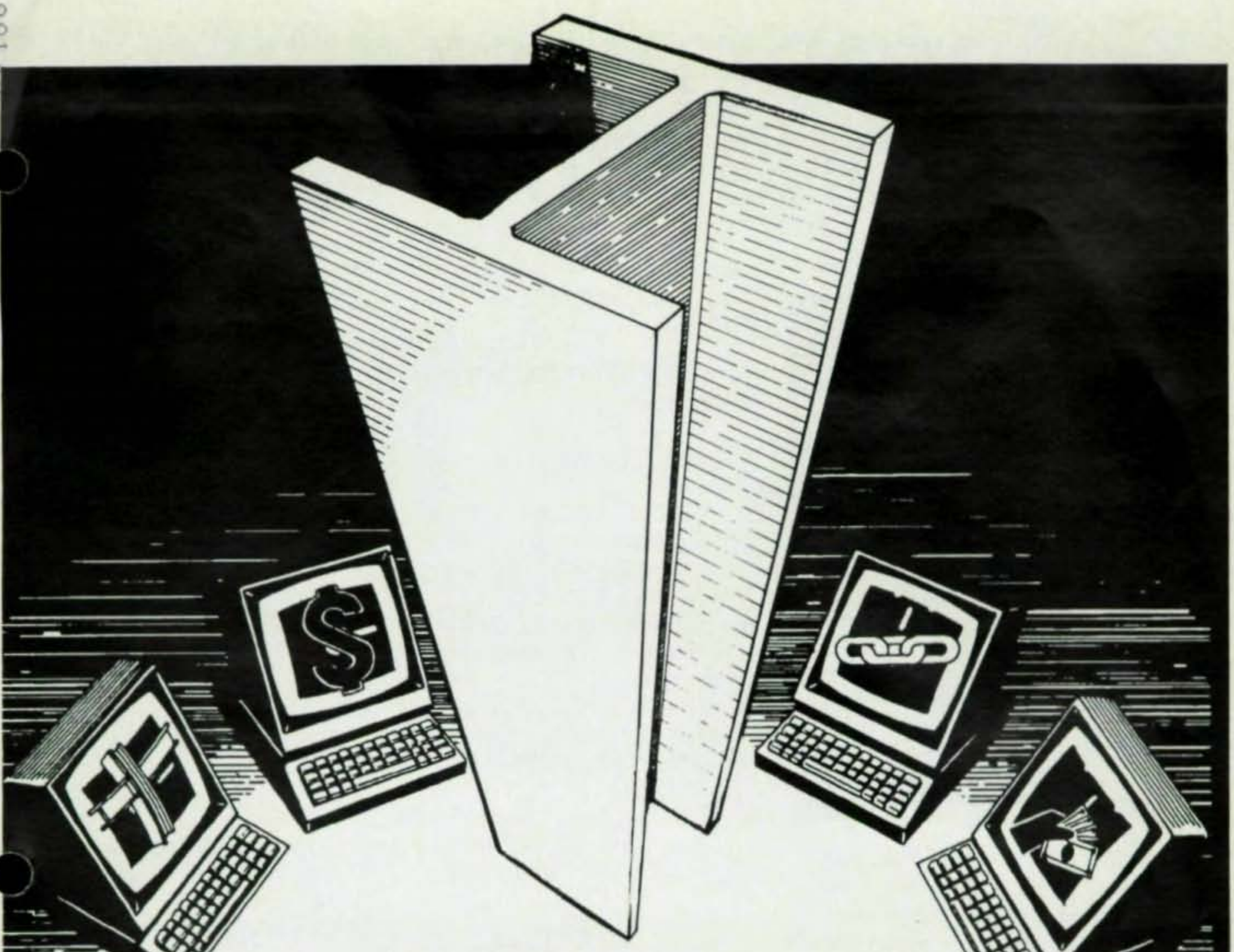

MODERN STEEL CONSTRUCTION

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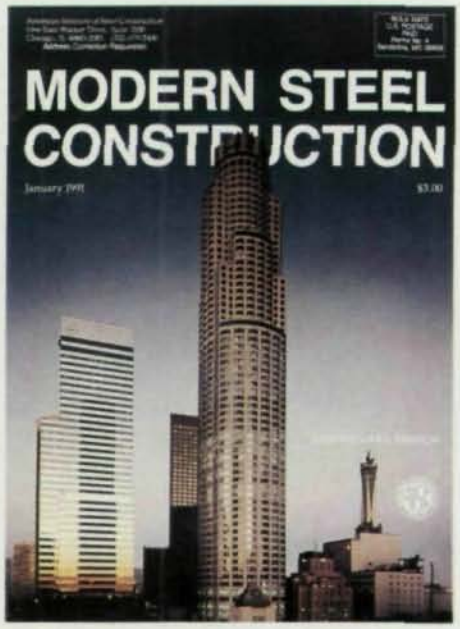

The 1,081'-tall First Interstate World Trade Center in Los Angeles is designed with an uninterrupted braced square spine to resist seismic forces. For more information on spine structures, see the aricle beginning on page 13.

FEATURES

NEWS AND DEPARTMENTS

00150

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Predicting Seismic Safety

Prognosticators are very useful for the public attention they can bring to a subject. Dramatic prophecies—such as expecting an earthquake this past December 3—are sure to capture the media's interest.

In the case of earthquake predictions, the attention generated by a dire prediction is a necessary jolt in parts of the country that have not experienced a seismic disaster in more than a century. For the first time in recent memory, ordinary people outside of California are concerned about the readiness of their built environment to withstand an earthquake.

This awareness is good and may lead to more stringent seismic codes.

The problem develops when no earthquake is forthcoming. When the dramatic predictions prove wrong, people are lulled into a false sense of security and ignore the fact that an earthquake is coming—though we don't know exactly when.

Experts don't predict precisely when an earthquake will occur, only that one is likely to occur within our lifetimes. Some cities in vulnerable locales—such as St. Louis—have begun to prepare. New buildings must meet seismic requirements, and some older buildings are being retrofitted. But other cities—and Memphis comes most notably to mind—have blithely ignored their danger.

It's possible to argue that whether cities such as Memphis provide for the inevitable earthquake is no one's concern but those who live in Memphis. But that is clearly a faulty argument. In addition to any humane person's concern with the loss of life that would occur with a serious quake, the pragmatist must be concerned with the costs involved in rebuilding the city—costs that will be borne by the entire nation, either through insurance fees or direct federal assistance to a disaster area.

These costs—in both money and life—can be minimized by the correct application of seismic design. Engineers throughout the country should be lobbying their local code making authorities to institute the most recent codes applicable in their region. **SM**

Fujitsu Interview

with Dave Hutchinson, Vice President/Structural Engineer, Buehler & Buehler Associates, Sacramento, CA

Q: "Tell me a little about your job at Buehler & Buehler."

A: "At Buehler & Buehler, we are retained by architectural firms to design structural framing for buildings. We create plans for the construction of foundations, columns, beams and bracing. We prepare calculations and material specifications showing actual versus allowable member stresses. The art of what we do is taking a building concept and converting it into practical construction documents. My job is to see that it gets done quickly and efficiently."

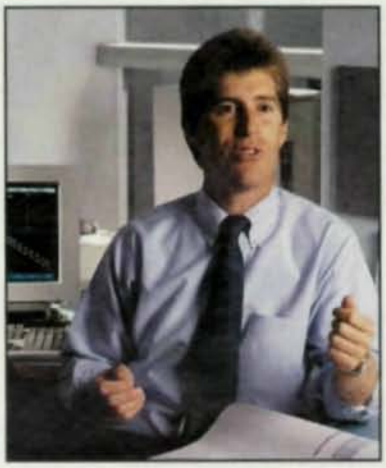
Q: "Why did you select Fujitsu's ElmAnalysis® over other FEA programs?"

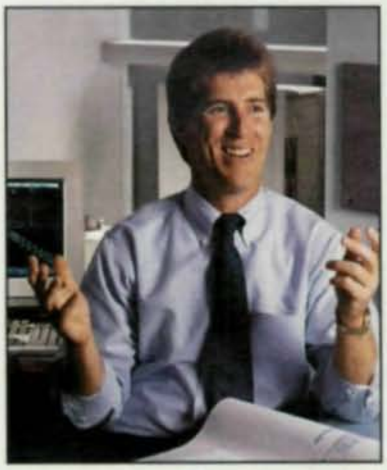
A: "It was the only powerful yet easy-to-use FEA program available on the market. Plain and simple. We required a package that could be used by everyone in the office. We have other FEA programs for specialized projects, but most are so complicated that one person must use them constantly just to stay tuned.

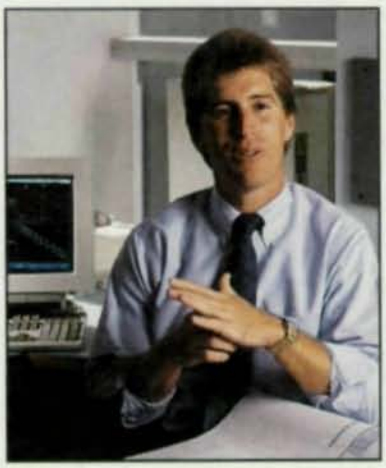
Speed was also critical. Entering data can be a painstaking process that takes hours. ElmAnalysis allows you to create a model of the structure on screen—graphically—in minutes. Which means that you can get more jobs done in less time and with more reliable results."

Q: "What do you mean by 'more reliable results'?"

A: "The fact that it gives a real, visual representation of the project greatly reduces the possibility of error. Member orientation is critical. For example, if a column entered into a program is rotated 90° off its correct axis, the structural integrity is compromised.







And if it isn't caught, the result can be disastrous—not to mention expensive. ElmAnalysis was the only program that gave a clear, visual representation of potential mistakes. And under the pressure of deadlines, anyone can make mistakes."

Q: "Tell me about some of the jobs you've done."

A: "Like most structural engineering consultants, about 75% of our work consists of smaller projects—schools, warehouses, and office buildings—and a few very large projects like the Memorial Auditorium being built here in Sacramento. It is a large building with some very difficult framing. Elm Analysis is the most practical solution—changes in the design can be viewed and identified graphically without compromising the schedule. And that's critical."

Q: "Do you think ElmAnalysis has improved the way structural and civil engineers work?"

A: "By making FEA programs easier, more engineers will be able to get more work done, and with ElmAnalysis' graphic interface to SAP90™, more options can be evaluated, resulting in better solutions."

Q: "How has Fujitsu's support been?"

A: "It's been great. They have experienced engineers available to answer real engineering questions, so I don't have to worry about being hung up in the middle of a job."

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Overpass Survives Despite Losing Two Out Of Three Columns

Tragedy struck as morning rush hour drew to a close on Chicago's Calumet Expressway November 20. At 9:20 a.m., a tire blew out on a northbound car. It swerved into the path of a truck, which jackknifed and spilled its cargo onto the roadway. A second car slammed into the rear of the truck, killing the car's driver.

The truck's cargo of large metal coils rammed into the support structure of an overpass, which consisted of three concrete-columns supporting a three-span steel I-beam bridge. The coils weight was estimated at more than 20,000 lbs.

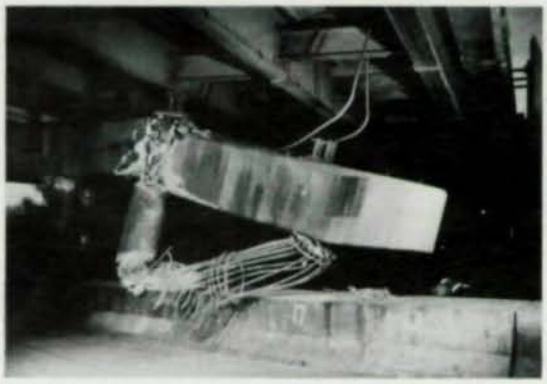
"One coil impacted the first column of the easterly pier and went right through it and hit the second column," explained Todd Ahrens, P.E., bridge investigation engineer with the Illinois Department of Transportation.

"The first column completely disintegrated and the second column buckled at the top and bottom and the pier cap also buckled. The third column couldn't take the eccentric load and it buckled at the top and sheared about 1'."

Fortunately, however, the overpass superstructure did not collapse. "The structure buckled about 5' down on the south end and the north end dropped about 1'," said Ahrens, who spent nearly 40 straight hours at the site after the accident. Damage was minimized, he theorized, when the parapets jammed together to create an arch-like support.

"It would depend on the detailing, but if it had been a precast, prestressed concrete beam bridge, I think you would have had much

Continued on page 10



A large metal coil slammed into the concrete columns of an overpass in Chicago and utterly destroyed one column while substantially damaging a second (above and top left). The steel overpass, however, did not collapse but merely buckled approximately 5' on one end and 1' on the other. After the structure was jacked back into position, damage to the superstructure and the decking was found to be minimal (bottom left). Photo credits: Above and top left—Keith Hale, Chicago Sun-Times; bottom left—Fred Beckmann

Overpass, Cont.

more of a problem when the pier dropped out," Ahrens said.

Temporary supports for the overpass were installed the same afternoon as the accident, and jacking operations began late in the day. Two jacking towers were placed on each side of a column location with a cross beam on top of the towers. The jacking took 16 hours and allowed four of the six lanes of the expressway to be reopened. About 85,000 to 90,000 cars use the expressway each day, according to highway department officials. The overpass, of course, is closed until it can be repaired.

Work on the new concrete columns is expected to begin the first week of December, and the construction is expected to take three to four weeks.

"The steel I-beams are in good shape," Ahrens said. "Only the pin connections were severely damaged. We think one pin plate may be cracked, but we won't be able to tell until the columns have been replaced and we can take the outside plates off and further assess the damage."

Two jacking towers were placed on each side of the three column locations to push the overpass back into position, which allowed four of the expressway's lanes to be reopened.

The overpass' deck is in good shape, according to Ahrens, and can be reused essentially as is. "We have to redo the expansion joints on the deck, and we have to do some concrete repair of the parapets," he said. The highway department is hoping to have the overpass reopened early next month. "Once the pin connections are repaired, we will reopen the overpass to traffic." □

Book Review: Schaum's Outline On LRFD

By Robert Lorenz, P.E.

During our college days, many of us benefitted from a college outline book that covered the entire subject of a course (say, Fundamentals of Psychology) in 100 pages instead of the 470-page course textbook. Often this was done to balance the bad effect of too many skipped classes.

Now, for the first time ever, the Schaum's Outline Series has added the subject of structural steel. However, this new book, *Theory and Problems of Structural Steel Design (Load and Resistance Factor Method)*, by Abraham J. Rokach, will not inspire students to cut their steel class. Rather, it will encourage them to be even more diligent.

This work is devoted primarily to problem-solving. For those who learn best by exposure to number-crunching exercises, this book is a bargain at $12.95—and not just for students. After buying a copy of it, many experienced engineers may dust off their AISC LRFD Manual, and, with this text as a companion volume, be able to avoid the need for a strict formal exposure to LRFD design methods.

As Rokach states in his preface: "This book is written for as wide an audience as possible, including students enrolled in undergraduate and graduate engineering and architectural curricula, and practicing engineers, architects, and structural steel detailers. The only prerequisite for an understanding of this text is the same as for an undergraduate course in structural steel design: a basic knowledge of engineering mechanics."

As a bonus, the book has text-book value. For example, the brief explanations of C_b in Chapter 5 and limit states of flexural buckling in Chapter 6 are supported with graphics that give a visual explanation of the technical formulas. Also, in Chapter 10, several worked-out solutions on composite columns are valuable, since they are not abundantly found in textbooks.

I recommend this book as an economical supplement to fill in or clear up some of the procedural aspects of the new steel design technology found in LRFD.

Theory and Problems of Structural Steel Design (LRFD Method), by Abraham J. Rokach; 187 pp.; McGraw-Hill, Inc. To order a copy, send $12.95 to: AISC Publications, P.O. Box 806276, Chicago, IL 60680-4124 (or phone 312-670-2400 ext. 433). □

AISC 1991 Prize Bridge Competition

Eligibility
To be eligible, a bridge must be built of fabricated structural steel, must be located within the United States (defined as the 50 states, the District of Columbia, and all U.S. territories), and must have been completed and opened to traffic from July 1, 1986 through May 1, 1991.

Judging Criteria
Will be based primarily upon aesthetics, economics, design and engineering solutions. Quality of presentations, though not a criterion, is important.

Award Categories
Entries will be judged in one or more categories, but may receive only one award.

Long Span One or more spans over 400 ft. in length.

Medium Span, High Clearance Vertical clearance of 35 ft. or more with longest span between 125 and 400 ft.

Medium Span, Low Clearance Vertical clearance less than 35 ft. with longest span between 125 and 400 ft.

Short Span No single span greater than 125 ft. in length.

Grade Separation Basic purpose is grade separation.

Elevated Highway or Viaduct Five or more spans, crossing one or more traffic lanes.

Movable Span Having a movable span.

Railroad Principal purpose of carrying a railroad, may be combination, but non-movable.

Special Purpose Bridge not identifiable in one of the above categories, including pedestrian, pipeline and airplane.

Reconstructed Having undergone major rebuilding.

Entry Requirements
All entries must contain an entry form, photographs and a written description of the project.

1. *Entry form:* All information requested on the form must be completed in full.

2. *Photographs:* Professional quality 8x10 color prints of various views showing the entire bridge, including abutments, 35 mm slides should also be submitted if available. All photographs must be cleared for use by AISC.

3. *Description:* Explanation of design concept, problems and solutions, aesthetic studies, project economics and any unique or innovative aspect of the project. Include no larger than 11x17 drawings showing elevation, framing system and typical details.

Method of Presentation
Each entry should be submitted in an 8½" x 11" binder, containing transparent window sleeves for displaying inserts back to back. The entry form included in the brochure must be easily removable, so that the identification of the entry can be concealed during judging. All information requested on the entry form must be included.

Awards
The winners will be notified shortly after the June judging. Public announcements of the winners will be made in the September issue of *Modern Steel Construction* magazine. Award presentations will be made to the winning designers during the National Steel Bridge Symposium, September 16, 1991, in St. Louis, MO.

Deadline for Submission
Entries must be postmarked on or before ***May 24, 1991****, and addressed to: American Institute of Steel Construction, Inc., Attn: Awards Committee, One East Wacker Drive, Suite 3100, Chicago, IL 60601-2001. For further information, call 312/670-2400.*

AISC 1991 Prize Bridge Competition

Entry Date____________________

Name of Bridge____________________ Completion Date____________________

Location____________________ Date opened to traffic____________________

Category in which entered____________________ Approx. total cost____________________

Span lengths__________ Roadway widths__________ Steel wt./sq. ft. of deck__________

Vertical clearance__________ Steel tonnage__________ Painted: Yes__________ No__________

Structural system(s) (describe briefly here)____________________

Innovative Concepts____________________

Descriptive data: Attach separate sheets (see entry requirements)

No. of photographs enclosed: Color prints____________________ 35 mm slides____________________

Design Firm:____________________
Phone

Address:____________________
Street: City and State Zip

Person to contact:____________________
Title

Consulting Firm (if any):____________________
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(ADDITIONAL ENTRIES MAY BE SUBMITTED ON COPIES OF THIS FORM)

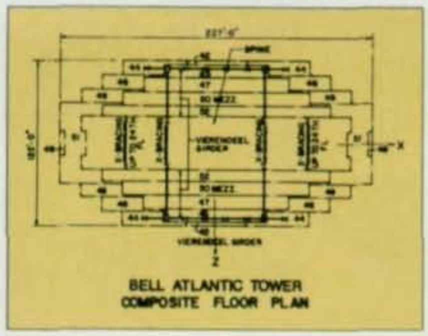
Spine Structures Provide Stability In Seismic Areas

Vertical uninterrupted lateral load resisting systems are a cost effective method of designing tall buildings with irregular architectural features

By P.V. Banavalkar, P.E.

In areas of high seismic activity, the selection of a structural system for high-rise buildings is primarily dictated by the need for ductility. In addition, however, the structure must provide enough stiffness to avoid an uncomfortable amount of wind sway and wind induced accelerations on the upper floors.

Over the years, a wide variety of systems have been developed for use in seismic areas, including: orthogonally placed ductile moment resisting plane frames; perimeter tubular structures; dual systems with combinations of ductile frames; and concentric or eccentric bracings. But modern architecture—with its multiple step-backs, distinctive tops, tall entrance lobbies, and subterranean parking that places a cost premium on closely spaced columns—demands innovation and modification to conventional structural systems to achieve a cost effective design. One solution is the "spine" structure.



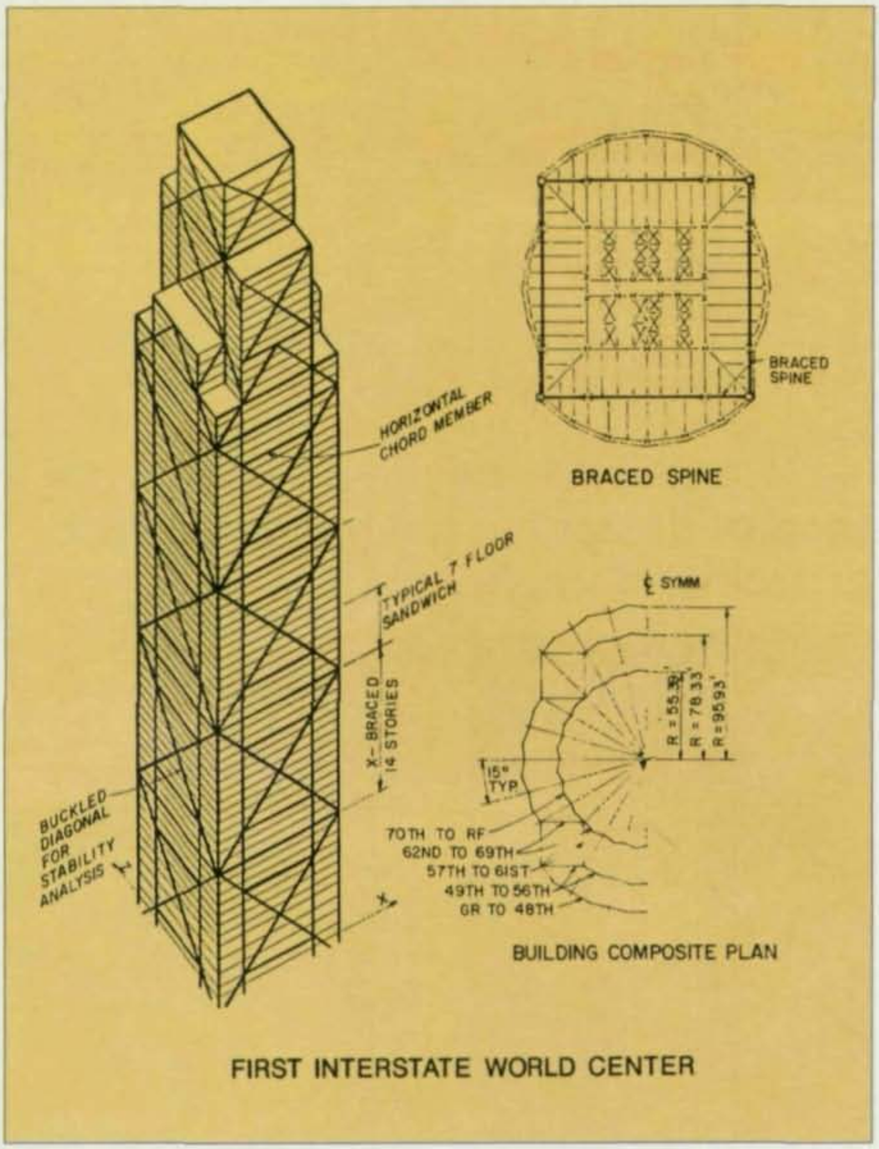
The Bell Atlantic Tower in Philadelphia is an example of a steel-framed "spine" structure with a vertical uninterrupted lateral load resisting system.

In this type of structure, the spine is the lateral load resisting system that provides stiffness and stability. The spine consists of vertical or inclined elements, and shear membranes in the form of braced frames, walls or vierendeel girders. The vertical or inclined elements resist axial loads due to overturning moment whereas the shear membranes resist shear forces due to lateral loads.

Structural Principles

Spine structures must follow certain basic principles.

- To achieve continuity in resistance of both overturning moment and shear loads, the spine should be located within an uninterrupted portion of the building. This prevents discontinuity in overturning resistance and the existence of weak or soft stories within a structure that hinder satisfactory seismic performance.
- The principal axes of the spine should coincide with the geometrical mass principal axes of the structure.
- The design of the floor framing has to be consistent with the spine



The original plan for the First Interstate World Center in Los Angeles was a pure braced spine structure, as pictured above (figures 1a, 1b, 1c). This was rejected, however, because it was felt that the ductility of the structure to absorb energy in case of an unforeseen seismic event was below the desired level. Also, the diagonals crossing through the floor space presented a problem for space planning.

of the building. Heavily loaded gravity columns strategically placed in the spine of a structure supporting free spanning, but not necessarily optimized floor framing, can provide efficient overturning resistance for transient lateral loads thereby achieving overall economy of the structural frame.

- The area enclosed by the spine should provide optimum torsional stiffness for the structure. At times, supplementary ductile frames may be necessary to improve the torsional characteristics of the building.
- The texture of the spine, either in the form of a vierendeel girder, bracing, or gravity load carrying vertical element, should be located in such a way that it does not adversely affect the space planning in the lease area or the efficiency of subterranean parking.

Design Criteria

The design of the spine both in spirit and letter should conform to the seismic provisions of the 1988 UBC code. For high-rise buildings with longer lateral fundamental periods of motion and displacement dictated dynamic response, it is recommended that the spine be designed to remain essentially elastic for a maximum credible earthquake (5% critical damping) at the closest active fault. This criteria is mandatory in structures with axially loaded bracing elements, while the criteria can be relaxed in structures with totally ductile moment resisting frames by designing it for the maximum probable earthquake and placing more reliance on ductility.

In a deterministic approach to desgining buildings, the spine is designed to remain elastic for a postulated ground motion with an unknown global ductility beyond the yield of the spine. In this approach, the safety and performance of the spine depends primarily on the accuracy of the prediction of the postulated earthquake. But while deterministic approach sometimes is used, engineers should aim for a deterministic approach with ductility. That is, the spine is not only designed to remain elastic for a postulated ground motion, but its ductility is investigated beyond its yield by a monotonically increasing load.

Additional design criteria that should be considered includes: time history analysis; response to vertical acceleration; and non-linear analysis.

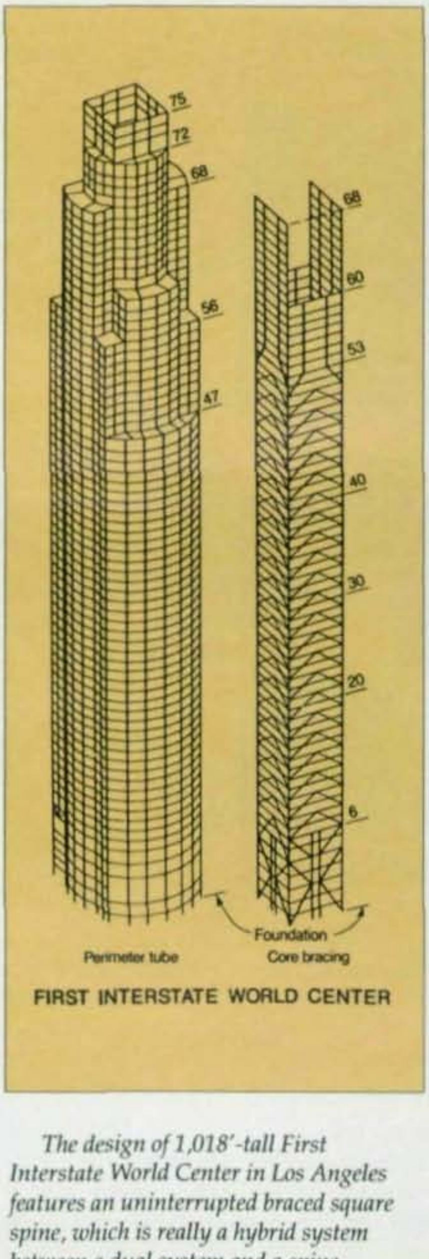
First Interstate World Center

CBM Engineers, Inc., Houston, was the structural engineer on the 1,018'-tall First Interstate World Center (Library Square) in Los Angeles. This 75-story building—the world's tallest in a seismic zone 4 or equivalent—is granite clad and features multiple step backs. Project architect was Pei Cobb Freed & Partners.

The simplest and most obvious structural system for this building would have been a perimeter ductile tubular frame. However, due to the multiple step backs, this system was not only very expensive, but its dynamic characteristic was sensitive to wind induced acceleration at the top floor.

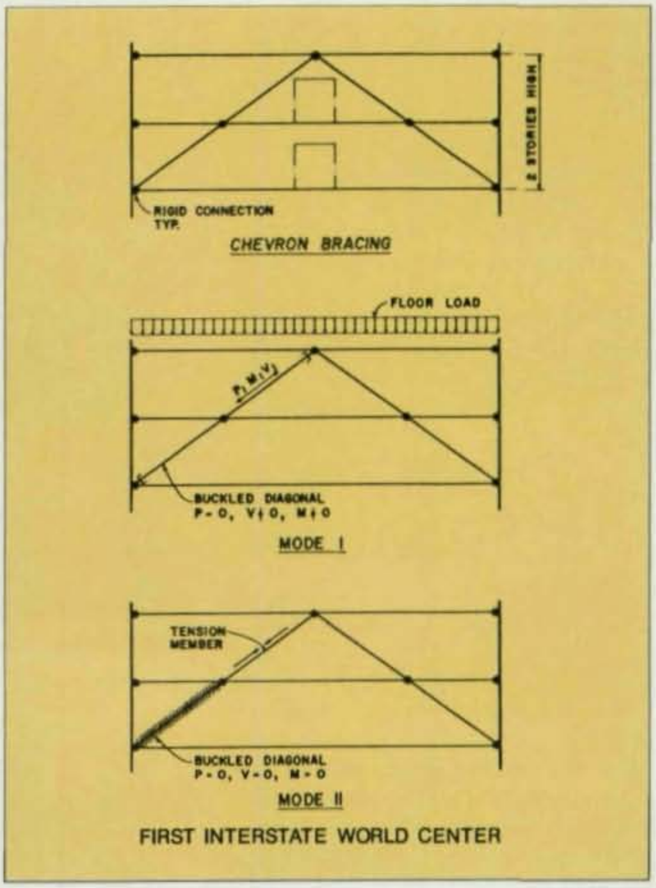
Instead, the engineer examined the potential for designing a braced spine structure (see diagrams 1a., 1b., and 1c.). Three basic design criteria were introduced:

- Braced structures are commonly used for offshore platforms. Studies have shown that the higher the strength and stiffness of the horizontal members, the better the overall ductility performance of the structure. Experimental findings on braced frames also have demonstrated that satisfactory ductile non-linear behavior can be ensured by preventing any premature failure of the connections. Therefore, on this project



FIRST INTERSTATE WORLD CENTER

The design of 1,018'-tall First Interstate World Center in Los Angeles features an uninterrupted braced square spine, which is really a hybrid system between a dual system and a spine structure.



The final design consists of a square spine composed of a two-story tall chevron braced core with a perimeter ductile moment-resisting frame with radially placed columns. The bases of the four core columns consist of 7½" plates, each measuring 4 sq. ft. and weighing 60 tons (top). The lobby levels have K-braced sections below the two-story chevron bracing (above).

To avoid any progressive catastrophic failure of the structure in case of buckling of a chevron-braced diagonal, two separate possible modes of failure were considered. In Mode I, the buckled diagonal assumes to have lost only axial load carrying capacity, while in Mode II the lower end of the buckled diagonal was physically removed.

the joints connecting the members were designed for strength far in excess of the capacity of the members themselves.

- The ductile subframe was designed by the method of allowable stress design for 25% of code level forces. The axial load carrying capacity of the diagonals was ignored during this analysis.
- The stability and overall behavior of the structure was investigated in case of a failure of diagonals at different levels.

The buckling of a diagonal changes the dynamic characteristic of the entire structure. The structure with a buckled diagonal responds unsymmetrically to an unidirectional ground motion. The input seismic energy is dissipated in two sway modes along with a rotational mode. The effects of a buckled diagonal between floors 10 and 14 were quantitatively examined. As anticipated, the overall stiffness was not dramatically affected. The check of members between the sandwich of buckled diagonals showed that the ductility demand on all members was less than 1.2.

A braced spine structure was rejected, however, because it was felt that the ductility of the structure to absorb energy in case of an unforeseen seismic event was below the desired level. Also, the diagonals crossing through the floor space presented a difficulty in interior space planning.

Instead, a design was developed featuring an uninterrupted braced square spine, which is really a hybrid between a dual system and a spine structure. The design of this spine was based on the deterministic approach for the ground motion of the maximum credible earthquake on the San Andreas Fault (5% dampened).

The final design of the structure consists of a 73'-10" square spine composed of a two story tall chevron braced core with a perimeter ductile moment resisting frame having radially placed columns at 15° on center.

The long span floor framing coupled with two-story-tall free spanning core loaded the corner core columns of the spine in such a way that the design was primarily governed by gravity load. To avoid any progressive catastrophic failure of the structure in case of buckling of a chevron-braced diagonal, two separate possible modes of failure were considered. In Mode I, the buckled diagonal assumes to have lost only axial load carrying capacity, whereas in Mode II the lower end of the buckled diagonal was physically removed. The safety of the structure was ensured by checking the remaining effective members and designing the connections for all possible load combinations.

An attempt was made to proportion the stiffness of the perimeter frame and core bracing in such a way as to set yielding in the ductile frame prior to buckling of the core diagonals. To establish the overstrength and the post yield behavior, a simplified non-linear analysis was conducted by applying monotonically increasing load.

The efficiency of the spine system can be further documented by comparing the efficiency of the core column for gravity load alone to the combination of gravity, wind, and seismic loads.

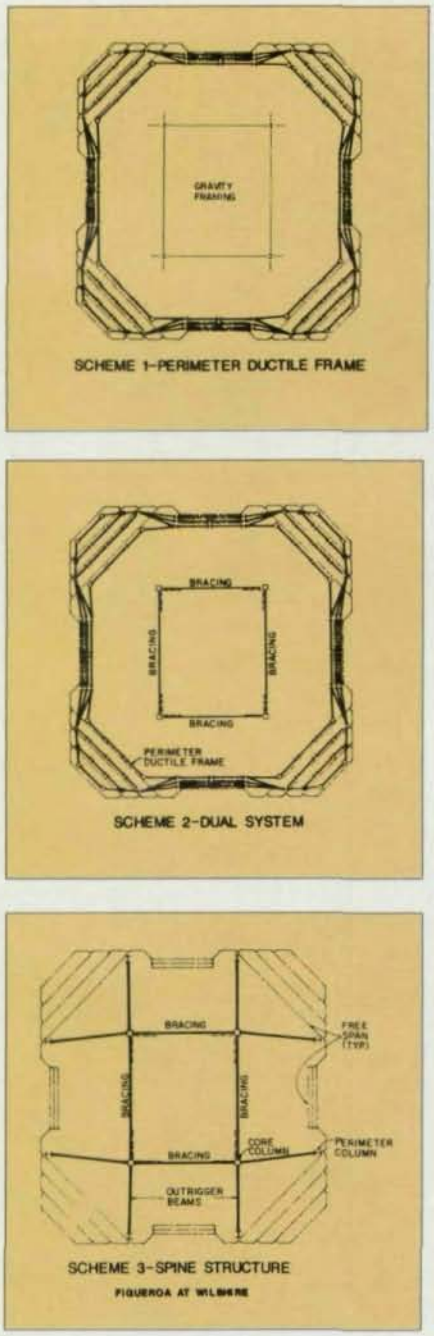
This hybrid spine structure required 5,500 fewer tons of structural steel than a conventional frame and 2,500 more tons of steel than a pure braced spine structure. The final design required 26,500 tons of structural steel.

A spine structure was chosen for the Figueroa at Wilshire building in Los Angeles because it used the least amount of steel and provided an opportunity for column-free lease spaces. Photo by Erich Koyama

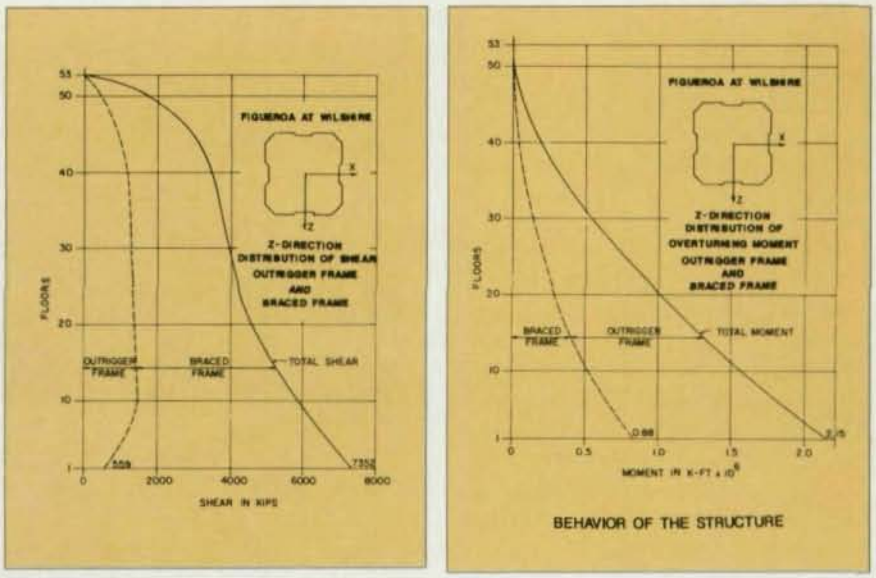
Figueroa at Wilshire

The 53-story Figueroa at Wilshire tower in downtown Los Angeles provided an opportunity to design an efficient spine structure. Structural engineer on the project was CBM Engineers and architect was Albert C. Martin and Associates, Los Angeles.

Three lateral load resisting systems were studied: perimeter ductile moment resisting frame; dual system with braced (concentric or eccentric) core with perimeter ductile frame; and a spine structure



Three lateral load resisting systems were studied for Figueroa at Wilshire: perimeter ductile moment resisting frame; dual system with braced core and perimeter ductile frame; and a spine structure consisting of braced core and outrigger ductile frames.



***Figures 2a and 2b**. The interaction of the braced core and the outrigger ductile frames about both principal axes is essentially the same. In general, more than 65% of the shear is carried by the braced core, while almost 65% of the overturning movment is resisted by the perimeter columns in the outrigger frame of the Figueroa at Wilshire building.*

consisting of braced core and outrigger ductile frames with the long span floor framing structured in such a way that the main columns participating in the lateral load resisting system are loaded heavily by gravity load.

A cost comparison of all systems showed that the spine structure used less structural steel and provided an opportunity for column free lease spaces. The structure used 16,000 tons of structural steel.

The interaction of the braced core and the outrigger ductile frames about both principal axes is essentially the same. In general, more than 65% of the shear is carried by the braced core, while almost 65% of the overturning moment is resisted by the perimeter columns in the outrigger frame (see figures 2a. and 2b.). Essentially, the structure has three major components: interior core bracing; outrigger beams; and columns.

Interior Core Bracing

The interior core is concentrically braced. The most important design criteria of the braces is that in the event of buckling of one of the compression diagonals, the horizontal members supporting the floor loads do not form a mechanism resulting in catastrophic failure of the floor.

Outrigger Beams

The outrigger beams span approximately 40′ and have three major functions.

- They support the design floor loads.
- The outrigger beams along with the core and perimeter columns have to act as a ductile moment resisting frame to carry a minimum of 25% of the design code level forces without the presence of interior core bracing.
- The beams have to possess enough stiffness to create effective linkage between interior core and perimeter columns to provide efficient overturning resistance to the seismic loads. To accomplish this, 36"-deep outrigger beams were required.

The depth of the outrigger beams were restricted by the ceiling cavity, which has to provide optimum depth to minimize floor-to-floor height. Therefore, the beams were offset into the floor and notched for the passage of the

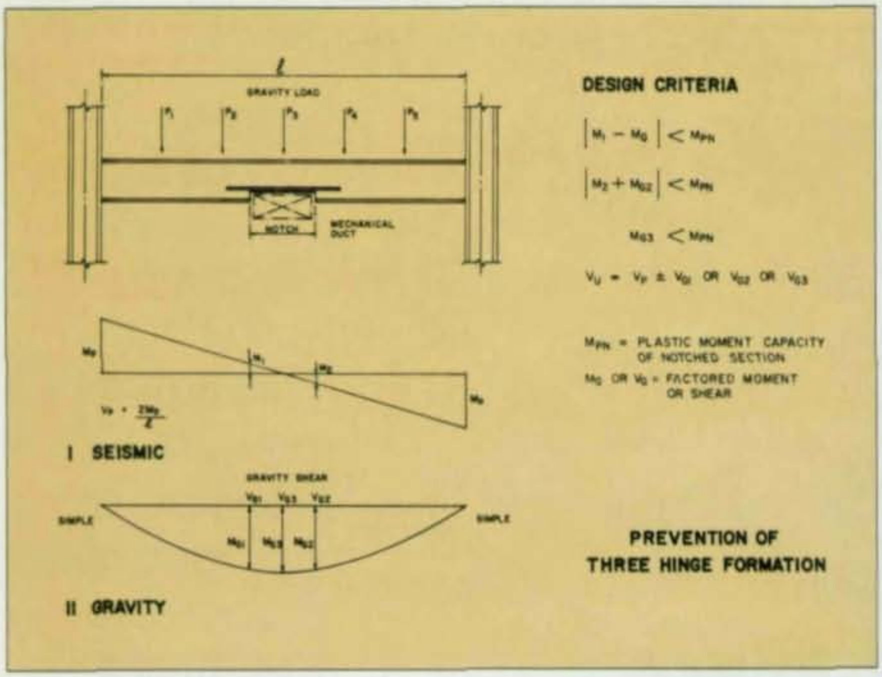


Figure 3. The notches in the beam were stiffened to prevent the formation of a three-hinge mechanism with substantial moment capacity at the notches.

mechanical ducts (see figure 4).

In case of a major seismic event with the plastification of both ends of the beam, the notches in the beam were stiffened in such a way as to prevent the formation of a three-hinge mechanism with substantial moment capacity at the notches (see figure 3). With full plastification at the supports, the minimum factor of safety of 1.33 was provided for the total gravity load carrying capacity.

The outrigger beams are laterally braced to prevent lateral torsional buckling and they are effectively connected to the floor diaphragm by shear studs to transmit horizontal shear force to the frame.

Both the perimeter and core columns were heavily loaded to provide overturning resistance to the entire structure. The perimeter columns were checked against full plastification of outrigger beams.

Spine structures have been used extensively in non-seismic zones due to their proven cost efficiency. In addition, their superior structural performance in providing overturning resistance while eliminating the existence of weak or soft stories is leading to their growing acceptance in seismic areas. ■

P.V. Banavalkar is chief structural engineer/executive vice president of CBM Engineers, Inc., Houston. This article was adapted from a paper presented at the ASCE Metropolitan Section, Spring Seminar 1989, in New York City.

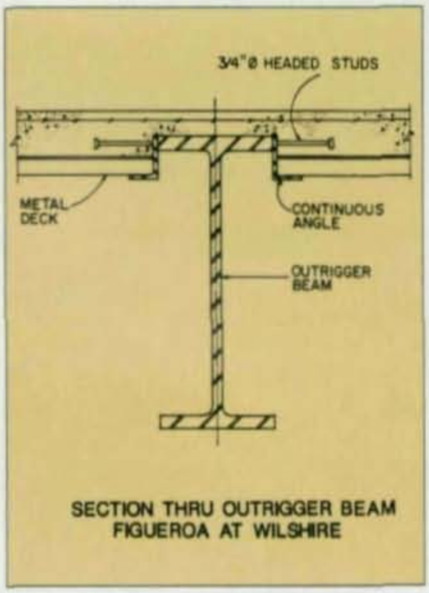
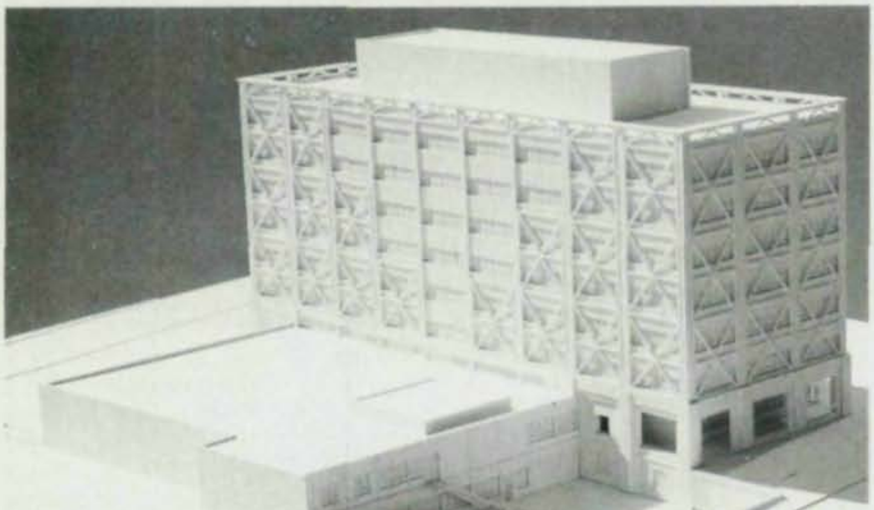


Figure 4. Notched outrigger beam.

Seismic Upgrade Preserves Architecture



The seismic performance of the concrete-framed University Hall building (top) in Berkeley, CA, was rated as "very poor" by the University of California. A steel bracing frame (above) was chosen as the most economical method of improving that rating.

New steel bracing is expected to upgrade a concrete building's seismic performance from "very poor" to "good"

By Loring A. Wyllie, Jr., S.E.; John A. Dal Pino, S.E.; and Jeff Cohen

The importance of a University of California program to seismically upgrade their buildings was underscored by the October 1989 Loma Prieta earthquake. While the university emerged relatively unscathed, there was enough damage to nearby structures to warrant concern.

The program's $50 million in recently appropriated funds includes $8.2 million for the major seismic strengthening of University Hall in Berkeley, which is expected to be completed in July. The seismic performance of the building—which is located within a mile of the active Hayward Fault—had been rated as "very poor" as defined in the Seismic Safety Policy of the University of California. The retrofit will change that rating to "good".

Located on Oxford Street across from the main entrance to the University of California at Berkeley, the 1957 building clearly reflects the Modern design that was then popular. It consists of a seven story rectangular tower and an adjoining two-story wing. Projecting concrete columns run along the face of the tower, except at ground level where they are freestanding, as the building facade steps back to form an arcade.

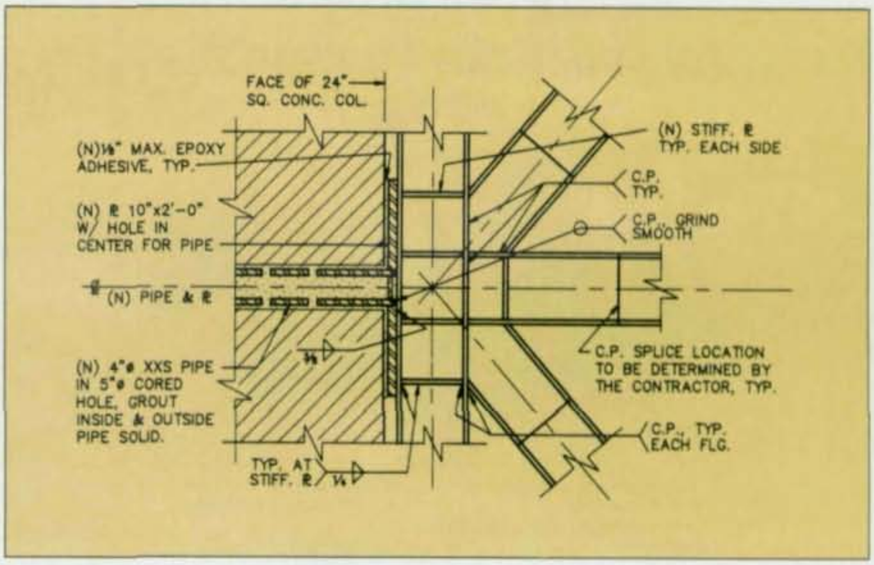
Alternating bands of strip windows and ceramic tile veneer create a horizontal look, in contrast to the supporting columns. Typical of that period, the original designer was concerned with the handling of the corner column, which resulted in a slight modification of the window configuration at the corner bays.

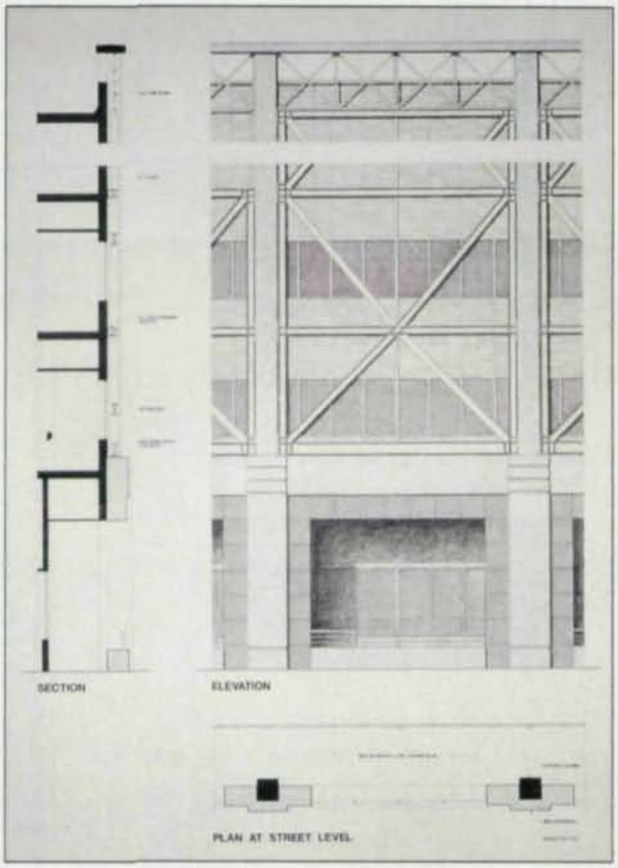
The tower is a concrete frame structure 200' x 70' in plan. Floors are a combination of concrete flat slabs and slab and beam framing. In the original design, lateral forces are resisted by the exterior frames around the perimeter of the building, consisting of 24" square columns and 8"-wide by 7'-deep concrete spandrel beams, and by three interior transverse shear walls in the core.

Seismic Retrofit

Because of the need to work with the materials and existing configuration of the building, developing a seismic retrofit scheme was a difficult task. Architect for the retrofit project is Hansen/Murakami/Eshima, Inc., Oakland, and structural engineer is J.J. Degenkolb Associates, San Francisco.

Any retrofit design process involves first identifying the weak links in the lateral force resisting system. For University Hall, the weak link was identified as the deep spandrel beams being stronger than the columns under frame action. The columns below the fourth floor were spirally reinforced and would provide ductile performance, but above the fourth floor they are nominally tied, and





The connection between the new steel bracing and the existing concrete column on the University Hall project is shown above. Shown at left is an elevation and section of the project at street level.

are therefore susceptible to damage. Unless braced, they could lose their integrity and be unable to support the weight of the upper floors during a major earthquake.

Steel Bracing System

Three approaches for bracing the tower were proposed by Degenkolb. One scheme relied on new shear walls to be built within the tower; and another called for substantial concrete piers—rising the full height of the building—to be formed around most of the existing columns. The third approach—a steel bracing system—was chosen because it was the most economical option, and because it would create the least amount of disruption to the building's occupants during construction. Equally important, the steel option gave Hansen/Murakami/Eshima the greatest opportunity to enliven the appearance of the building.

The retrofit design calls for a composition of symmetrically arranged X-braced steel frames to be placed within the existing bays of projecting columns, creating an open structural screen over the building. Each X-brace will be two-stories high and will be painted to complement the light colors of the original building. A new concrete colonnade at the base of the tower, designed as shear walls with boundary members with closely spaced ties, will stiffen the lower story and transfer shear forces from the X-bracing to the foundation.

Decorative Structure

Architecturally, the colonnade keeps within the "clean" aesthetic of the original building. It also creates a stronger arcade than currently exists at the first floor, and is more appropriate to accept the "visual weight" of the new bracing. A new, narrow band of decorative braces is planned for the top of the building to enhance the vestige of a cornice, which now exists as a thin, concrete cap running along the top of the columns, which rise above the walls of the building. The adjoining two story wing will be strengthened with a new roof diaphragm truss system and three segments of transverse shear walls.

Once the bracing pattern was established, the next task was selecting member sizes and the various connection details, with the member selection largely dictated by the connections. After many comparisons, 8" wide flange sections placed with their webs vertical so water will not be trapped were selected. The 8" sizes provide a maximum slenderness ratio of $L/r = 98$, which ensures that the diagonals will be effective in compression as well as in tension.

Full Penetration Welds

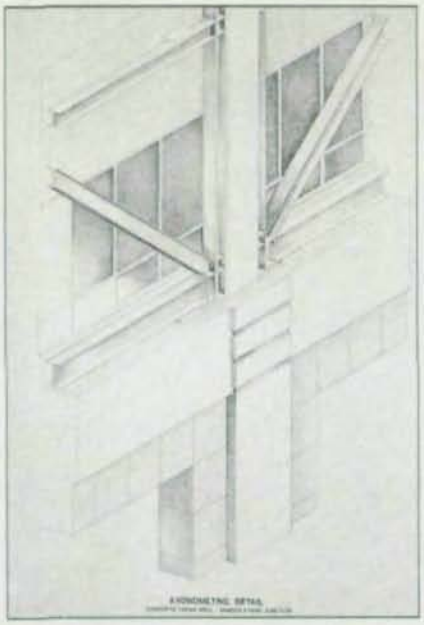
The steel-to-steel connections are full penetration welds, to develop the full strength of the members. Since actual seismic forces often exceed code level design loads, a less than full strength connection would create a weak link in the system, which could lead to a non-ductile failure. The architects also wanted to avoid gusset plates since they often are large and would destroy the aesthetic lines of the braces.

Final details indicating which welds are shop or field welded will be determined by the contractor consistent with his erection scheme. Some of the welds in the connection will be difficult because of the proximity of the existing structure, but it was concluded that all welds are possible using the selected wide flange shapes.

Minimal Construction Noise

Connections of the new steel frame to the concrete also was a challenge. Since the building will be nearly 70% occupied during the work, the University and design team wished to minimize the amount of noise and vibration from construction activities.

A method was developed for anchoring the vertical members of the X-braced frames to the existing columns using a minimum number of attachment points. Typically, at each floor level a 4" double extra strong pipe is inserted into a 5" hole cored horizontally through the column (see University Hall detail).



A new concrete colonnade at the base of the tower is designed as shear walls with boundary members and closely spaced ties and will transfer shear forces from the X-bracing to the foundation.

A flat plate is shop welded to one end of the pipe and a similar plate with a pipe-sized hole is field welded after installing the pipe. The center of the pipe and the annular space between the pipe and the concrete is filled with non-shrink grout. The space between the steel plates and the concrete columns is epoxied after the steel vertical wide flanges are in place.

The vertical wide flanges are connected to the vertical flat plates with fillet welds on both sides. These connections also transfer the seismic load of the building into the bracing system through bearing on the columns.

The horizontal members of the bracing system are secured to the existing horizontal concrete beams with steel brackets. These brackets are welded to the steel beams and epoxy bolted to the concrete, through the ceramic tile veneer. The vertical wide flanges extend down into the concrete colonnade for attachment.

Seismic Bracing

There was some concern about the nominally tied columns above the fourth floor at location where new steel braces are not being installed. If lateral deformations are large enough, these columns might develop excessive cracks, perhaps leading to loss of vertical load capacity. To locally strengthen and provide confinement to these columns, steel plates will be added on both sides of the columns and epoxy bolted with the column core.

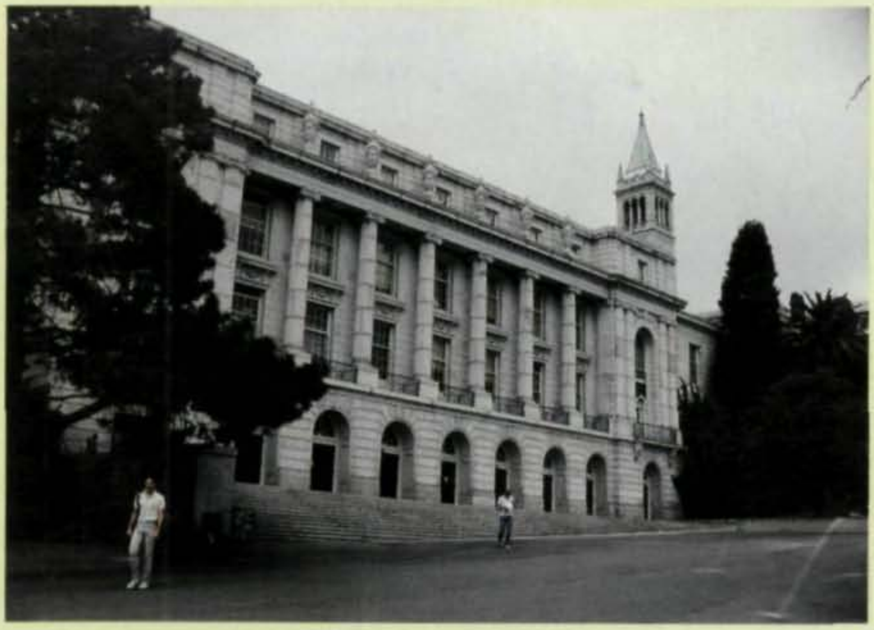
The design of this seismic bracing system relies on some experimental work conducted in the mid-1980s and funded by the National Science Foundation. At that time, Degenkolb cooperated with Professor James Jirsa at the University of Texas at Austin in research to seismically strengthen non-ductile concrete frames.

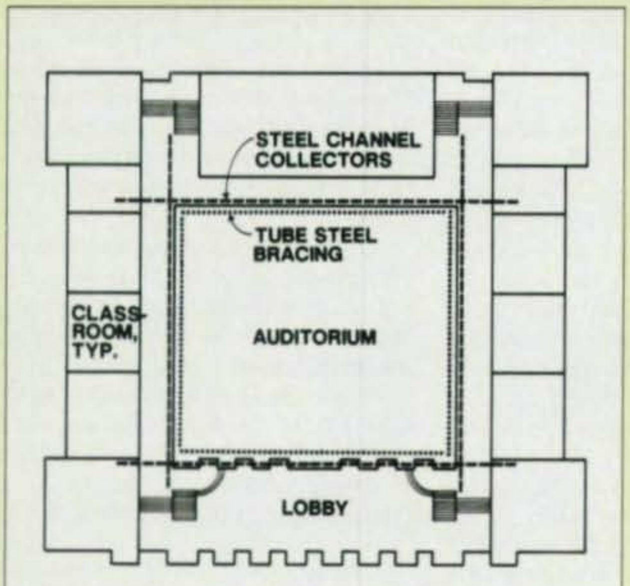
A two-story, two-bay specimen of very similar proportions was strengthened with steel bracing. The project demonstrated that the bracing could successfully provide increased lateral strength and stiffness for a non-ductile concrete frame.

Computer modeling has shown that steel bracing is effective in increasing stiffness and protecting concrete columns, although during an earthquake some cracking in the concrete is still to be expected. The extent of structural or nonstructural damage expected during a strong earthquake, however, is not anticipated to jeopardize life. ■

Loring A. Wyllie is chairman of the board and John A. Dal Pino is a principal of H.J. Degenkolb Associates, Engineers, San Francisco. Jeff Cohen is with Hansen/Murakami/Eshima, Inc., Oakland.

Strengthening Behind The Scenes





Wheeler Hall in Berkeley, CA, was classified as a "poor seismic risk" by the University of California. The bracing scheme that was developed (left) involved reinforcing the walls around the auditorium and tying the perimeter spaces around the auditorium back into the inner court.

By David L. Messinger, S.E.; Jeff Cohen; Kearny Chun, R.A.

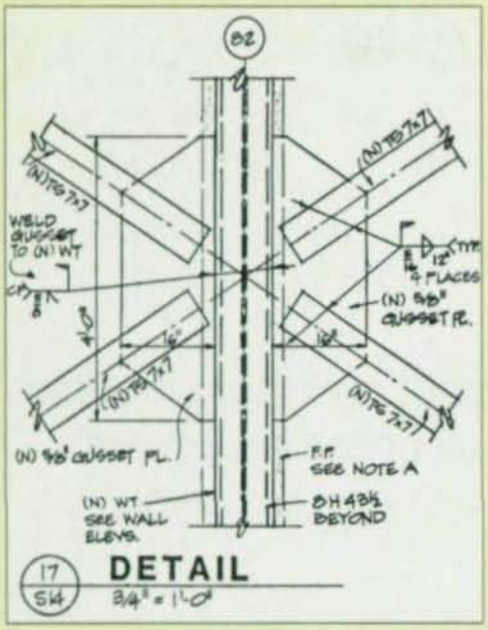
While the new structural bracing being added to University Hall is mostly visible, the new bracing is almost completely hidden for the nearby Wheeler Hall building.

Classified as a "poor seismic risk", structural strengthening of the University of California at Berkeley building was necessary to improve its seismic resistance and to provide substantial life-safety for the occupants of the building during an earthquake. Architect for the retrofit is Hansen/Murakami/Eshima, Inc., and structural engineer is David Logan Messinger and Associates, Oakland.

Wheeler Hall, which was designed by architect John Galen Howard in 1915, is a prominent building of historical significance. It houses one of the largest and nicest auditoriums on campus, and also is a focus of the university's humanities program.

Built like a square donut, Wheeler Hall has a two-story auditorium/lecture hall surrounded by four stories of classrooms and offices. The top two floors of the building overlook a light court above the roof of the auditorium. It is a steel-framed building with reinforced concrete floors, roof, walls and foundation totaling 126,636 gross sq. ft. The concrete walls forming the light court do not extend through the building, but rather are supported by steel columns at the auditorium level.

While the exterior, granite-faced walls were determined to be seis-



The new wall bracing for Wheeler Hall was accomplished by a diamond-shaped bracing system using tubular steel and introduced within the existing steel beams and columns. The detail at left shows the typical connections of the diagonal bracing.

mically adequate, the inner court walls were identified as a major seismic resisting deficiency, causing all interior lateral forces to be transferred to the exterior walls by the floor diaphragms.

The project had several constraints, including: a tight budget; the need to provide a non-combustible structure; and the need for a light-weight structure in order to meet the seismic zone 4 requirements of the University's seismic policy. Also, the auditorium had been remodeled before a decision to proceed with the seismic design work was made, so any proposed solution could not effect either the renovated interior or the historic exterior.

After careful consideration of structural steel and gunite shearwalls, structural steel was chosen as the best method to meet all requirements.

The structural solution was two-fold: reinforcing the walls around the auditorium; and tying the perimeter spaces around the auditorium back into the inner court. The wall bracing was accomplished by a diamond-shaped bracing system introduced within the existing steel beams and columns. The diamond shape was used because the gusset plates could be welded to the middle parts of the beams and columns, thus avoiding the existing beam-column connections.

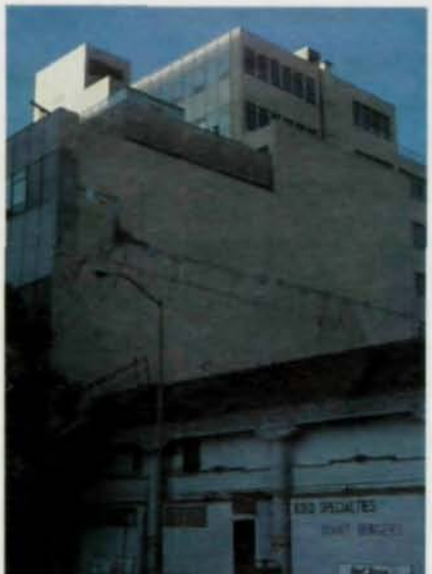
Tubular steel sections were found to be the most efficient steel section as they used the least space in the wall, which enclosed ducts, piping and electrical conduit. Connections to the steel gusset plates were accomplished by slotting the steel tubes and welding them to the gussets.

Steel channel collectors had to be added under the floor and roof slabs and extended beyond the footprint of the auditorium to tie the building together at the re-entrant corners of the inner court. These collectors had to be spliced at existing column steel beams. This was accomplished by cutting slots in the webs of the existing steel beams and passing through steel plates which welded to the channel collectors.

All of the new and exposed existing steel elements were fireproofed with spray-on fireproofing. ■

David L. Messinger is the president of David Logan Messinger & Associates Inc., an Oakland-based structural engineering firm. Kearny Chun is an associate and Jeff Cohen is affiliated with Hansen/Murakami/Eshima, Inc.

Renovating For The Future



The most dramatic damage to Broadlake Plaza occured on the north face, which was repaired in part with sandwich plates at the fourth floor and wall ties at the fifth and sixth floors.

While not required, an upgrade was the only way to ensure survival through future earthquakes

By William A. Andrews, C.E.

In the wake of last year's Loma Prieta earthquake, many building owners in the San Francisco-Oakland Bay Area re-evaluated their seismic options. Some buildings with substantial damage were razed, while others with only cosmetic damage were quickly repaired.

Still a third group suffered just enough damage to convince their owners to commence seismic upgrades. Broadlake Plaza, an eight-story, 114,000-sq.-ft. office building in downtown Oakland, falls into this third category. The building was constructed in two phases, beginning in the 1920s with a three-story concrete frame and shear wall structure. In 1956, an eight story steel frame structure was built adjacent to the existing building, with two stories extending over and supported by the old concrete structure.

The three-story concrete building has a 4"-thick slab supported by one way beams at approximately 8′ centers. The beams are supported by deep haunched girders that are supported by columns and pilasters. The building has concrete walls on three sides and concrete spread footings. Lateral resistance is provided by the concrete walls in combination with frame action of the concrete girders and columns. All of the concrete is reinforced with mild, square, reinforcing steel.

The steel building has a metal deck with 2½" of concrete fill supported on wide flange steel beams. The beams are supported on wide flange girders and columns. Lateral resistance for the north-south direction is provided by concrete shear walls located along the east and west sides. In the east-west direction, the lateral resistance is provided by a moment frame with riveted and bolted connections.

The steel-framed portion of the building is tied together with rigid concrete and metal deck floor diaphragms. However, the second and third floors are structurally separated at the interface between the concrete building and the 1956 steel addition.

The portion of the steel building above the old concrete building is supported by steel columns penetrating through the concrete slabs to new and existing footings. Steel columns along the east and south sides bear on the old concrete roof and parapet. A reinforced concrete masonry curtain wall was constructed along the north side and it bears on the old concrete parapet but was not dowelled to it. The masonry wall was structurally connected to the floors of the steel building.

The site is near several active faults, including the San Andreas (15 miles) and Hayward (3 miles) faults. The building was approximately 60 miles from the epicenter of the 7.0 magnitude Loma Prieta earthquake, and the ground motions measured during the quake in Oakland were lower in intensity and duration than what would be expected for a major earthquake from either the San Andreas or Hayward faults.

Earthquake Damage

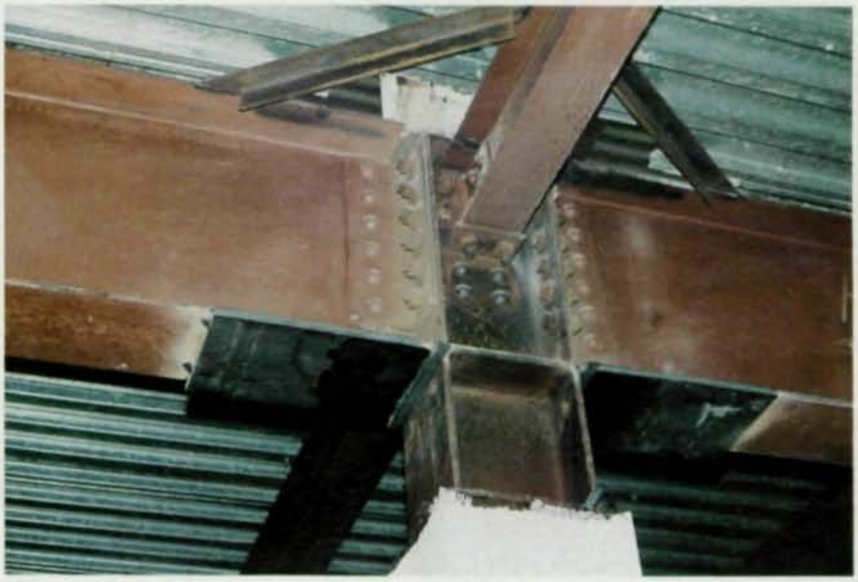
The structure had obvious severe conceptual deficiencies, including the incompatibility of the steel and concrete systems, the incomplete lateral system of the concrete building, and the rigid curtain wall supported by a flexible building. Fortunately, the magnitude and duration of the shaking of the building during the Loma Prieta quake were not enough to excessively stress the primary lateral resisting elements.

The most dramatic damage was the movement of the masonry portion of the north wall on top of the old concrete parapet. The final position of the masonry wall was 1½" west and ¼" south of its pre-earthquake location. This movement occurred when the upper portion of the steel moment frame structure moved relative to the old concrete building.

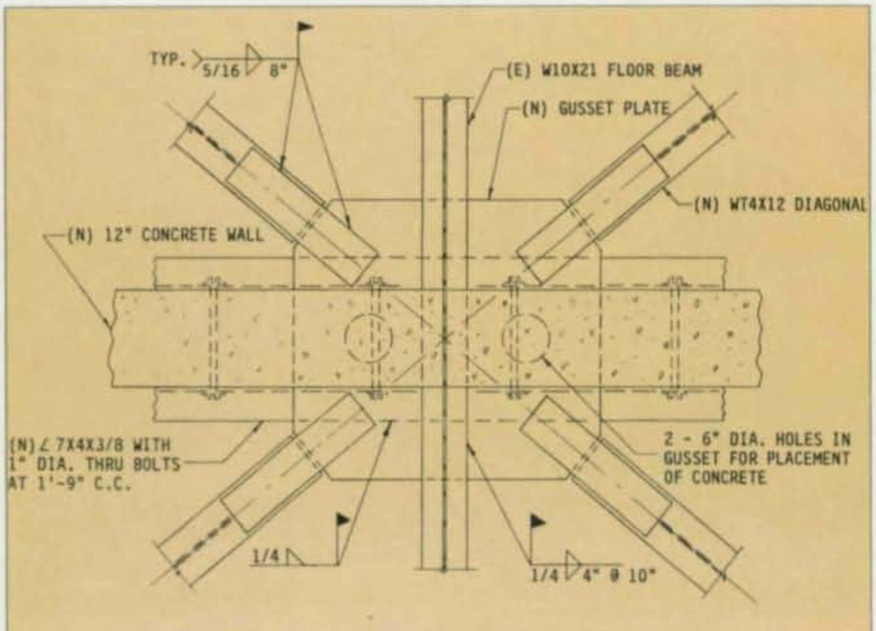
This relative movement also caused the steel columns bearing on the concrete roof to shear their base plate bolts and to translate. The metal curtain wall wracked out of plumb.

Other damage included cracked stucco facades, diagonal cracks in concrete curtain wall piers, and wracking of window frames out of square. The west wall cracked along a deficient construction joint at the third floor.

Because some Bay Area elected officials seem to believe in the theory of "Structural Darwinism"—that it is not necessary to do anything about unfit buildings because an earthquake will take care of them—many buildings with low-levels of damage from the Loma Prieta earthquake, including Broadlake Plaza, were not required to comply with the current code



The existing beam-column moment connections (top) were shop rivited and high strength bolted in the field (above). Shear connections were made at the beam webs with double angles. The top and bottom beam flanges were bolted to the colulmn flanges with split-beam tees. Shown is a beam-column connection at the eighth floor.



FLOOR TRUSS DETAIL

A diaphragm at the fourth floor transfers large lateral forces out of the steel moment frame into the new concrete shear walls. The existing metal deck and concrete fill diaphragm was overstressed, so a series of new horizontal steel trusses were designed to transfer these large forces to the walls. The truss system is welded to the underside of the fourth floor framing.

(1988 UBC). Fortunately, however, this owner decided to go ahead not only with repairing the earthquake damage, but also with developing a program to upgrade the lateral system of the building to resist the seismic lateral forces as prescribed by the current code.

The challenge of seismically upgrading this building extended beyond a complex structural analysis. Existing tenants occupying portions of the first, fourth, and seventh floors could not be relocated, which minimized the allowable amount of new construction. Time also was important because the owner needed to fill the unoccupied space as soon as possible. And finally, the cost of the seismic upgrade had to be economical. The program developed included sampling and testing existing structural steel and concrete, repairing earthquake damage, performing a dynamic analysis of the lateral system, strengthening existing structural members where required, and designing new members for the lateral system.

Program Evaluation

The presence of significant torsional responses, significantly different story stiffnesses, and split floor diaphragms required that the structure be subjected to dynamic analysis. A response spectrum approach was employed using the 1988 UBC normalized response spectra. The building's lateral system was modeled and analyzed using ETABS, a software package from Computers and Structures of Berkeley, CA.

The response spectra were applied in two orthogonal and two skewed directions to obtain the maximum building response. The weak links in the existing lateral system were high shear stresses and overturning forces in the concrete shear walls, excessive drift in the steel moment frame, and a large overstress of the fourth floor diaphragm.

Materials Testing

A materials testing program was

initiated to determine the strengths of the existing structural steel and concrete. There were no drawings available of the original 1920s structure, and the drawings for the 1956 addition provided minimal information.

Three 3" x 16" coupons were taken from the webs of three different column wide flange sections and subjected to chemical analysis and tension tests. The chemical analysis showed the steel to be low carbon, readily weldable and meeting the requirements for ASTM A36 steel. Test results showed the tensile yield strengths ranging from 39.1 ksi to 41.9 ksi. The actual yield strengths of these columns were used in a composite design of the new wall at Line 4 (see floor plan).

The existing concrete and concrete masonry block walls were X-rayed with a pachometer to determine the size and location of the reinforcing steel. Compression tests from core samples taken from the 1920s structure averaged 2,555 psi. Compression tests from samples of the 1956 addition average 4,270 psi, compared to the 3,000 psi minimum specified on the drawings.

Damage Repair

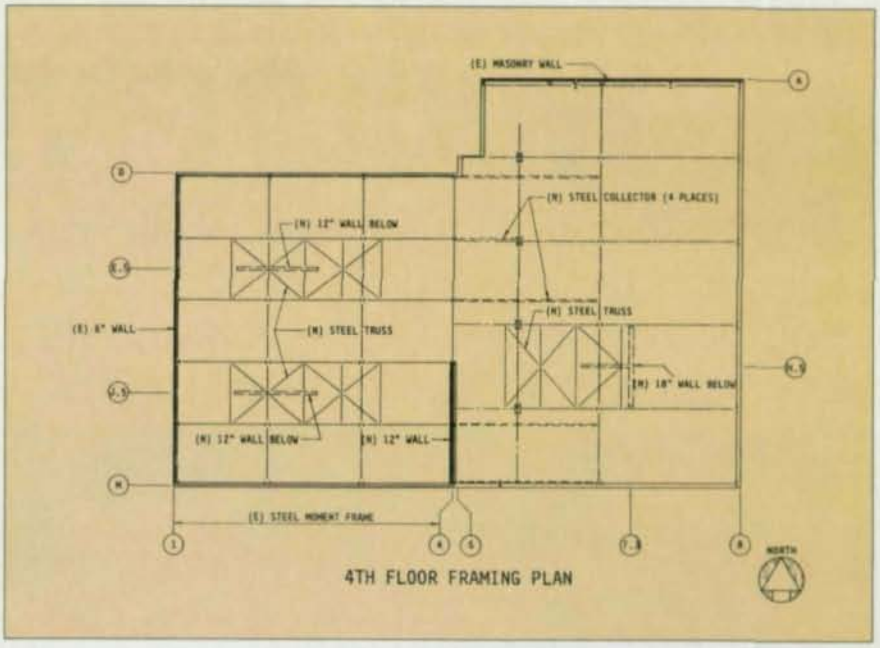
The original seismic strengthening proposal was for new concrete shear walls extending the full height of the building in both directions. While this would have brought the building into conformance with the 1988 UBC, it would have been extremely costly and also would have interfered with the leasability of the building.

Instead, an approach was adopted that achieved a greater compatibility between the old concrete building and the 1956 addition.

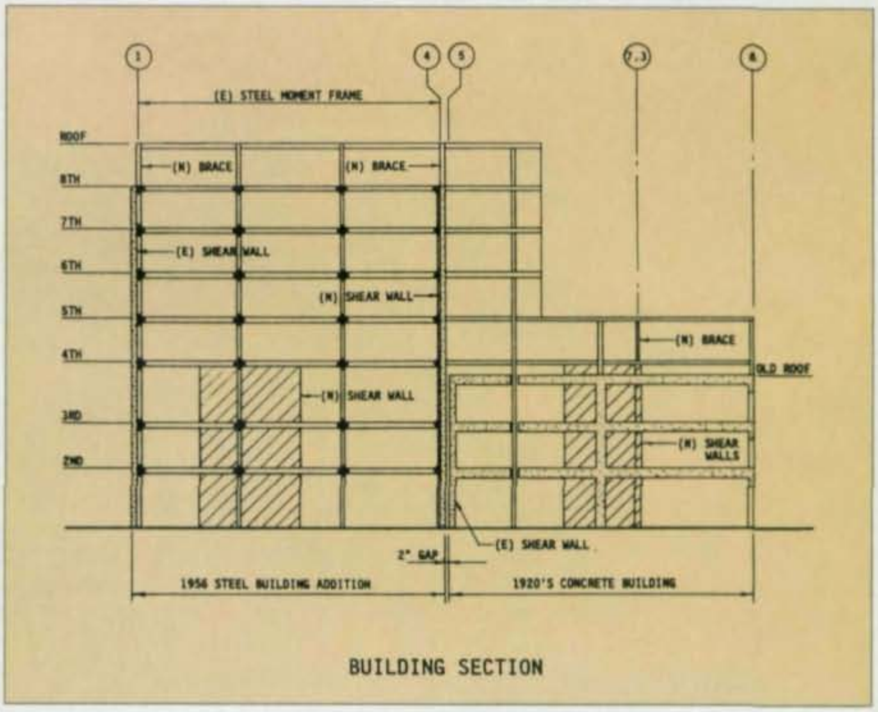
Upgraded Lateral System

For compatibility, the steel building required concrete shear walls in the bottom four stories in the east-west direction. Fortunately, after adding the new shear walls the steel moment frame had

Continued on page 32



4TH FLOOR FRAMING PLAN



BUILDING SECTION

MAKING THIS BUILDING S
WOULD GIVE MOST CO

But not Vulcraft. We saw it as one of our greatest challenges ever. Because we not only supplied steel joists and joist girders for the project, we also helped design the framing system so that only limited structural damage could be expected from an earthquake measuring up to 7.5 on the Richter scale.

The Evans & Sutherland building, which contains millions of dollars worth of sensitive computer equipment, is located within a mile of the Wasatch Fault in Salt Lake City.

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.

The 114,000-sq.-ft. Broadlake Plaza building in Oakland was seismically upgraded to ensure its continued survival in an earthquake-prone region.

sufficient strength to resist 75% of the 1988 UBC seismic lateral forces, with some strengthening at the eighth floor.

The existing beam-column moment connections were shop riveted and high strength bolted in the field. Shear connections were made at the beam webs with double angles. The top and bottom beam flanges were bolted to the column flanges with split-beam tees.

Field surveys revealed that the flanges of the penthouse and the eighth floor beams were either inadequately fastened to the columns or not fastened at all. To provide lateral bracing for the penthouse, x-braces of square steel tube sections were welded to the existing steel framing. The eighth floor beam-column moment connections were strengthened by fillet welding plates onto the beam flanges and complete penetration welding to the column. Shop drawings, fabrication, and erection of this new steel was quickly completed at the owner's request to allow tenant improvements to start early on the seventh and eighth floors.

Also for compatibility, the masonry north wall needed to be disengaged from the fifth and sixth floors to allow for lateral drift above the fourth floor. The wall was stabilized for out-of-plane motion with push-pull connections commonly used in the architectural precast industry. A36 threaded rods bolt through the wall and onto a new steel angle welded to the top or bottom flange of an existing floor beam.

In the east-west direction, the fourth floor diaphragm transfers large lateral forces out of the steel moment frame into the new concrete shear walls. The existing metal deck and concrete fill diaphragm was overstress, so a series of new horizontal steel trusses were designed to transfer these large forces to the walls. The truss system is welded to the underside of the fourth floor framing.

The web members are new WT4's and the existing wide flange girders act as chord members. The WT web members are welded to a large gusset plate at the center of each truss. Where a gusset plate is located within a new concrete wall, two 6" diameter holes are cut in the plate to facilitate the flow of concrete when the wall is cast. Forces are transferred into the wall by a pair of 7" x 4" x ½" ledger angles bolted through the wall with A307 bolts. The steel was erected prior to casting the new concrete walls.

For additional strength, a new concrete shear wall was added at line 4 and a new "T"-shaped concrete shear wall was added in the old concrete building. The new wall at Line 4 was designed for composite behavior with the existing steel frame by welding #6 dowels to the column webs and beam flanges. The walls were then shotcrete into the frame bays.

The steel wide flange columns became ideal boundary members for resisting the large tension forces present at the wall ends due to the seismic overturning. The existing column splices, which were bolted connections were inadequate to carry these tension loads. The bolted connections were replaced by complete penetration welds at the column splices.

The existing wide flange beams at Line 4 are used as collectors to drag the seismic lateral forces into the new wall. Where the existing beam-column bolted connections were inadequate to transfer these drag forces to the next beam, new tension tie plates were added. The column web was slotted to pass through a ⅝"-thick plate along the side of the beam webs. The tie web, and the new high strength bolts, were installed through the beams and tie plate to carry the gravity loads.

Foundations

The new shear wall footings have to resist uplift forces due to seismic overturning of the walls. For new construction, precast concrete piles would typically be driven. However, for this project there was not enough overhead clearance in the existing building to do this. Drilled piers also were considered, but the footings would have been enormous, the pier rebar cage would need to be spliced, and there could have ground water to pump out.

The solution was to use driven 12"-diameter steel pipe piles. The pile driving could work with an overhead clearance of as little as 12', and the overall footing sizes were smaller than the drilled pier alternative.

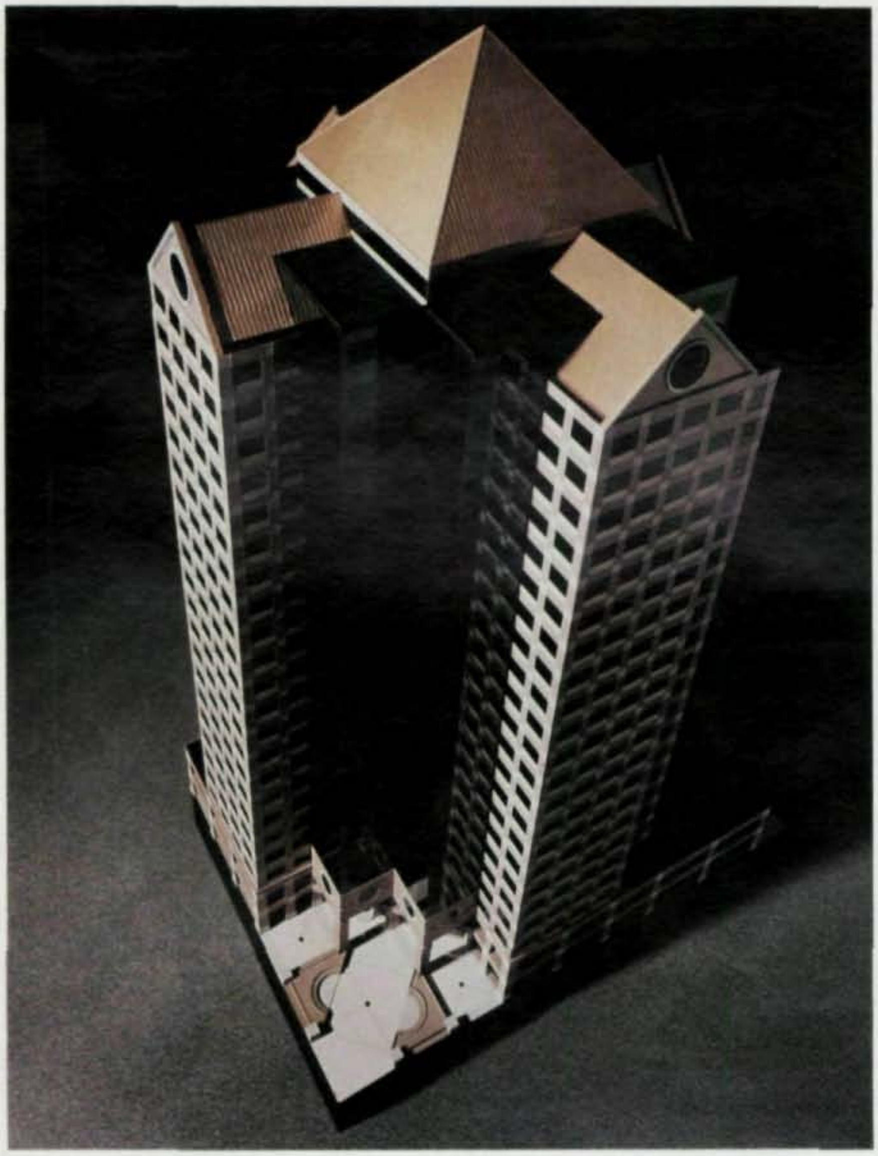
Costs

The lateral system combining structural steel and reinforced concrete proved to be the fastest and most cost efficient solution to seismically upgrade Broadlake Plaza. The structural upgrade cost approximately $7 per sq. ft., or less than 10% of the structural cost for new office construction. ■

William A. Andrews is a project engineer with the structural engineering firm of Vogel & Meyer in Walnut Creek , CA.

Balancing Seismic Forces

One Utah Center required a second row of columns in its saw-toothed sections to solve torsional problems



One Utah Center in Salt Lake City was designed with a saw-tooth design to increase the number of corner offices and to create an attractive street-level plaza. Photography by Paul Trentelman

If every building was a simple rectangle, designing for an earthquake zone would pose no special problems. Of course, then our cities would look like some sort of Modernist nightmare.

One Utah Center in Salt Lake City will never be mistaken for a Modernist structure. The 24-story glass-and-stone building features a saw-tooth front face on the corner of a busy downtown intersection. The design of the building reflects both the owner's desire for a signature building and the prevalent nearby architecture, according to Sean Onyon, AIA, a partner with Valentiner Architects, Salt Lake City. Project developer was The Boyer Company, Salt Lake City.

To respond to the design criteria, the architect adopted several themes from nearby historic buildings, including punched windows, cornices and the use of exterior stone and masonry.

The 443,274-sq.-ft. building has three major elements: a 150′ x 230′ two-story base with recessed windows and a cornice across the top; a 150′ x 150′ columnar shaft with punched windows; and a pyramidal top with gables. "We developed the saw-tooth design to increase the number of corner offices and to give some visual relief to the street corner," Valentiner said.

Special Moment Frame

One Utah Center was designed with a steel Special Moment Frame. "It's a new designation in the UBC and it refers to special detailing in the joint and column web interfaces that allows a design below required earthquake loads," explained Lawrence D. Reaveley, vice president of Reaveley Engineers & Associates, Inc., Salt Lake City.

"The analysis takes place on the column web," he continued. "You analyze that joint and if the thickness is not great enough, you add a doubler plate to stiffen that element. It's used primarily to control overall building drift. You also do a yield line analysis of the steel joints."

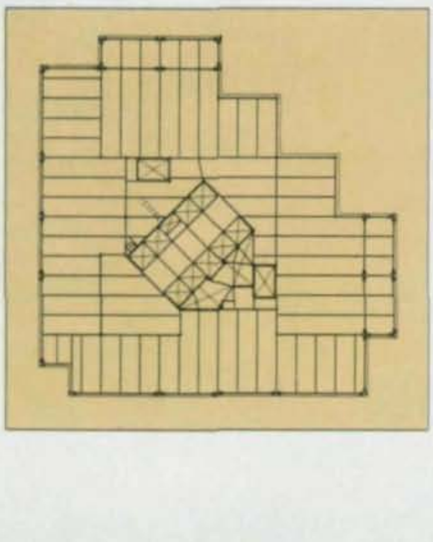
Special Moment Frames are most commonly seen in California, but their use is increasing in other seismically-conscious locales, such as Charleston, St. Louis, Boston and Seattle.

Torsional Problems

The building's unusual siting resulted in torsional problems that were accentuated by the structure's notchbacks, Reaveley said. To solve the torsional problem and eliminate the stiffness differential, it was necessary to have the same number of columns on each side of the building. Obviously additional columns couldn't be placed on the perimeter of the building without upsetting the building grid and window placement. Instead, a second row of columns was placed one bay behind the perimeter columns (see floor plan). Earl Eppich, P.E., was Reaveley's project engineer for seismic design and Mark Harris was project engineer for the structure's horizontal framing.

"We coupled those frames together to make a stiff box," Reaveley said. "Unfortunately, this required the placement of one column on the interior where the architect would rather not have had one."

The alternative to the second "layer" of columns would have been to turn all of the building's joints into moment connections instead of limiting the moment connections to the perimeter. "It would have been very expensive,"



The major challenges presented by the One Utah Center project were the saw-tooth design that required an additional row of columns (left) and the pyramidal top (top and above). The center peak of the pyramid has no column, and instead functions almost as an arch. Photography by Paul Trentelman

Reaveley said.

X-bracing couldn't be used because the core was twisted in response to the corner design of the building. Also, the core was fairly small—a healthy 72' x 72' in the low-rise section but only 38' x 50' in the high-rise portion.

Most of the structure was designed using ASD, though about 35% of the horizontal members were designed using LRFD, which allowed a significant weight reduction in the steel. "We tested the difference between LRFD and ASD and we felt comfortable reducing the weight of some members," Reaveley said. "However, we had some 35' spans and we were worried about serviceability and vibration, so we used heavier members in that area."

Fabrication and erection progressed very smoothly, despite severe space restrictions at the jobsite that virtually eliminated any area for off-loading and shake-out, according to Thomas G. McAllister, executive vice president with AISC-member Mark Steel Corporation, Salt Lake City. Erector was AISC-member Derr & Greunewald Construction Co., Henderson, CO. Erection commenced on Feb. 12, 1990 and the building was topped out on July 20.

4,150 Tons Of Steel

The main moment frame consists of 26 column sections designed as W36 sections reinforced with four plates on the inside of the flanges. The W36 x 150 column sections and W36 x 516 beams both are A572-50 steel. The balance of the structure includes approximately 250 column sections and 2,400 beams of various sizes, both of A572-50 and A36 steel. The structure used 4,150 tons of steel.

"We elected to fabricate the plate-reinforced columns entirely from A572-50 plate as a means for some cost savings, but primarily for substantially simpler welding procedures and better quality control," McAllister said. The continuity plates in the moment frame columns were installed using the Verti-Shield welding process, thereby greatly reducing welding time while consistently maintaining weld quality.

To save time and labor, the erector opted to use A325 tension control bolts in field connections for the floor beams even though the cost per bolt was somewhat higher than the hex-head variety. "A325 tension control bolts were used for shop installation of the connection angles for the floor beams, resulting in a time savings of about 30 minutes per beam when compared to welding. General contractor on the project was Jacobsen Construction, Salt Lake City.

Increased Live Loads

While the UBC required the structure to be designed to a minimum of 50 lbs./sq. ft. live load plus 20 lbs./sq. ft. for partitions, the engineers managed to add 10 lbs./sq. ft. to the live load at a minimal cost by adding some steel studs for composite action. "For this particular layout, the sizes were minimal and the cost for the added reinforcement was only \$0.03/sq. ft.

Because some of the tenants are expected to be law firms, the engineers are expecting to have to come back later and design reinforcement for library and other heavy file locations. "We finished another 20-story building four years ago and we are still asked back to reinforce different sections," Reaveley said. "The flexibility of steel is crucial to this type of design."

For spot reinforcement, the loading conditions are analyzed and a plate is added to the bottom flange of a beam. "We'll probably use an LRFD analysis, which will give us improved load factors and reduce steel use by up to 10%," he explained.

The project was further complicated by the low-rise extension on one side. "To avoid any problems, for the lateral force in the north-south direction we used a hinged connection," Reaveley explained.

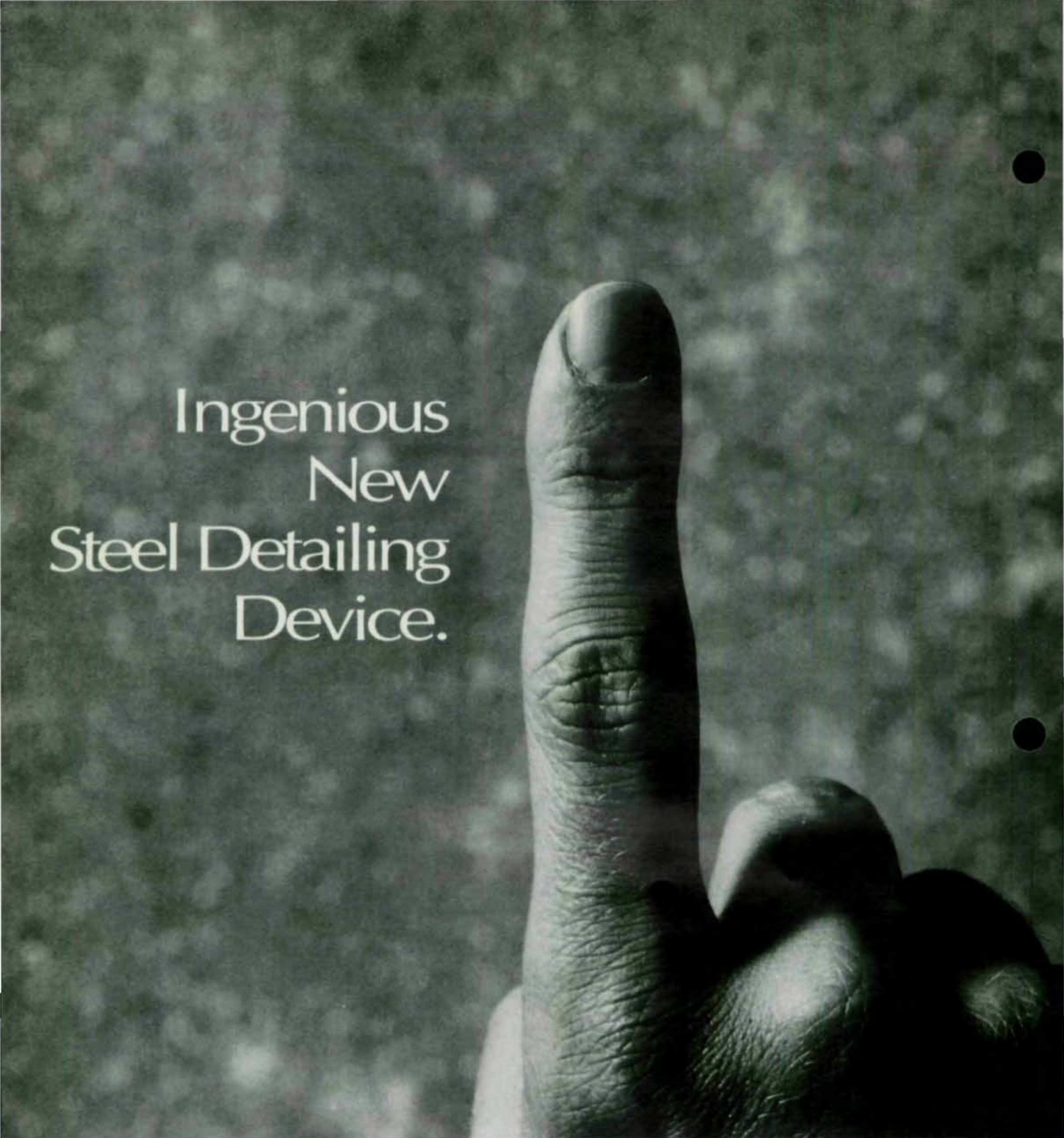
One Utah Center is designed with a Special Moment Frame, a designation that refers to the building's special detailing in the joint and column web interfaces that allows a design below required earthquake loads. Photograph by Paul Trentelman

The column frame in the north-south direction is flexible, with the low-rise coupled to the high-rise and the high-rise taking all of the load without a stiffness discontinuity."From a seismic point of view, the low-rise section is just a load mass. We didn't need expansion joints. Otherwise we'd be looking at a very irregular building with a very abrupt stiffness change.

The building's pyramidal top also created a structural challenge. "There's a lot of articulation of framing at the top of the building," Reaveley said. "It's almost a hat on the moment frame."

The center peak of the pyramid has no column, and instead functions almost as an arch. "It's supported on a moment frame for two floors below to a diaphragm that allowed us to shift the forces to the outside," he explained. "It required a truss diaphragm on the 24th floor to distribute the wind and seismic forces at the top."

Construction cost for the building shell was $22 million, and project cost with finishes was $40 million.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EARTHQUAKE DESIGN

Seismic Isolation:

An Alternative For Earthquake Design

By Stephen M. Weissberg, P.E.; David R. Van Volkinburg, S.E.; Ronald L. Mayes, P.E.; Lindsay R. Jones, P.E.

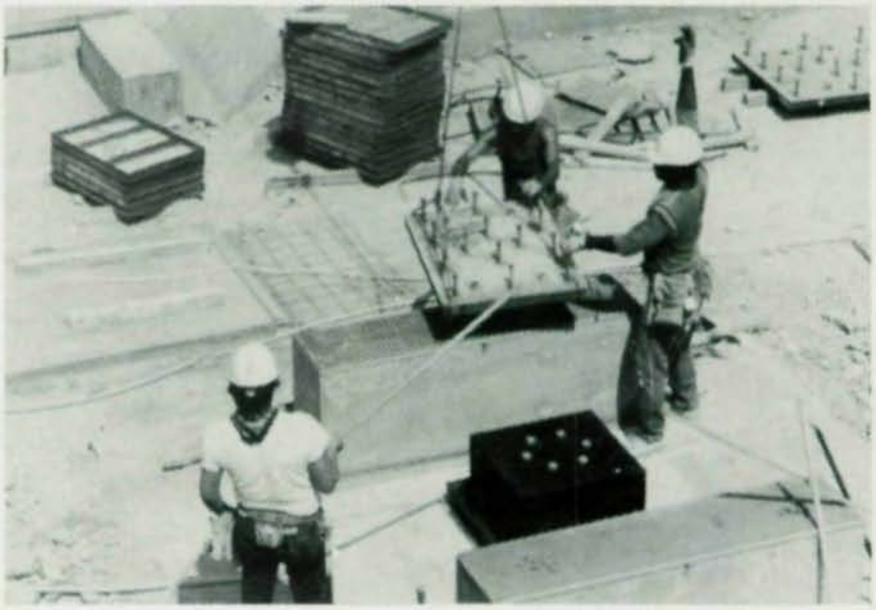
An innovative strategy called "seismic isolation" provides an economic alternative for the seismic design of new structures and the rehabilitation of existing buildings, bridges and equipment.

Rather than resisting the large forces generated by earthquakes, seismic isolation decouples the structure from the ground motion, providing the ability to reduce earthquake forces by factors of five to 10. In simplistic terms, this is equivalent to reducing the effect of a Richter Magnitude 8 earthquake to an event in the 5 to 5.5 range. The technique involves the separation of the superstructure from the substructure using reinforced elastomeric isolators ("seismic shock absorbers").

Conventional construction techniques can cause very high floor accelerations in stiff buildings, and large interstory drifts in flexible structures. These two factors make it difficult to ensure the safety of the building components and contents.

Mounting a building on an isolation system can prevent most of the rapid horizontal movement of the ground from being transmitted to and amplified by the structure above. This results in a significant reduction in floor accelerations and interstory drifts, thereby providing protection to the building contents and components.

More than 125 structures have been constructed worldwide using the principles of seismic isolation and about 20 of these completed structures have been subjected to real earthquakes, with the largest being the Richter Magnitude 7.1 1989 Loma Prieta earthquake in Northern California. All have shown the force and damage reductions expected.



The base isolation system for the hospital at the USC Health Sciences Center in Los Angeles is designed to act as a "seismic shock absorber."

Basic Concepts

There are three basic elements in any practical isolation system:

- A flexible support (spring) so that the fundamental period of vibration is lengthened sufficiently to reduce the force response;
- A damper or energy dissipator to limit the relative deflections across the flexible support to a practical design level; and
- Rigidity at low (service) load levels such as wind.

Flexibility: An elastomeric isolator is not the only means of introducing flexibility into a structure, but it certainly appears to be the most practical and the one with the widest range of application.

The idealized force response with increasing vibrational period (flexibility) is shown schematically in the acceleration response curve of Figure 1. For most earthquake ground motions, reductions in base shear occur as the period of vibration increases. The extent to which these forces are reduced is primarily dependent on the nature of the earthquake ground motion and the period of the fixed base structure. However, as noted above, the additional flexibility needed to lengthen the period will give rise

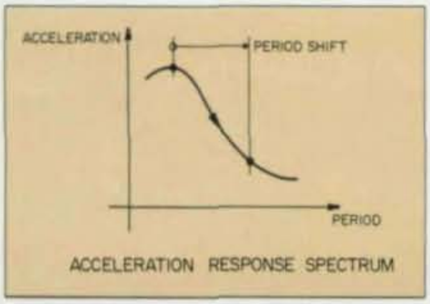


Figure 1.

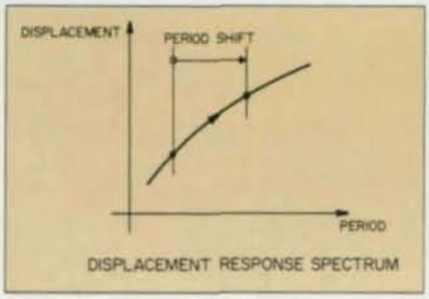


Figure 2.

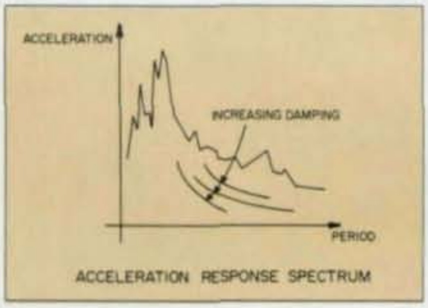


Figure 3.

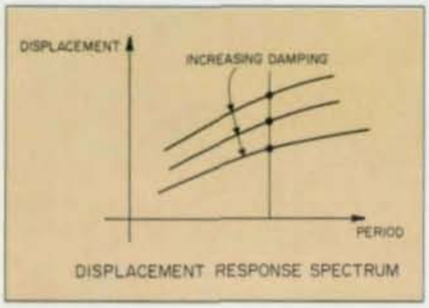


Figure 4.

to large relative displacements across the flexible support.

Figure 2 shows an idealized displacement response curve from which displacements are seen to increase with increasing period (flexibility).

Energy Dissipation: Large relative displacements can be controlled if substantial additional damping is introduced into the structure at the isolation level. This is shown schematically in Figure 3 and 4. Also shown is the smoothing effect of increased damping.

Rigidity Under Low Lateral Loads: While lateral flexibility is highly desirable for high seismic loads, it is clearly undesirable to have a structural system which will vibrate perceptibly under frequently occurring loads such as wind. Mechanical energy dissipators may be used to provide rigidity at these service loads by virtue of their high initial elastic stiffness.

Seismic Isolation Design Principles

The design principles for seismic isolation are illustrated in Figure 5. The top curve on this figure shows the elastic forces i.e. demand, which will be imposed on a non-isolated structure from the new Structural Engineers Association of California *Recommended Lateral Force Design Requirements* (SEAOC Blue Book) for a rock site if the structure has sufficient strength to resist this level of load.

The lowest curve shows the forces for which the Uniform Building Code (UBC 1988) requires a structure to be designed and the second lowest curve shows the probable strength, i.e. capacity, assuming the structure is designed for the UBC forces. The probable strength is 1.5 to 2.0 times higher than the design strength because of the design load factors, actual material strengths which are greater in practice than those assumed for design; conservatism in structural design; and other factors.

The difference between the maximum elastic force and the probable yield strength is an approximate indication of the energy which must be absorbed through ductile deformation in the structural elements. However, when the building is isolated, the maximum forces are reduced considerably due to period shift and energy dissipation.

The forces on a seismically isolated structure are shown by the small dashed curve of Figure 5. If a seismically isolated building is designed for the UBC forces in the period range of 1.5 to 2.5 seconds as shown in Figure 5, then the probable yield strength of the isolated building is approximately the same level as the maximum forces to which it will be subjected. Therefore, there will be little or no ductility demand on the structural system and the lateral design forces are reduced to approximately 50% of UBC levels for a fixed-base structure.

Seismic Isolation Feasibility

Structures are generally suitable for seismic isolation if the following conditions exist:

- Subsoil does not produce a predominance of long period ground motion (such as that obtained in the lake bed area of Mexico City);
- Site permits horizontal displacements at the base in the order of 6 to 9";
- Structure is relatively squat;
- Lateral wind loads or other non-seismic loads are less than approximately 10% of the weight of the structure; and
- Fixed-base building period of vibration is less than 1.5 to 2.0 seconds.

Codes

The SEAOC State Seismology committee adopted in late 1989 an appendix to the SEAOC Blue Book Chapter 1, *General Requirements for the Design and Construction of Seismic-Isolated Structures*. These requirement have been approved by the International Conference of Building Officials (ICBO) for adoption in the 1991 Uniform Building Code (UBC).

The California Office of State

Wide Health Planning and Development (OSHPD) adopted *An Acceptable Method for Design and Review of Hospital Buildings Utilizing Base Isolation* in May 1989.

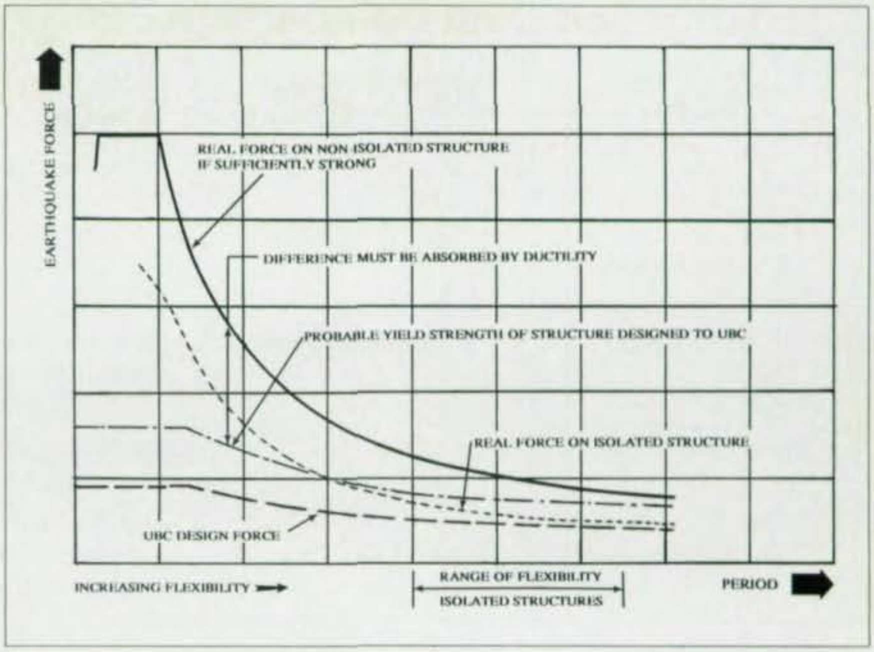


Figure 5.

Cost Factors For An Isolated Design

Cost Increases	Cost Reductions
•Seismic isolation system	•Savings in structural system
•Architectural details to accomodate isolation	•Savings in curtain walls (reduced drift requirements)
•Special mechanical and electrical details	•Savings in non-structural component bracing
•Additional design costs	

Economic Considerations

The economic feasibility of seismic isolation in building construction depends on an objective financial evaluation of the costs and benefits. Earthquakes have four principal cost impacts which may influence the decision making process in the selection of a structural resisting system: the initial cost of construction; earthquake insurance annual premium; physical damage that must be repaired after an earthquake to restore the building's pre-earthquake value; and disruption costs due to building and contents damage (lost rent, revenue, productivity), loss of market share or clients, and potential liability to occupants for their losses and injuries.

Since the performance characteristics of a conventional code designed building and a seismically isolated structure are significantly different, these four cost issues should be evaluated as part of the decision making process.

Construction Costs

Typically, the first costs of different structural systems are evaluated during the preliminary design phase of a project.

For codes that require reasonably stringent design force levels (e.g. hospitals and other essential facilities), it has been found that there is often sufficient savings in the structural system to offset the additional cost of the isolators and a new structural ground floor slab. For conventional buildings designed to the Uniform Building Code, the reduction in design forces achievable with seismic isolation is a function of the structural system (moment frame, braced frame, etc.).

The decision on the location of the plane of isolation will have a definite cost impact. If a sub-basement concept is used, an additional structural slab is required with a cost differential of a structural floor versus slab-on-grade. If the top-of-basement - columns/walls concept is used, there are no significant additional structural costs to incorporate seismic isolation and the net structural cost is the savings in the structural frame less the cost of the isolation system.

Another cost issue not generally accounted for is the cost of bracing requirements of nonstructural elements. In a California hospital this has been estimated to be in the range of \$2 to \$4 per sq. ft. more than that required for a UBC design. This issue can have a significant impact on the cost comparison if it is included in the first cost assessment.

Earthquake Insurance

Earthquake insurance is becoming more difficult to acquire in

Construction Cost Comparison For New Isolated Buildings

FACILTY	LOCATION	DESCRIPTION	COST
Fire Command & Control Center	Los Angeles	two-story steel braced frame	6% savings
USC Hospital	Los Angeles	seven-story + basement steel braced frame	0
Law & Justice Center	Los Angeles	four-story + basement steel braced frame	4% increase
Evans & Sutherland	Salt Lake City	four-story + basement steel moment frame	4% increase

high seismic regions and there is considerable concern within the Federal Government for the financial stability of the insurance industry should a severe earthquake occur in California. In an attempt to minimize their exposure, insurance companies have considerably increased insurance premiums and deductibles over the past five years. Deductibles of 10% of total building value are common and annual premiums are in the ⅓ to ¾% range of total building value.

If earthquake mitigation measures, such as seismic isolation, are able to reduce earthquake damage below current deductibles, e.g. $2 million on a $20 million building, then an owner has the option of incorporating these mitigation measures and saving the annual insurance premium ($100,000 - $150,000 per year on a $20 million building). This insurance premium savings will quickly amortize any additional first costs for the incorporation of seismic isolation. Sherwin Small, the Executive Vice President of National Medical Enterprises, the owners of the new seismically isolated USC University Hospital, said that "a small additional first cost to incorporate seismic isolation is a cheap insurance policy against a big earthquake."

Another important benefit of seismic isolation is its ability to significantly improve the chances of business survival after a major earthquake.

Business Disruption

In a recent article, Gary Jones, risk manager of Evans and Sutherland, discussed why seismic isolation was selected for his company's new manufacturing facility.

"We made an important decision even prior to starting the engineering of our new building: we were going to build to stay in business. One loss exposure that gave us the greatest concern was earthquakes. How could we protect our employees, our work in progress, our inventory, the building itself all the things that make up our business. We design and produce computerized flight simulators for NASA, the Navy, Air Force and airlines and, just like the other companies in the business, recognize a competitive window of opportunity. In other words, although our business may be disrupted, our competitors' may not be and we stand to lose our market share, our profits and our viability as a going concern.

"In 1987, when the Richter Magnitude 5.9 Whittier earthquake struck, it broke loose a concrete slab that killed a young pre-med student. A damage suit seeking $5 million has been filed against the State of California. Major issues in that suit will be the quality of the building's design and the State's responsibility to provide for the safety of students. The defendant could as well have been an employer. In any event, this catastrophe illustrates the type of claims that will surface after any natural disaster, particularly earthquakes. We opted for the viable alternative, seismic isolation, because we wanted something more than just minimum code standards.

"The risk of major earthquakes is real whether your building is new or existing. For existing facilities, business owners should begin immediately to assess the inherent dangers and to implement preparedness programs. Experienced business people know that the only thing harder than finding time to address exposure to loss is repairing the damage done." ■

Stephen M Weissberg is director of marketing with Dynamic Isolation Systems, Berkeley, CA., David Van Volkinburg is regional director of marketing, Ronald L. Mayes is president, and Lindsay R. Jones is executive vice president.

Information On Base Isolation Systems

Rubber Bearing Isolation Suppliers include:

• Dynamic Isolation Systems, Inc., 2855 Telegraph Ave., Suite 410, Berkeley, CA 94705 (415) 843-7233

• Fyfe Associates, Inc., 1341 Ocean Ave., Del Mar, CA 92014 (619) 792-1501

• Base Isolation Consultants Inc., 20 Jones St., San Francisco, CA 94102 (415) 863-2420

• Societe E.R.A., Residence "La Bruyere", Batiment B2, 2, Rue de l'Horticulture, 13009, Marseille, France

• Kajima Corporation, 2-7, Motoakasaka 1-chome, Minato-ku, Tokyo 107, Japan

• Skellerup Polymer Products Ltd., P.O. Box 19-555, Woolston, Christchurch, New Zealand

•Shimizu Construction Co., Ohsaki Research Institute, Fukoku Seimei Building, 27th Floor, 2-2-2 Uchisaiwai-cho, Chiyoda-ku, Tokyo 100, Japan

• Andre, Division of BTR Silvertown Ltd., P.O. Box 4, Horningglow Road, Burton-on-Trent, Staffs DE130ON, England

•Telesis International, 4605 Lankershim, Suite 710, North Hollywood, CA 91602 (818) 769-8894

•Bridgestone, Inc., IRP Division, P.O. Box 6147, Huntington Beach, CA 92815-6147 (714) 962-1665

Souce: Structural Engineers Association of California, Seismology Committee

Preserving An Investment

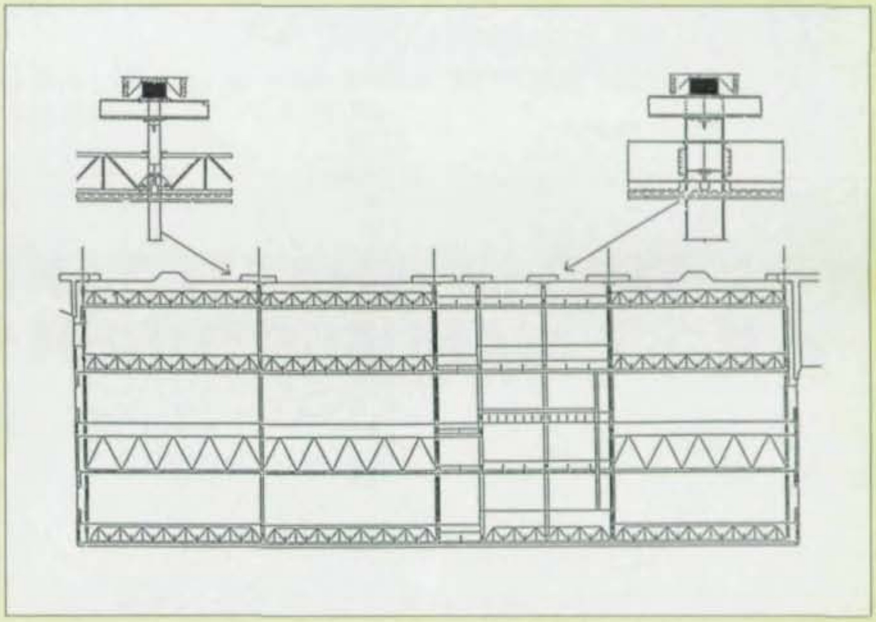
The Evans and Sutherland Building is a new, four-story manufacturing facility for computerized flight simulators used to train NASA pilots and others. Located near the Warm Springs and East faults in Salt Lake City, the building measures 280' x 160' in plan and rests on 98 *DIS* lead, steel and rubber isolators.

Lateral loads are resisted by a steel moment frame designed to satisfy UBC Seismic Zone 3 criteria. The value of the contents and work-in-progress in this building is estimated at $100 million, or approximately 12 times the cost of the structure. As a consequence, seismic protection of the contents of the building is of paramount importance to the owner.

In the early phases of design development, preliminary cost estimates were developed for both a conventional and an isolated structure. It was determined that the nominal cost premium required to incorporate seismic isolation as the primary mechanism to protect the contents of the building was very cost effective. The alternative means of achieving a similar level of protection was to require substantial bracing of the simulators during the manufacturing operation and this was considered impractical.

Architect for the project was Ehrlich-Rominger of Palo Alto, California, structural engineer was Reaveley Engineers and Associates of Salt Lake City, and geotechnical/seismological consultant was Dames and Moore of San Francisco. Dynamic Isolation Systems was the seismic isolation consultant and supplier of the isolation system.

The building was automatically modeled as a simplified moment frame and the isolators incorporated their nonlinear force deflection characteristics. Several computerized dynamic analyses were performed to assess the building's performance.



The Evans and Sutherland Building is supported on 98 base isolators. The building has a steel moment frame designed ot satisfy UBC Seismic Zone 3 criteria. While the structure is only $8.33 million, the building's contents and work-in-progress is valued at more than $100 million.

For the 475-year maximum probable earthquake, the maximum average deflection of a building corner at the isolation level was 2.6" with the maximum average deflection of the isolators equal to 2.4". The corresponding base shear was 0.10W. Conservatively allowing for a 50% increase for near-fault effects, the maximum design displacement for this event was 3.9".

For the maximum credible earthquake (1,000 year event), the maximum deflection of a building corner was 4.8" with the maximum average deflection equal to 4.7" and a maximum base shear of 0.16W. Again allowing for a 1.5 increase for near-fault effects the maximum design displacement for this event is 7.2". The isolation system has been designed for a maximum displacement of 9.0" at which time the building will act as a fixed base structure.

Floating Building Resists Earthquakes

Above ground, the Fire Command and Control Facility for the County of Los Angeles Fire Department is a steel braced frame, while below grade it is supported on 32 base isolator units. The building's metal facade is designed to accentuate it's structured elements.

Base isolation units in a new fire command station are designed to ensure the building remains operational during an earthquake

While building codes are primarily concerned with occupant safety, a growing number of designers are faced with situations where preserving a structure is nearly as paramount.

These structures include laboratories, power plants, delicate manufacturing spaces, and most recently, the Fire Command and Control Facility for the County of Los Angeles Fire Department. "It had to be designed beyond the code requirements to ensure that the building would remain operational during and after an earthquake," explained Thomas L. Anderson, general manager of engineering services for Fluor-Daniel, Inc.

"During the design phase, we looked at all of the realistic alternatives," he said. And essentially, those alternatives boiled down to either seismic hardening or seismic isolation.

"Fixed-base designs have proven historically to be very damage resistant in low-rise construction with either concrete shear walls or steel braced frames." However, the designers opted for a steel frame because it gave them more architectural flexibility, Anderson explained.

Fluor-Daniel developed two designs for the building, one with a braced-frame and the other using seismic isolation. While the two designs weren't identical, they were quite similar.

"We felt we could have achieved our purpose with either method, but we chose the seismic isolation design because it saved about $2 million," Anderson said. Most of those savings were not in construction costs, however, but in savings accrued by not having to strengthen non-structural elements such as electrical components. In addition, there are anticipated savings that will occur if the building does not have to be repaired after an earthquake.

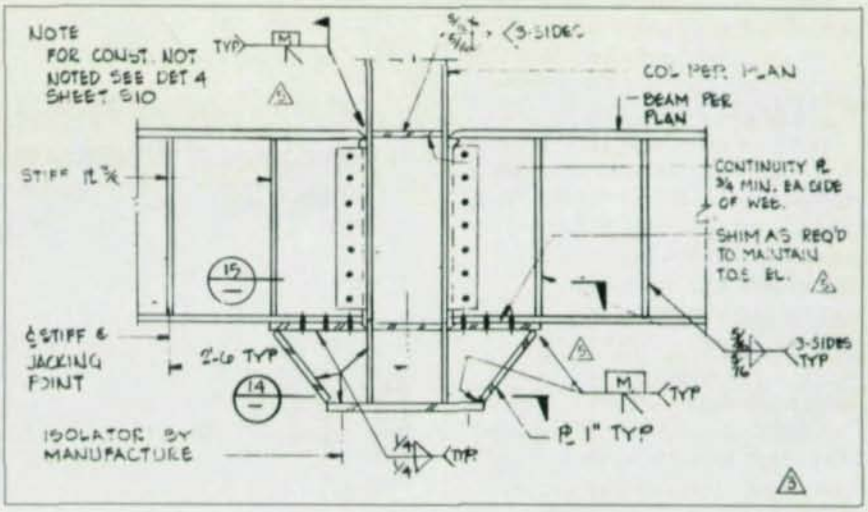
32 Base Isolators

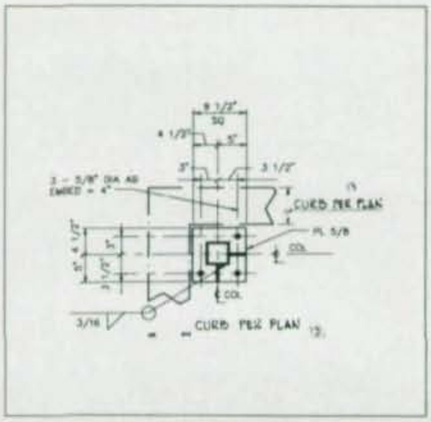
The 32,675-sq.-ft., two-story building rests on 32 base isolators. "The system we used has a 150 psi shear modulus strength at a shear strength of 100%, which was sufficient for the weight of this building," Anderson said. "The interior columns are carrying about 200 kips." The final design used 283 tons of A36 steel.

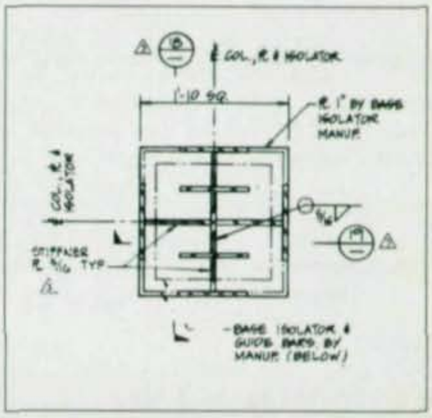
The natural rubber isolator bearings were supplied by Fyfe Associates, Inc., Del Mar, CA. The isolation system shifts the building's natural period from about 0.2 seconds as a fixed-base structure to about two seconds as an isolated structure. The period shift limits the base shear coefficient to about 0.1 for the maximum design earthquake, an event expected every 1,000 years.

Energy dissipation will be achieved by the isolators themselves as they go through shear deformation cycles. This is an intrinsic property of the rubber compound and eliminates the need for supplementary energy dissipation elements.

The estimated maximum differential displacement of any isolator during the design earthquake is 7".







Each base isolation unit was shipped to the fabricator where a steel base plate was bolted to the top of each isolator. Then, during erection, steel columns are welded to the base plate. A 1½"-thick base plate attached to the bottom of the isolator acts as a footing.

The metal cladding was carefully attached to the steel bracing to accentuate the building's structural elements and provide visual excitement.

The isolators, however, can accommodate displacements of up to 12". As a backup, at this point an ultimate restraint system built into the isolator units themselves is engaged, and it provides an additional 125% base shear capacity while limiting displacements to less than 15".

Light-Weight Structure

"We originally had some worry that a two-story steel building would be too light to work with the isolators, but our calculations showed the weight wouldn't be a problem," Anderson said. "We later found out that there are stiff rubbers available with shear modules of 60 psi that can accommodate a one-story steel building."

A steel base plate is bolted to the top of each isolator, and steel columns are welded to the base plate. Between the isolators and the bottom of the first floor is a 4'-high

The structural system needed to be elevated and isolated from the ground in order to completely isolate the building. As a result, the building appears to "float".

space. "To isolate the building, you can't have it sitting on the grade level," Anderson explained. "There's a tremendous PΔ movement below the first floor. We had to have very heavy reinforcement to brace the columns just above the isolator on up to the first floor framing."

As an added feature, in the center of the building the designers deepened the 4' high space to create an observation area. It is expected that schools and other groups will tour the building just as they would an ordinary fire house, and the observation area will provide them a view of the isolators.

Assembly Sequence

The isolators were shipped to the fabricators yard, where they were installed to the column assembly. Then a 1½"-thick base plate was attached to the bottom of the isolator. The base plate acts as a footing.

From the first floor up, the structure has a conventional concentrically-braced frame. "An isolated system requires very little drift so all of the deformations are concentrated into the isolators," Anderson explained. The building is designed to a more stringent standard than the existing codes.

The chevron braces are located only on the exterior frame. "The architecture was designed to accentuate the braces," Anderson added. An aluminum exterior building component was selected to complement the facility's high-tech, state-of-the-art functions, as well as to minimize possible repair costs associated with earthquakes.

Utility connections entering the building are designed with flexible connections to accommodate 15" of horizontal movement, although maximum expected seismic displacement is only half that amount.

Steel erection was very smooth, in part because during construction the steel columns could be shifted slightly since the isolators provided a measure of flexibility. One drawback, however, is that the building couldn't be tied off below the isolators because the structure must be kept independent from the isolators. "The building floats," Anderson explained. ■

LRFD/ASD Computer Data Base For Structural Shapes

The AISC Computer Data Base contains properties and dimensions of structural steel shapes, corresponding to data published in Part I of the *1st Edition, LRFD Manual of Steel Construction* as well as the *9th Edition, ASD Manual of Steel Construction.*

LRFD related properties, such as X1, X2, and torsional properties, are included in addition to ASD related values.

The program includes the Computer Data Base in ASCII format for the properties and dimensions of the following shapes: W Shapes; S Shapes; M Shapes; HP Shapes; American Standard Channels (C); Miscellaneous Channels (MC); Structural Tees cut from W, M and S shapes (WT, MT, ST); Single & Double Angles; Structural Tubing and Pipe.

Steel Connection Design Software (CONXPRT)

CONXPRT is a knowledge-based PC software system for the design of steel building connections. Three basic types of connections are included in Version 1.0: double framing angles, shear end-plates, and single-plate shear connections. More than 80 configurations are possible.

All designs are according to procedures in the AISC *9th Edition (ASD)* or latest available references. CONXPRT includes complete data bases for standard shapes, the structural steel, weld and bolt materials listed in the *9th Edition ASD Manual of Steel Construction.* All strength and serviceablility limit states and dimensional requirements for each design are checked. Help menus are included.

Provisions are available to set default values for particular project or shop needs, for example, detailing dimensions.

Steel Member Fire Protection Computer Program

STEMFIRE determines safe and economic fire protection for steel beams, columns, and trusses. It is intended for use by architects, engineers, building code and fire officials, and others interested in steel building fire protection. The software data base contains all the pertinent steel shape properties and many listed UL *Fire Resistance Directory* construction details and their fire ratings. In this manner, user search time is minimized and the design or checking of steel fire protection is optimized. (5¼" disks only)

WEBOPEN

This state-of-the-art software package is based on and includes the new AISC *Design of Steel and Composite Beams with Web Openings.* The program is designed to enable engineers to quickly and economically design beam web openings. The easy-to-use color coded input windows provide a clear, logical data entry system.

WEBOPEN was written by practicing engineers and incorporates "expert" design checks and warning messages that enhance the application of the AISC Design Guide to your design problems.

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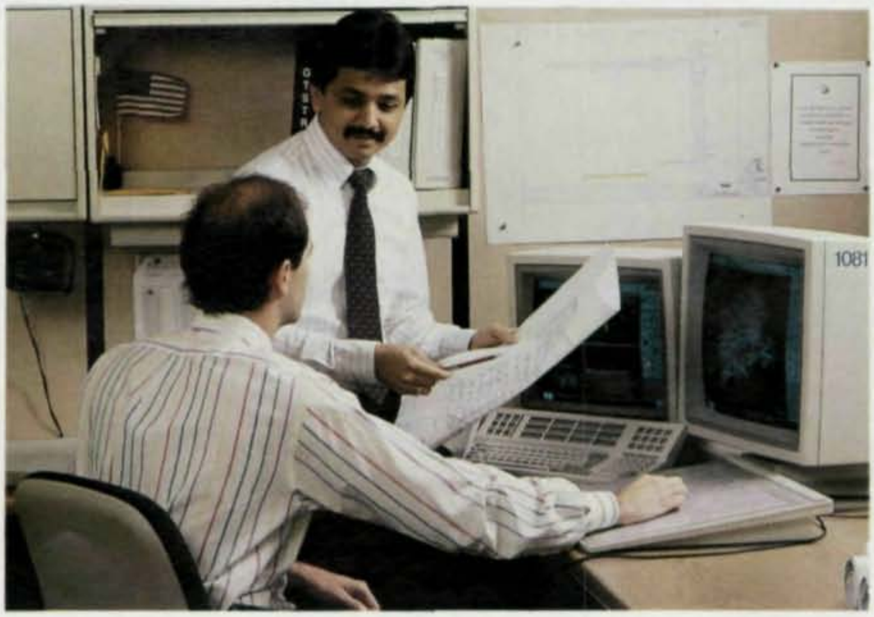
Electronic Transfer Brings Together Engineer And Fabricator

Direct transfer of information between design engineers and fabricators' computers increases production speed and accuracy

Increased use of computers has already made the production process faster than ever before for both engineers and fabricators. But to utilize computerization to its fullest, an electronic transfer between these two parties is essential. Instead of the hours—if not days—needed today to transfer data, the latest computer software allows almost instantaneous transfer.

"Anytime we can do something electronically, we see time savings and better quality," said Tom Schwarz, computer application coordinator of civil engineering for The M.W. Kellogg Company in Houston.

Through computer transfer, designers can send three-dimensional data including individual member specifications of structures to fabricators, eliminating possible human error and valuable re-input time. Jim Dager, president of Design Data in Lincoln, NE, a software developer for steel fabrication, said electronic transfer could soon become the industry norm. "Electronic transfer is not just a possibility for the future. At Design Data, we have developed an interface that will communicate with existing engineering software such as Du Pont's in-house system, SCADS, and we're discussing possible applications with users of Intergraph, another influential design system."



Fluor Daniel uses electronic transfer to increase accuracy and cut time. Pictured at a typical CAD work station are Robert Cipollone (seated), a structural engineer with Fluor Daniel and Sudhir Bhavsar, a designer with the firm.

According to Jack Kerr, president of AISC-member Ohio Valley Steel in Wheeling, WV, electronic transfer from designer to fabricator would save as much as six to eight weeks from production schedules. The time saved is especially important to fast-track customers building additions to house equipment for new product lines. "If we can cut six or eight weeks off their schedule, then that's six or eight weeks they can be making profits," Kerr said.

The increased accuracy provided through electronic transfer would allow projects to come on line faster, reducing overall costs, said Sayle Lewis, senior design engineer for Fluor Daniel in Greenville, SC. "If we can cut weeks out of a schedule, we're going to get structural steel to a job site faster," he explained. "This results in a reduced total project cost to the client."

Reducing human error by downloading directly to a fabricator's computer also would reduce redetailing and lower production costs, he said.

"Electronic transfer should eliminate all of the keyboard entry errors and reduce the time it takes to enter the data," according to Edward Easterday, president of E.J.E. Industries, Inc., Washington, PA. He did note, though, that while most detailing programs will kick out "bogus" data—for example, data about beam sizes that don't exist—programs will not recognize minor errors. "That's up to the engineer to catch in his data," Easterday said.

However, not everyone is convinced electronic transfer would result in substantial time savings. "If the data we import from an engineering program is right 99% of the time, we still have to check all of the data, and that eliminates most of the time savings," according to Stephen Roberts, marketing director of Dogwood Technologies, Inc., Knoxville, TN. Dogwood's detailing program runs on an Intergraph platform and can accept data from Intergraph-based engineering programs as well as other data in an ASCII format.

"Electronic transfer has the potential of misleading the industry," he opined. "But there is real promise for some market segments, such as for the oil business, for large warehouses, and for fast-tracked projects."

Electronic transfer could give engineers more lead time, explained Dick Hendricks and Bob Faccenda of Du Pont's engineering division in Wilmington, DE. The fabricators ability to deliver steel faster gives the designer the extra time to incorporate refinements into the drawings, making them more complete at the time of bid and resulting in fewer revisions.

Kerr uses the SDS/2 fabrication software from Design Data, which enables his company to quickly revise projects fabricated for Du Pont, a company that often uses fast-track construction.

"Drawings are sent to us to prepare detail drawings prior to the design being completed, and as a result, you're opening the door to a lot of changes that can really complicate a project from a fabricator's point of view," Kerr said. "But the computer helps coordinate those changes into the project and makes sure they're all made and interactive with the areas they impact." Still another advantage is the ability to accommodate and better manage revisions, he added.

Sharing data back and forth between designers and fabricators can aid the revision process. "Currently, we link layout and drafting models with analysis and design models," explained Lewis. "Fluor-Daniel is very interested in the ability to download this data." Ultimately, fabricators may be able to upload connection details back into the engineer's layout model, though Lewis said he was unaware of any software product currently on the market with this capability.

"It's becoming more an educational issue," he said. "People need to be educated as to what's available and what can be done. We can do some phenomenal things now that three to five years ago we never thought we'd be doing. I think it's going to change even more drastically in the next three to five years."

One of the main advantages of sharing information through compatible software is that it makes the relationship between fabricator and designer more of a partnership. "When there are a lot of design changes, relationships can get a little strained, as when a detailer prepares the drawings and then he has to go back and incorporate about 100 changes," said Kerr. "I feel we can eliminate a lot of that and view our relationship with the engineer as more of a partnership. It's an overall team approach to completing a project."

While direct cost benefits are sometimes difficult to prove, some of Schwarz's clients already are requesting that he do as much work as possible on computers. "They realize it increases quality and scheduling," he explained.

When exchanging information

electronically, the designers said the areas of responsibility does not change and that engineers are still liable for the design. "We're just utilizing a more advanced tool," Lewis said. "We're not going to have a change in responsibility just because we changed the medium. The present responsibilities of the engineer, detailer and fabricator do not change."

However, not everyone is certain that the liability issues have been completely clarified. "There could be a liability issue if a fabricator takes drawings directly from an engineer," said Norman Alterman, president of Computer Detailing Corp., Southampton, PA. "Is the engineer willing to take responsibility for any errors on the disk that he sends over?"

Added Roberts: "Most engineers won't let you use design drawings for erection, why would they let you use AutoCAD drawings for fabrication?"

While some issues still remain to be answered, many software manufacturers have either developed or are in the process of developing detailing programs that are compatible with engineering software. These software producers include:

- **Computer Detailing Corp.**, 1310 Industrial Blvd., Southampton, PA 18966 (215) 355-6003. Detailing programming works on an AutoCAD platform and can import AutoCAD and other compatible programs.
- **D.C.A. Engineering Software**, Inc., P.O. Box 955, Henniker, NH 03242 (603) 428-3199: Offers both engineering and detailing software on an AutoCAD platform. Can inport plans and elevations from its own engineering program, as well as STADIII, AutoSteel and SAP90 and other AutoCAD-based programs. Is working to expand its 3D interface.
- **Design Data Corp.(AISC Member)**, Suite 324, Gold's Galleria, 1033 O St., Lincoln, NE 68508 (800) 443-0782: SDS/2 software interfaces with Intergraph engineering software and Du Pont's SCADS engineering software.
- **Dogwood Technologies, Inc.**, 1900 Winston Road, Suite 407, Knoxville, TN 37919 (800) 346-0706: Software operates on an Intergraph platform and can interface with other programs running on an Intergraph base as well as programs that can output in an ASCII format.
- **ECOM Associates**, 8634 W. Brown Deer Road, Milwaukee, WI 53224 (414) 354-0234: Working on an integrated engineering/detailing package for connections that also will allow some integration with other engineering packages.
- **E.J.E. Industries, Inc.**, 287 Dewey Ave., Washington, PA 15301 (412) 228-8841: Has developed an external data interface that allows the transfer of data from any engineering package using ASCII coding once the format parameters are described.
- **KOPE-ING**, 1850 Fulsom, #1001, Boulder, CO 80302 (303) 449-2251: Steel-Pac detailing program can accept some data from AutoCAD and ARRIS programs; plans to expand to interface with other programs if client demand grows.
- **Mountain Enterprises (AISC Member)**, P.O. Box 190, 117 E. German St., Shepherdstown, WV 25443 (304) 876-3845: Plans on producing a combined engineering/fabricating package for engineers who do their own detailing.
- **Steelcad International**, 550 Alden Road, Suite 201, Markham, Ontario L3R 6A8, Canada (800) 387-4201: Can interface with CAD packages such as AutoCAD or FastCAD as long as the data is in a node-based system.
- **Structural Software Company (AISC Member)**, P.O. 19220, Roanoke, VA (800) 776-9118: Program currently only allows for ASCII transfer of data.
- **Vertex Design Systems**, 282 Second St., 4th Floor, San Francisco, CA 94105 (415) 987-2799: Vertex Detailer runs on an AutoCAD platform, so any file that runs on AutoCAD can be directly imported into the program. The Detailer includes more than 25,000 building components.

Steel Detailer

DCA Software is issuing its Release 11 of its Steel Detailer program to coincide with the availability of AutoCAD Release 11. The new program will take advantage of AutoCAD's powerful new features, including A.D.S. interface. The Steel Detailer couples structural information with parametric programs to quickly produce high quality shop fabrication, erection and bolt-setting drawings. Operating interactively within AutoCAD, the detailing system provides the user with total control over drawing production.

New capabilities include: improved bill of material functions; shear plates; connection angles; easier menu navigation; increased performance; and more powerful detailing routines.

New features include: moment preparation connections for plates and steel shapes; skew connections for bent plates and angles, end plates, and shear plates; ability to create and manage sections and details at any scale within the current drawing; improved steel database access that displays dimension and design properties; user defined bolt heads and nuts; horizontal sloped beams with five types of end connections; and a variety of stair design features.

Contact: Bob Southard, DCA Software, Inc., 7 Liberty Hill Road, Henniker, NH 03242 (603) 428-3199.

Structural Steel Shop Drawings

A new package of software programs for producing structural steel shop drawings is available from SSDCP. All of the programs are independent of each other, so not all programs need to be purchased. The parametric programs are written in LISP by a detailer with 35 years of experience. They do all drawing and calculations inside of AutoCAD Release 10 or greater. The main programs include: beams; columns; horizontal and vertical bracing; pan and grating stairs; stair rails; level rails; bracing plates; erection plans; and AB plans. Prices vary based on the number of programs purchased, but the entire package is available for $3,920. A free demonstration disk is available.

Contact: SSDCP, 110 Shady Oak Circle, Florence, MS 39073 (601) 845-2146.

Integrated Software

Steel Solutions has announced the release of Version 2.0 of the Steel 2000 software package for Structural and Miscellaneous Steel Fabrication. Steel 2000 is written in Foxpro—a relational data-based management system—and Version 2.0 is designed to take advantage of the program's extensive windowing capabilities.

Steel 2000 includes the ability to consolidate the material requirements for multiple projects and a warehouse operation into a single order list. The program tracks the details of the consolidated order through purchasing, receiving, cutting, fabrication, processing, and finally, shipping. Individual modules are available for Inventory, Purchase Orders, Mill Orders, Drawing Control and Estimating.

Contact: Steel Solutions, Inc., 2260 Florwood Dr., P.O. Box 1128, Jackson, MS 32915 (601) 932-2760.

3D Interface

Engineers can now download three-dimensional drawings to a computerized steel fabricator through an interface developed by AISC-member Design Data. Electronically transferring data saves valuable re-input time for the fabricator and increases the overall efficiency of the system.

The interface is applied to Design Data's SDS/2 software, which is used by fabrication companies to integrate estimating, detailing, and production.

Now the program can interface to the common structural data base (blue file) of engineering software developed by Intergraph and with a "neutral" file created by SCADS engineering software developed by the Du Pont Co.

Contact: Design Data, Suite 324, Gold's Galleria, 1033 O St., Lincoln, NE 68508 (800) 443-0782.

Detailing System

Dogwood Technologies has augmented the breadth of its Unix-based Procedural Detailing System with software that directly links detailing data to fabrication equipment. The unique design of PDS/FC removes the detailer from involvement in downloading tasks. Instead, as detailing progresses, Dogwood's software presents approved detailing output to a fabrication manager for composition of shop floor assignments. A simplified access program allows floor operators to select assignments, automatically performing the downloading of fabrication data at the critical moment.

This intelligently designed system provides greater accountability and reliability. Unix's multi-user capabilities allows activities to occur simultaneously even across even the largest shop.

Contact: Dogwood Technologies, P.O. Box 52831, Knoxville, TN 37950-9928 (800) 346-0706.

Interactive Fabricating

Data Management System's Interactive Steel Fabricating System is a series of eight on-line interactive modules designed and written for steel fabricators and related industries. The system consists of approximately 200 programs that can help fabricators be faster and more exact in bidding as well as more productive and cost effective in plant operations.

The Estimating system provides a variety of methods of input and output conforming to standard industry usage. The Bill of Materials system provides the user with: a bill of materials list; production records; sorted bill of materials; delivery tickets; production status reports; discrepancy reports; daily activity reports; and, most importantly, a cut list.

The Inventory system provides control of inventory at three levels: purchase; active (on-hand); and used. Inventory is maintained by AISC size, length, chemistry and reservation. The Job Cost module can trace all costs related to a job as well as calculate a percentage overhead to the job.

Contact: Data Management Systems, Inc., 12308 Twin Creek Road, Manchaca, TX 78652 (512) 282-5018.

Material Management

The Structural Material Sorter from E.J.E. Industries is a series of programs designed to aid steel fabricators and detailers in managing material lists. It is available for MS-DOS or Novell Network-based computers. The program is designed to reduce labor time and increase accuracy by computerization of labor-intensive tasks. For example, weights, surface areas, bolt counts and lineal totals are automatically computed for each item and the entire job without the need for manual calculations.

The latest release, Version 3.0, features an inventory tie-in that allows the Nesting Module to interface into in-house stock, vendor's stock, or the best combination of the two.

For more information or a free demonstration, contact: Edward F. Easterday, E.J.E. Industries, 287 Dewey Ave., Washington, PA 15301 (412) 228-8841.

Computer Assisted Sawing

INSCO's new ICAS software is designed to select the proper blade and correct feeds and speeds to maximize beam cutting efficiency. The program runs on IBM and IBM-compatible machines with DOS 3.3 or higher and costs $349.95. After a beam and a machine is selected from provided graphic illustrations, ICAS will specify: blade size; number of teeth; RPM; number of expected cuts; cutting angles for blade service; and sawing costs per beam or sq. in.

Contact: Insco Saw Division, 320 International Circle, Summerville, SC 29483 (800) 845-3816.

Steel Detailing

A collection of four AutoCAD-based programs—including Beams & Columns and Plans & Elevations—for steel detailing is available from Computer Detailing Corp. The system is versatile enough to create fabrication detail drawings of anything that can be fabricated. By using visual menus a detailer can dramitically increase his output with minimal training. Any type of fitting for beams and columns can automatically be created, inserted and dimensioned. Extension dimensions for the location of holes and fittings are automatically inserted. Version 4 includes a Bill of Material systems that not only adds a user formated bill on the drawing but also will create a consolidated cutting list of all material on the job.

Contact: Computer Detailing Corp., 1310 Industrial Blvd., Southampton, PA 18966 (215) 355-6003.

Details From Drawings

The ME2 System from Mountain Enterprises produces finished details from erection drawings built with easy on-screen menu choices, by direct entry of individual members, or through a combination of both. All programs are mouse-based for ease of data entry and immediate graphic feedback. Final detail sheet composition is completely automatic.

ME2 Version 3.0 offers: easy access to controlling every aspect of your details, including downloading of CNC data; a simplified but even more powerful Erection Drawing system; a greater variety of already computerized connection types; and enhancements to the ME2-CAD Compute and Draw system.

Contact: Mountain Enterprises, Inc., 117 East German St., P.O. Box 190, Shepherdstown, WV 25443 (304) 876-2534.

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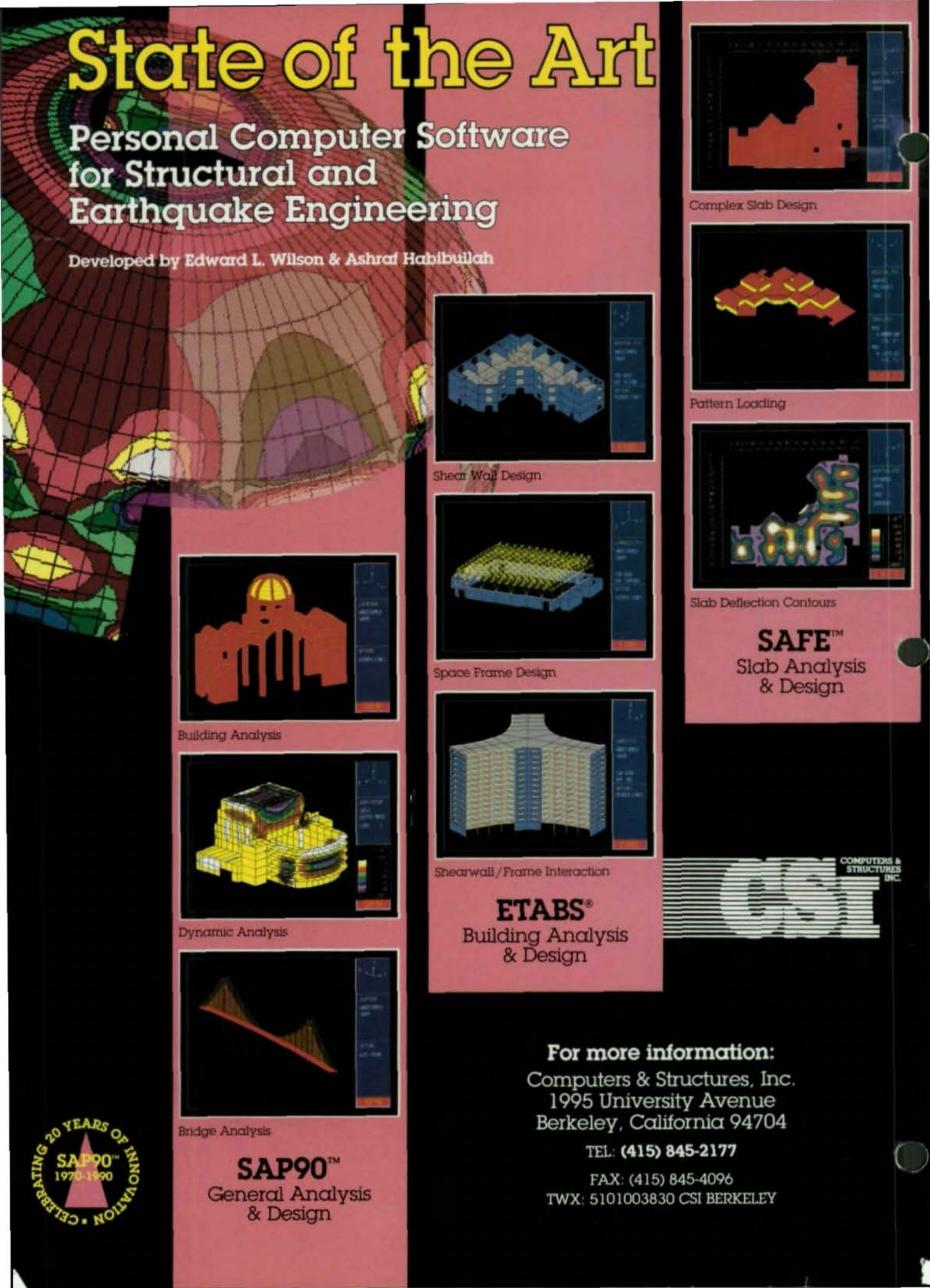
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