AMERICAN INSTITUTE OF STEEL CONSTRUCTION
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MODERN STEEL CONSTRUCTION

NUMBER 3 • 1987

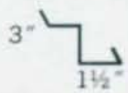
THIS ISSUE

DECK DESIGN DATA SHEET.

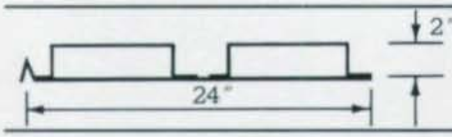
NO. 9

SPECIAL SHAPES

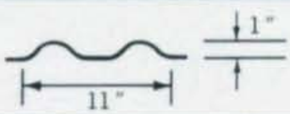
SPECIAL DECKS OR SPECIAL SHAPES ARE SOMETIMES REQUIRED FOR SPECIAL JOBS. WITH OUR BENDING EQUIPMENT AT UNITED STEEL DECK, INC. WE HAVE PRODUCED THE SHAPES SHOWN BELOW:



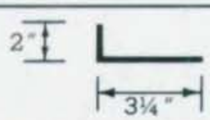
12 GAGE SUPPORT TRACK.



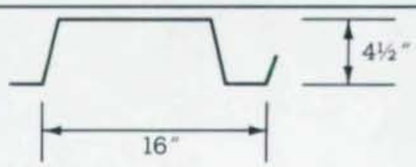
12 GAGE TOP HAT/ 16 GAGE BOTTOM PLATE HEAVY SLAB FORM.



¼" FORMED PLATE—HEAT SHIELD.



½" FORMED PLATE—SPECIAL ANGLE.



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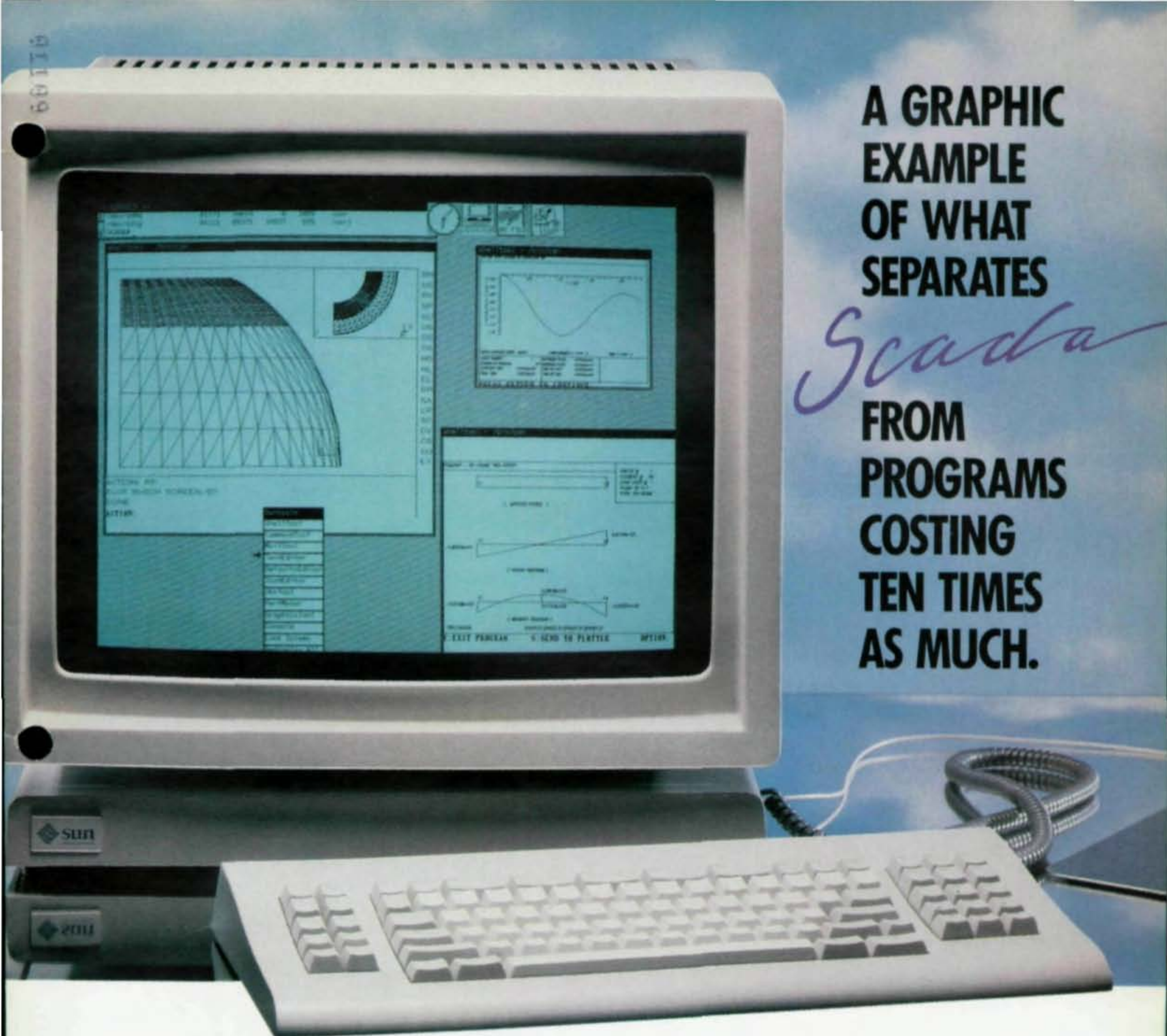

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MODERN STEEL CONSTRUCTION

VOLUME XXVII • NUMBER 3
MAY-JUNE 1987

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ON OUR COVER:
New Orlando, Fla. bus depot a marvel of steel-framed construction. Watch for feature in next issue!

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Steel Sculpture Revisited

In the No.1-1987 Issue of *Modern Steel Construction* a University of Florida-Gainesville sculpture was shown. In beautiful, living black and white, it didn't come off so well. So we thought you would like to see it in full color.

Prof. Duane Ellifritt taught the beginning Steel Design class, which was always followed by frustration because students could not visualize a simple web-angle shear connection. Contractors who would permit students on a jobsite were becoming fewer because of liabilities for injuries.

So, the next best thing was to take samples of construction details to the students. These details were designed and fabricated in Steel Fabricators' Ft. Lauderdale shop and moved to the site. The sculpture—yet to be named—was painted in the school colors, with the bolts painted black so they would stand out. Much attention has been drawn from passersby—and the minds of future designers have been impressed with some of the advantages of steel-framed construction. □

Remodeling with Steel

RENSSELAER RINK

Steel Beams and Steel Blades—a Winning Combination

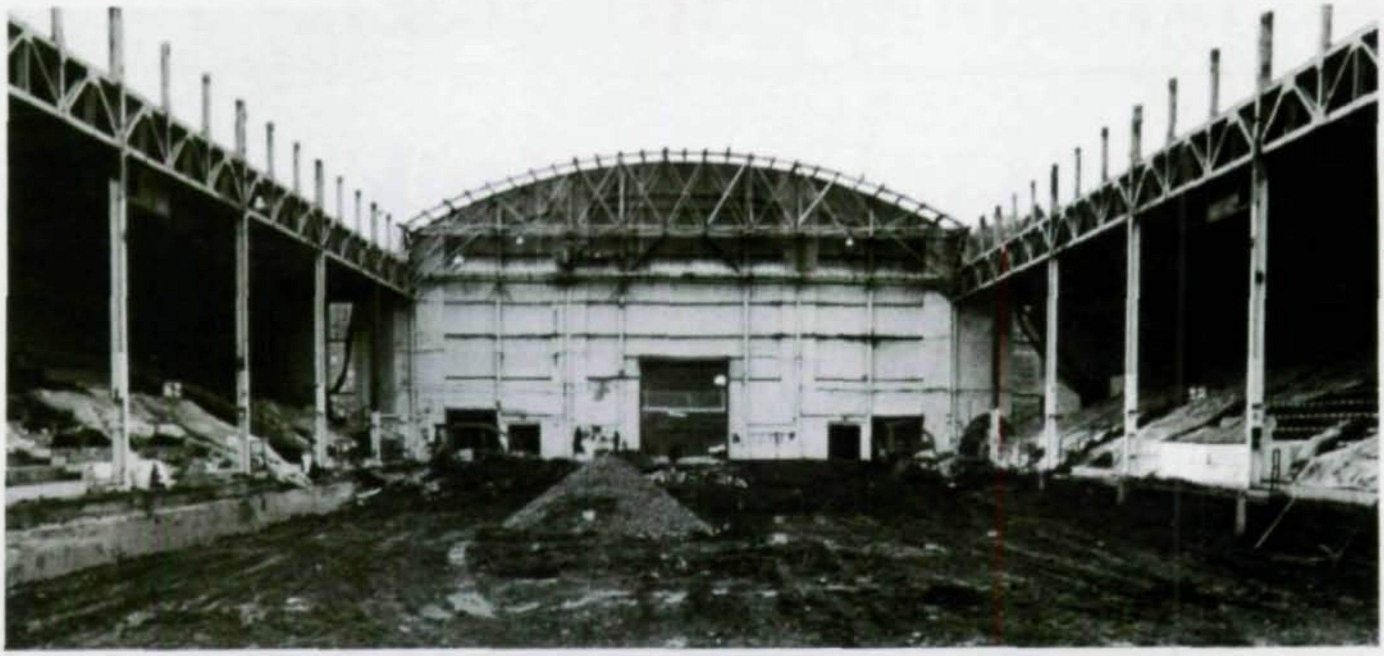
The Houston Field House at Rensselaer Polytechnic Institute in Troy, N.Y. is the major athletic facility for RPI's 6,000 students. RPI prides itself on being a good neighbor, and makes the facility available to the community for travelling performers. But back in 1981, the Field House was becoming an embarrassment to RPI, the oldest degree-granting engineering institution in the English-speaking world.

"The old battleship," as one consulting architect called it, was never a great facility, having begun life as a navy warehouse in WWII. RPI acquired it in 1949 and trucked it to Troy where it was refitted for hockey, RPI's only major sport. Since columns were posted at intervals of 36 ft around the perimeter of the rink, only 25 to 30% of the spectators had an unobstructed view. Also, Troy's location in the snowbelt meant the old roof was overloaded and had to be cleared with every snowfall.

Time was even more unkind to the rink. Over the years, poor soil conditions and

A championship rink—Rensselaer Polytechnic's remodeled hockey rink, Troy, N.Y. Now home to hockey and circuses.

"Old Battleship" needed major repairs, or total replacement. Old roof came off of Houston Field House. Steel came to rescue by providing column-free viewing areas.



the old refrigeration system created frost heaves under the floor which caused an uneven freezing pattern. Referees penalized the RPI team at home games by making them take the bad end of the ice—the slush. Bouncing pucks and catching skates frustrated both teams.

The hockey players were not the only ones having a difficult time. Ringling Brothers Barnum and Bailey Circus would not bring its prime acts to Troy because the roof was too weak and too low to handle the special loads. The Ice Capades group needed equipment and a stronger roof for curtain drops and lighting rigs. Announcers and musicians declined to perform in a place with such notoriously bad acoustics.

Steel to the Rescue!

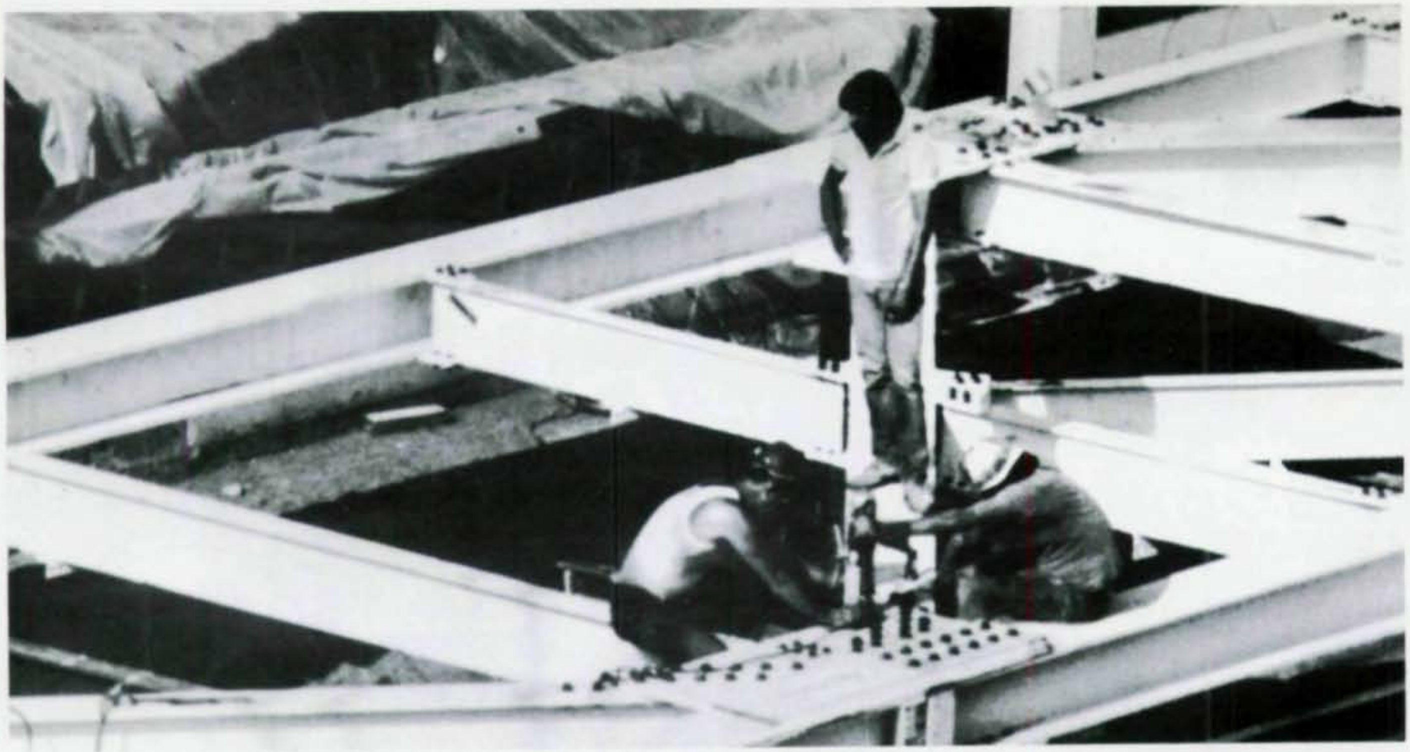
In the Fall of 1982, RPI contacted several architectural and engineering firms to see what could be done. Most wondered why the building had not been condemned—and recommended demolition as the first step. But Carlson Group of Boston had a better idea. Their engineers asserted they could renovate "the old battleship" by using advanced structural steel techniques. The group would deliver a facility with much improved sightlines, good acoustics, a safe, strong roof that could take the point loads, a good floor surface and smooth ice. Even better, the renovation would save RPI at least 50% over the cost of a brand new building. "Interesting!" RPI responded, "but can you get it off the drawing boards before this season's end and use the six and a half months between sporting seasons to put it in place?"

The group, equipped with up-to-date computer systems and accustomed to working together under a practiced plan of action, was able to meet both the tight deadlines and every other requirement.

In three months, from late December to mid-March, when construction began, the team developed project plans, architectural and engineering designs, a highly accurate cost estimate and a life-cycle energy cost analysis. Immediately after the last hockey game, bulldozers were at work.

Using horizontal steel trusses, each 240-ft long and weighing in at 21 tons, it was possible to redistribute the substantial building loads from both the spectator bay and main roofs to four large columns at the corners of the rink. These, in turn, were supported by a steel pile foundation. This marks a substantial improvement over the 16 smaller columns previously surrounding the rink. Now 95 to 98% of the seats have unobstructed views, and the seating capacity has increased by 10%.

The architect also designed and built a



Fabricators assemble 240-ft trusses (above). Each weighing in at 21 tons, trusses were key to fieldhouse renewal. Three cranes lifted each truss in place (r.).

new center roof which will withstand the heaviest winter storms. It was fitted with customized reinforcements for the rigging for Ringling Brothers and the Ice Capades. And, the Albany Symphony Orchestra performed in the Field House, now that an acoustical deck inconspicuously lines the roof between joists. Increased use of the facility by outsiders helps support RPI's hockey program.

Home to Champions—and Elephants

In 1985, the rink was home to champions. RPI won 30 of 32 games, tied one and lost one for the Eastern College Athletic Conference—then went on to win the N.C.A.A. championships. To rebuild the rink surface, contractors removed 2½ ft of poor top soil and redistributed the remaining material to better support the floor, which was lowered by 2½ ft. This move made the center arena even more visible from the stands and gave trapeze artists more room to perform.

Since the rink also had to sustain stresses from jumping elephants, pipes for the refrigeration system had to be laid 1½ in. instead of 1-in. below the surface and the concrete floor poured in a single slab.

Deadline Met!

And yes, the team met its deadlines! By October, the RPI hockey team was practicing happily on the new ice. According to coach Michael Adessa, "Not only did the improvement in the hockey rink radically improve our training program, but also it gave us a considerable edge in recruiting promising young players."

Indeed, the new Field House meets nearly everyone's standards. Ringling Brothers Circus brought its best acts to RPI. There are professional wrestling events, a performance by the Royal Lipizzaner Stallions, a visit from the Sesame Street gang, an increased number of rock concerts—and an appearance by a symphony orchestra who previously refused to perform in the old facility. □

Architect/Engineer/Contractor

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Cochituate, Massachusetts

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ALTERNATE DESIGN BIDDING LOWERS BRIDGE COSTS

by Stephen R. Simco

Pennsylvania, since 1980, has permitted contractors to submit their own alternate designs in bidding on new bridge construction.

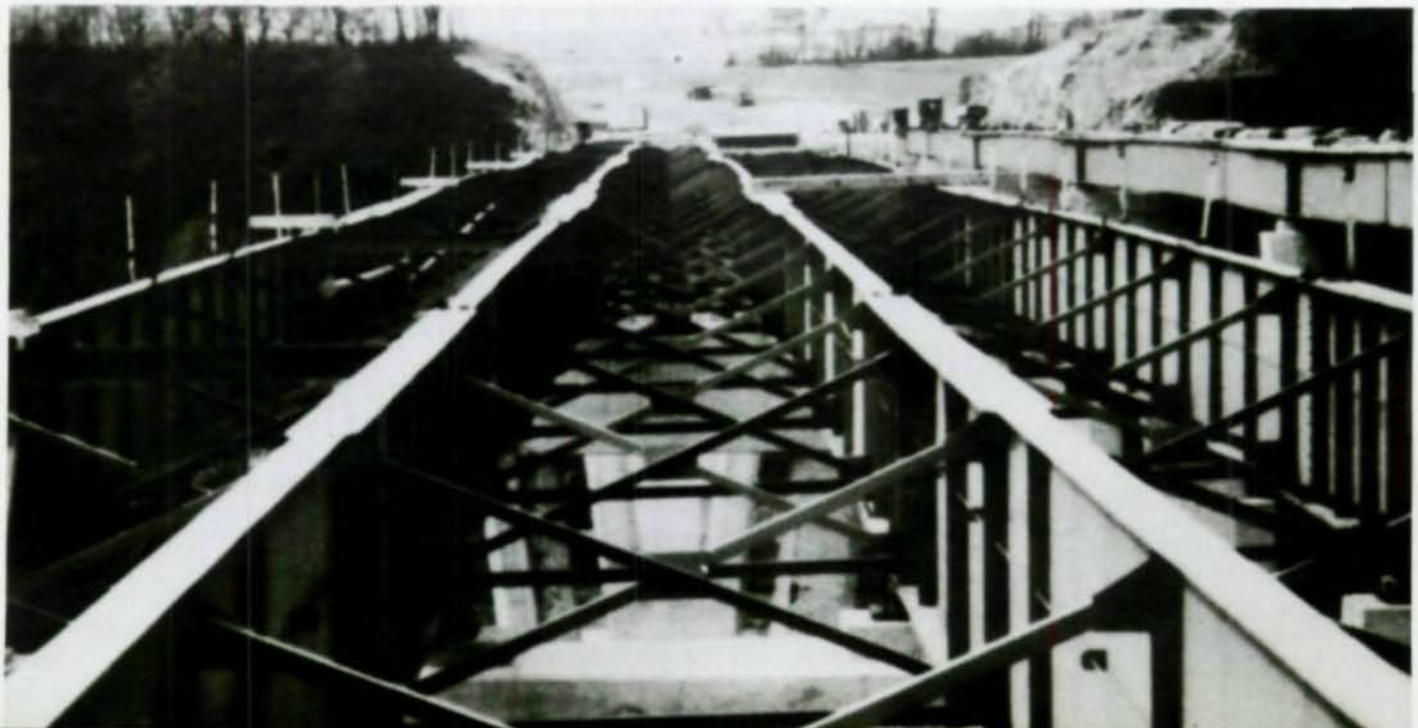
This policy has proven effective in reducing bridge costs without sacrificing safety, load capacity or durability. An analysis by the FHWA-Region 3 office of all bridge projects let to contract since adoption of the procedure indicates 32% of major bridges (those over $5 million) are being built with contractor-designed alternates. Non-major bridges are also being erected with contractor designs, but to a lesser extent (only 12%).

This feature explains how the procedure works, some key results, its advantages, drawbacks and other pertinent facts. Other states may wish to investigate and consider adoption of alternate design bidding, since it has worked well for Pennsylvania.

Cost savings are the primary objective. On the average, a 10% reduction was realized on major bridges, 7.2% on non-major bridges. Additionally, once contractors gained experience with the procedure, they became adept at obtaining bridge contracts by using innovative fabrication/erection techniques and refined designs. The program also permits the two materials industries—steel and concrete—to work closely with contractors in developing low-bid designs. Steel and concrete suppliers strive to improve the as-designed plan by adopting methods such as finite element analysis and redesign to take full advantage of the capabilities of their fabricating plants.

The precursor to the concept came from the FHWA which, in 1979, recommended alternate designs (now mandatory)—one steel and one concrete—for major federal-aid bridges. The goal was to foster competition between contractors and perhaps result in savings which exceed the additional

Alternate design bidding on I-78 bridge over Delaware River cut costs for Pennsylvania DOT. Average of 10% was realized on major bridges.



engineering fees. In 1980, Pennsylvania, under the leadership of Dr. Thomas D. Larson, secretary of Transportation, adopted a contractor alternate policy for all bridges.

Today, Pennsylvania, with more than 21,000 bridges on its state and local systems, is in the midst of a $1.6-billion bridge replacement/rehabilitation program. The program, commenced in 1983, culminating in 1991, covers 979 bridges, most of which are or were deficient major river crossings. PennDOT is also in the process of implementing a succeeding $1.7-billion, eight-year program, to cover about 3,000 smaller bridges.

However, in 1980, the financial picture was bleak, due primarily to severe cutbacks in gas tax-dependent revenues. Desperate for cost savings, PennDOT Secretary Larson believed the alternate design program would be the best opportunity to reduce costs and develop non-traditional, innovative ideas for bridge design and construction.

Space constraints limit mentioning the numerous details but, essentially, the procedure works this way. On major bridges, PennDOT provides plans for an as-designed steel bridge structure and an as-designed prestressed concrete bridge, with designs created by consultants or PennDOT staff. Only one design is prepared for non-major bridges. Prospective contractors can bid on these designs or present their own alternate design in steel or concrete, or a combination of both. We are obligated to select the lowest bid.

The only exceptions—and they are mentioned in bid documents—are cases where public commitments, site restrictions, aesthetics or environmental reasons dictate a particular structural type. Bridges with spans exceeding 150 ft are being built from steel designs, and those under 150 ft from precast, prestressed concrete designs. Our contractor-designed alternate policy has yet to produce a long-span, segmental, prestressed concrete bridge.

If the successful bidder uses an alternate, he must submit a conceptual design of it to PennDOT for approval within six calendar days of contract award. PennDOT reserves the right to reject it and request a resubmission of another, acceptable alternate. If an acceptable alternate is not presented within 30 calendar days—six days for the submission and 24 days for PennDOT's review—the contractor must build the as-designed structure at no additional cost. The contractor must also pay a share of PennDOT's design review expense, up to $5,000, depending on the bridge's cost. After a conceptual design is approved, PennDOT permits construction to proceed as soon as plans can be approved for part of the structure.

Now six years of experience leads us to these findings:

- At first, there was concern among engineers that owners would lose control over bridge appearance and/or the safety factor. The truth is that bridges built with contractor alternate designs still look like other bridges, and they all meet the demanded load-carrying capacity with the required factor of safety.
- The number of low-bid, contractor-designed bridges has increased over the policy's first years. In 1980, there were none; in 1981 - 6; 1982 - 7; 1983 - 14; 1984 - 20; 1985 - 22; 1986 - 13. As contractors become more familiar with what will be allowed, they bid more of their own, less expensive designs. They also develop working relationships with consultants who know a contractor's capabilities and can prepare alternate plans quickly.
- Computer-aided designs are assuming greater importance because of the time limitation in presenting the initial, conceptual design. In most cases, alternate bids came in below our as-designed cost estimates. As a consequence, we believe the program has brought us to the leading edge of the state of the art in

Two views of I-78 steel bridge.

bridge engineering and done it much faster than possible without the program. It forced us to do more new things in the last five years than in the previous 25 years.

- Among the most common contractor design changes is elimination of one or more lines of girders by increasing the spacing between girders. One winning steel bridge design, for example, cut six girders back to four, spaced the girders 14 ft-3 in. apart rather than the nine ft on the original design, and reduced the steel weight from 47.2 to 35 psf.
- The relatively low percentage of non-major bridges bid as contractor alternate designs may be due to the fact that many short-span bridges are designed using Pennsylvania's Standards for Low-Cost Bridges, which have already optimized many design parameters. Therefore, there would be little room for significant cost-saving alterations. Also, because small bridges involve a relatively small cost, there is less incentive for contractors to expend the time and costs necessary to produce another design.
- Nothing in the program prevents us from trying other ideas. For example, in December 1986, we let a design/construct contract for a major river crossing. Here, we prepared a steel and a concrete conceptual design, not a complete set of plans. We included all necessary restrictions: span length limitations, widths, flood clearance, foundation bearing pressures, pile sizes and loads, etc. The contractor then advanced the concept into a completely finished design and submitted a bid for the construction work. The winning bid was in steel.

As with any program, there are some drawbacks. On major bridges, the state may pay for three designs—our two as-designed plans plus a contractor's alternate, whose cost will be incorporated in his bid. Despite this, our findings show we still save money with lower bids. Also, the previously mentioned design/construct concept will significantly reduce such redundant design costs.

If the bid period is too short, the contractor may not have sufficient time to complete all of the details from which he can develop refined cost estimates. There must be a commitment of agency or consultant personnel at award time to expedite the review of contractor conceptual designs. The contractor also takes a risk in not knowing whether or not his design will be approved without material changes, which may have an impact on his costs.

The benefits of the contractor-designed-alternate program, however, far outweigh any disadvantages. We continually update our design manuals and standards to reflect the cost savings and concepts brought out by our policy. Eventually, our as-designed bridges should produce a low bid for the majority of the cases which are now being bid as contractor-designed alternates.

However, allowing contractor alternates will always be advantageous, if only to initiate and unleash the fullest creativity. We believe wholeheartedly any program which encourages use of the latest in design and construction technology, to produce more cost-effective and innovative designs, is well worth pursuing. □

Stephen R. Simco is chief bridge engineer, Bureau of Design, Pennsylvania Dept. of Transportation.

CALL FOR PAPERS

1988
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HILTON FONTAINEBLEAU HOTEL
Miami Beach, Florida
June 8-11, 1988

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ABSTRACT FOR PROPOSED 1988 NEC/COP PAPER/PRESENTATION
(See Reverse Side for Abstract Guidelines, Preparation of Final Papers, etc.)

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AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.
Call for Papers

This combined National Engineering Conference and Conference of Operating Personnel will be held in Miami Beach, Florida, June 8-11, 1988 (Hilton Fontainebleau Hotel).

Participants will include structural engineers, fabricators, erectors, educators, and researchers. Potential authors *are requested to submit abstracts of papers on design, fabrication, and erection of steel structures for buildings and* bridges.

Guidelines for Abstract Proposals

* Abstracts for Papers to be considered for presentation at the Conference must be submitted to AISC **before August 31, 1987**.
* Abstracts should be approximately 250 words in length, and may be typed directly on the lower portion of the reverse side of this application, or submitted on a separate sheet of 8½ x 11" white paper attached to this submission form.
* Authors will be informed of the Organizing Committee's decisions by September 30, 1987. Successful authors must submit their final manuscripts for publication in the official 1988 Conference Proceedings by February 15, 1988.
* Registration fees for the Conference will be waived for the Primary Author presenting a paper at the Conference.

Poster Session

* Papers not accepted for presentation at the Conference may, at the author's expense, be presented at the Conference Poster Session. Guidelines for the Poster Session will be provided upon request after September 30, 1987.

Topics of Particular Interest

* Practical application of research results
* Advances in steel building and bridge design and construction
* Composite members and frames
* Buildings designed by LRFD
* Heavy framing connections
* Steel framed high rise residential buildings
* Partially restrained connections and frames
* Economical fabrication and erection practice
* Quality assurance and control
* Case studies of unique projects
* Computer aided design and detailing
* Material considerations

Preparation of Final Paper

Final manuscripts for publication in the official 1988 Conference Proceedings are expected to be approximately 20 pages in length, copy (including photographs) must be camera-ready. Complete instructions for preparation of final manuscripts will be forwarded to authors upon acceptance of Abstract Proposals.

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Call for Papers

Miami Beach, Florida
June 8-11, 1988

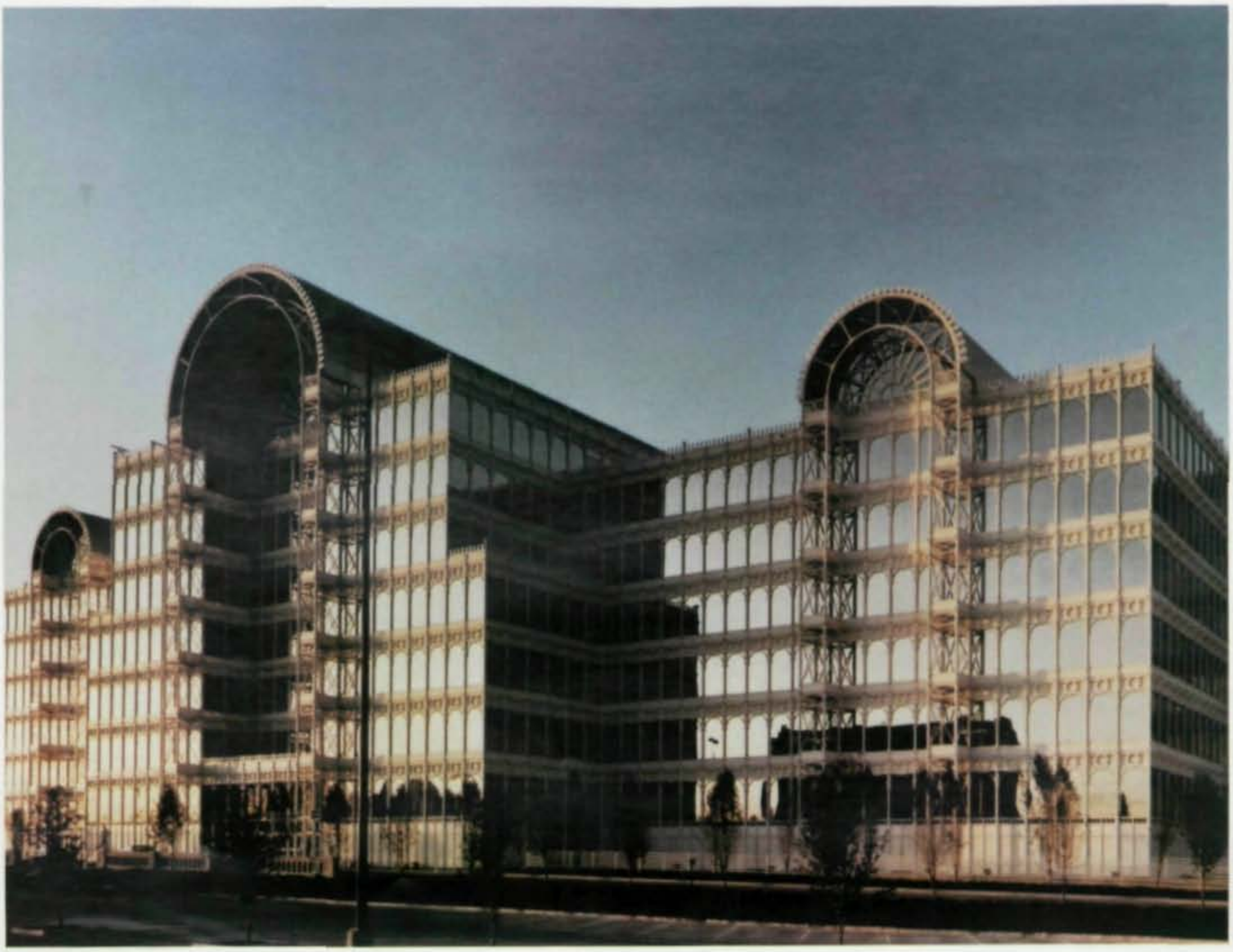
Architectural Showcase

INFOMART
A Modern Crystal Palace

The architectural design for Infomart, a market center which showcases the wares of the computer revolution, is based on that of the Crystal Palace, which was built in London for the Great Exposition of 1851 and showcased the Industrial Revolution. Infomart, 492 ft x 540 ft in plan, currently has seven floors (Fig. 1), but it has been designed for a vertical expansion to a maximum of 16 floors. The design includes a central atrium 60 ft wide, 252 ft long and 108 ft high which is crowned by a 60-ft dia., exposed steel cylindrical skylight extending the 540 ft length of the building. The atrium is highlighted by exposed steel elevator towers and escalator trusses at either end (Fig. 2).

The design provides 300,000 sq. ft of exhibition space on the two lower levels with a live load capacity of 250 psf; one million sq. ft of lease space with a capacity of 80 psf live load and 20 psf partition load; the capacity to expand upward to add another two million sq. ft of similar lease

Fig. 1. Modern-day Crystal Palace-Infomart market center, Dallas, Tex.



space in the future; and extensive and flexible electrical distribution capacities. The design and construction of the project was "fast-tracked."

A Value-engineered Center

The structural scheme was devised after considerable value-engineering was accomplished involving the developer, structural engineer and contractor. Typical bays, 36 ft x 36 ft, were chosen to provide sufficient open area for the exhibition space, flexible space layouts for the lease space, repetition of construction for ease of fabrication and speed of erection of structural steel and consistency with the 6-ft module of the exterior facade. The lowest two floors (the exhibition levels) are of concrete and all floors above them are steel construction. The lower exhibition level is a 7-in. slab-on-grade at 15 ft below the existing exterior grade. The upper exhibition level is a 15-in. thick, two-way reinforced concrete slab with 10 ft-4 in. dia. circular tapered column capitals (Fig. 3). Both exhibition levels have direct street access for arriving delegates and can accommodate truck traffic for unloading and erecting displays. The floor-to-floor heights for these two levels are 16 ft and 24 ft. These two concrete floors were constructed while the steel structure above was being designed, bid, mill-ordered and fabricated. This removed from the construction schedule any waiting time for the mill-order and fabrication of the structural steel.

The steel structure built on top of the exhibition levels (Fig. 4) supports the Infomart lease space. This steel scheme cost less, had significantly lower foundation costs and a shorter construction schedule than the concrete schemes evaluated during the value engineering phase. It also provided the ease of vertical expansion desired. The 36-ft x 36-ft bays are framed by composite steel beams at 12 ft o.c. spanning to composite steel girders which in turn span between columns. Each column

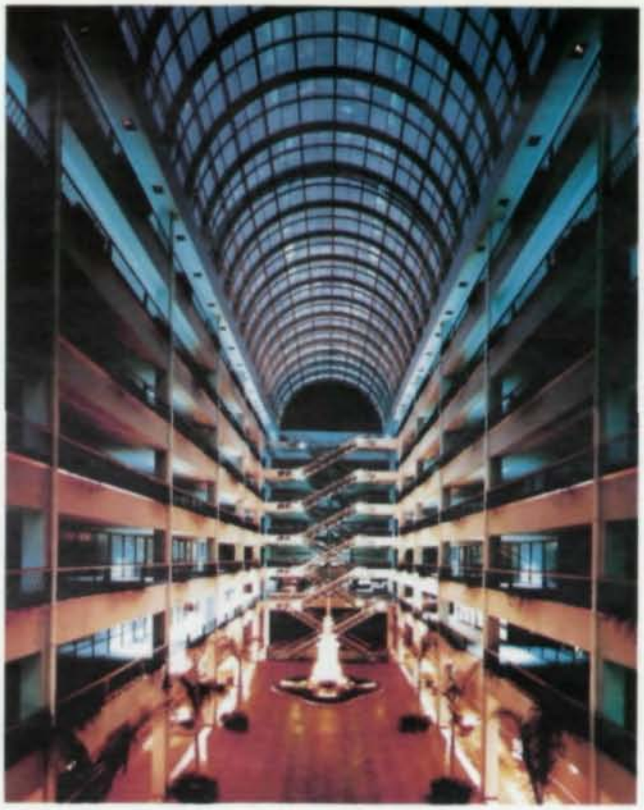


Fig. 2. Spectacular interior highlighted by exposed steel elevator towers and escalator trusses.

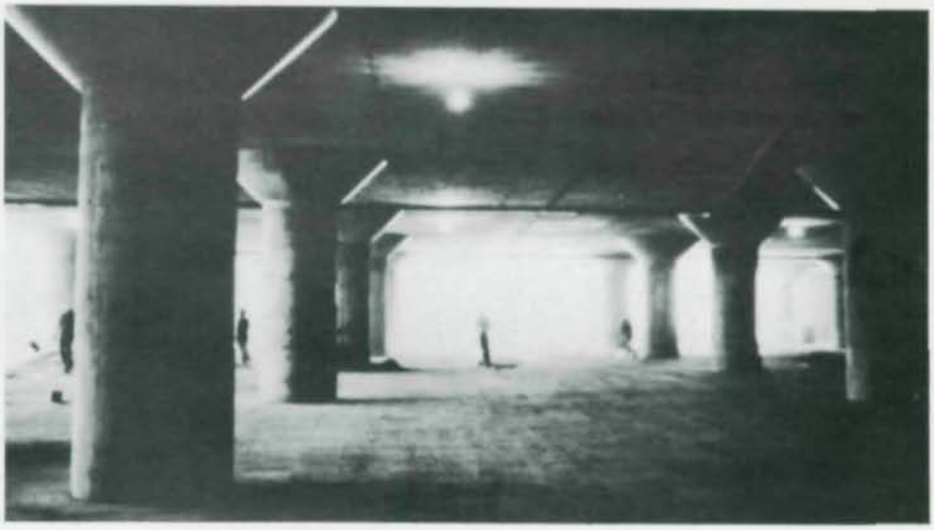


Fig. 3. Upper exhibition level showing tapered column capitals

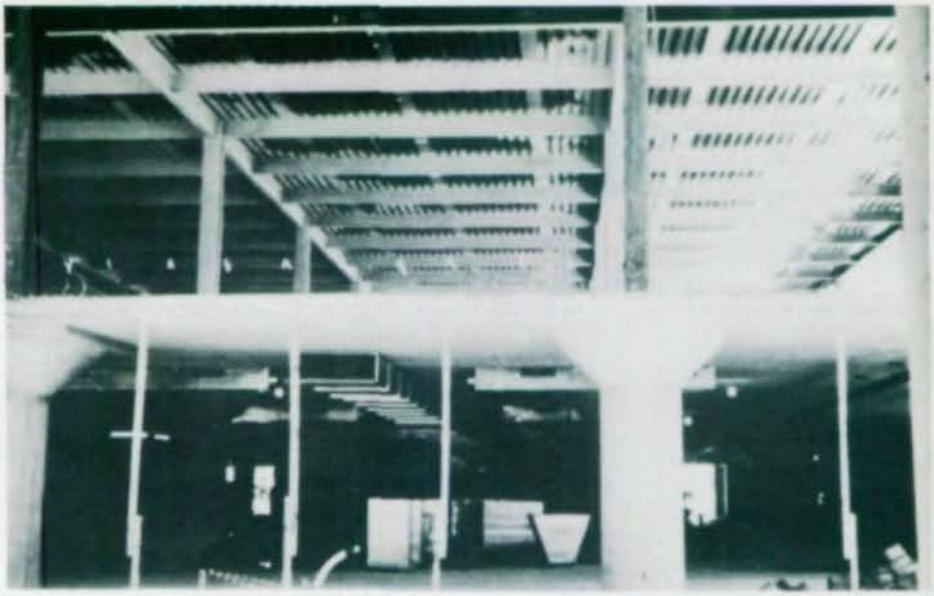
is supported by a single pier drilled through 60 ft of weak overburden down into shale. The weight saved by using steel construction instead of concrete reduces the column loads and resultant required pier capacities by 25%. The slab is 3¼-in. lightweight concrete over a 3-in. composite metal deck. The metal deck incorporates a blended cellular electrical raceway system for flexible electrical distribution (an important design consideration for the facility). The beams, girders and columns are typically 50 ksi steel. This reduces the steel tonnage by 30% while increasing the material cost of the steel by only 15%.

Several Stability Plans

Several schemes for providing lateral stability were studied. The most interesting, in keeping with the Crystal Palace's architectural theme, incorporated exposed steel bracing in the full-height atrium. This scheme was abandoned primarily because of fireproofing requirements. Ironically, London's Crystal Palace was destroyed by fire in 1936. The lateral system used is steel bracing at the four exterior corners of the building (Fig. 5). These wind bracings (as well as all building columns) are designed to be extended vertically to support the future expansion. The bracings eliminate the need for any welded frames and thus accelerated erection time and increased the number of typical bays. The total erection time for the structural steel, which framed 1.3 million sq. ft, was only 13 weeks.

Construction time was also compressed by borrowing from high-rise construction practices the idea of staggering the work of various trades. The staggering for this 6-story, above grade project (which has as many sq. ft as some 50-story projects) was accomplished horizontally. The building was completed in four quadrants, with construction trades following in sequence from one quadrant to the next. The Infomart opened 16 months after construction began, and only 12 months after the foundation piers were completed.

Some elements of the project which saved construction time required special design considerations. Building the lower two levels of concrete while the steel for the upper five levels was being mill-ordered and fabricated required the transmission of loads from steel columns to concrete columns. A 12-in. deep depression was cast in the top of each concrete column capital to conceal a steel column base plate. The steel column base plate was set and grouted into the depression and then it was filled with concrete flush with the rest



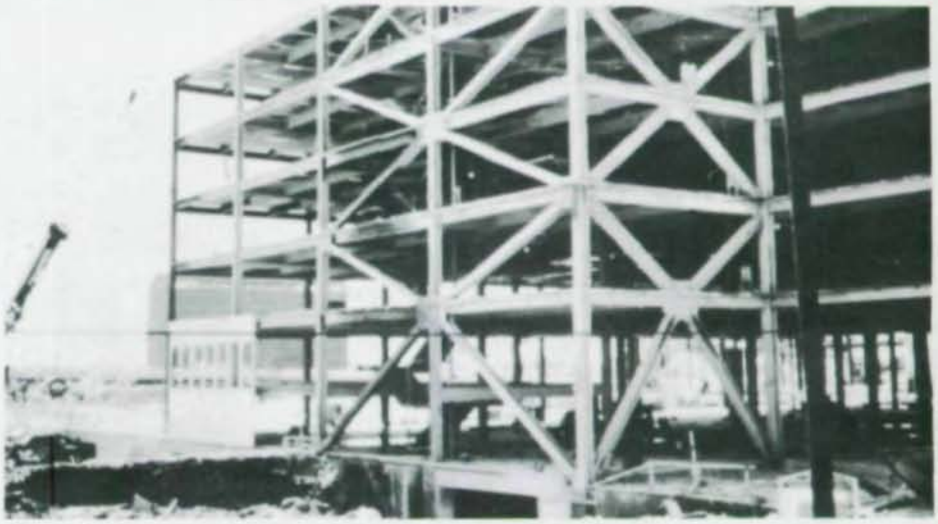


Fig. 4. Steel-framed structure (top) above exhibition levels supports leased space. Fig. 5. (bott.). Lateral system of steel bracing shown.

Figure 6

Figure 7

of the floor (Figs. 6 & 7). Additional reinforcing steel was placed in the concrete column capital to accommodate the depression in the capital and the bursting stresses due to the steel column load.

The huge floor plan which permitted horizontal staggering of construction was built without expansion joints (which can be unsightly and require maintenance). The shrinkage stresses in the 15-in. concrete first-floor slab and in the 6¼-in. slabs on metal deck above were virtually eliminated by using pour strips. A pour strip is a narrow strip of slab separating two large sections which is not cast until most of the shrinkage of the larger sections has occurred. The stresses in the structure due to the restraint of the thermal expansion and contraction of the large floor plates by the bracing at the four corners were computer-analyzed. The roof slab, which was insulated on top, was checked for the change in average slab temperature from the coldest to the hottest day. The floor slabs were checked for a nominal temperature differential of one slab to another. Since these in-plane slab stresses and the combined bracing forces were within allowable limits, expansion joints were eliminated from the floors.

Structural analysis techniques and construction planning methods have been revolutionized by the first generation of engineers and contractors to have computers. The creation of Infomart, using these tools, has provided a center for the propagation of computers to future generations who will ensure that steel construction continues to be "modern."

Architect
Growald Architects

Structural Engineer
CBM Engineers, Inc.

Mechanical Engineer
Blum Consulting Engineers

Contractor
Orion Construction

Fabricator
Hirschfeld Steel Company

Erector
Regional Contracting Corporation

Developer
Trammell Crow Design and Construction

Dr. Joseph Colaco is president of CBM Engineers, Houston, Texas.

Wally Ford is a senior associate of CBM Engineers.

GARDEN STATE PARK PAVILION

Walking Ring for a Running Track

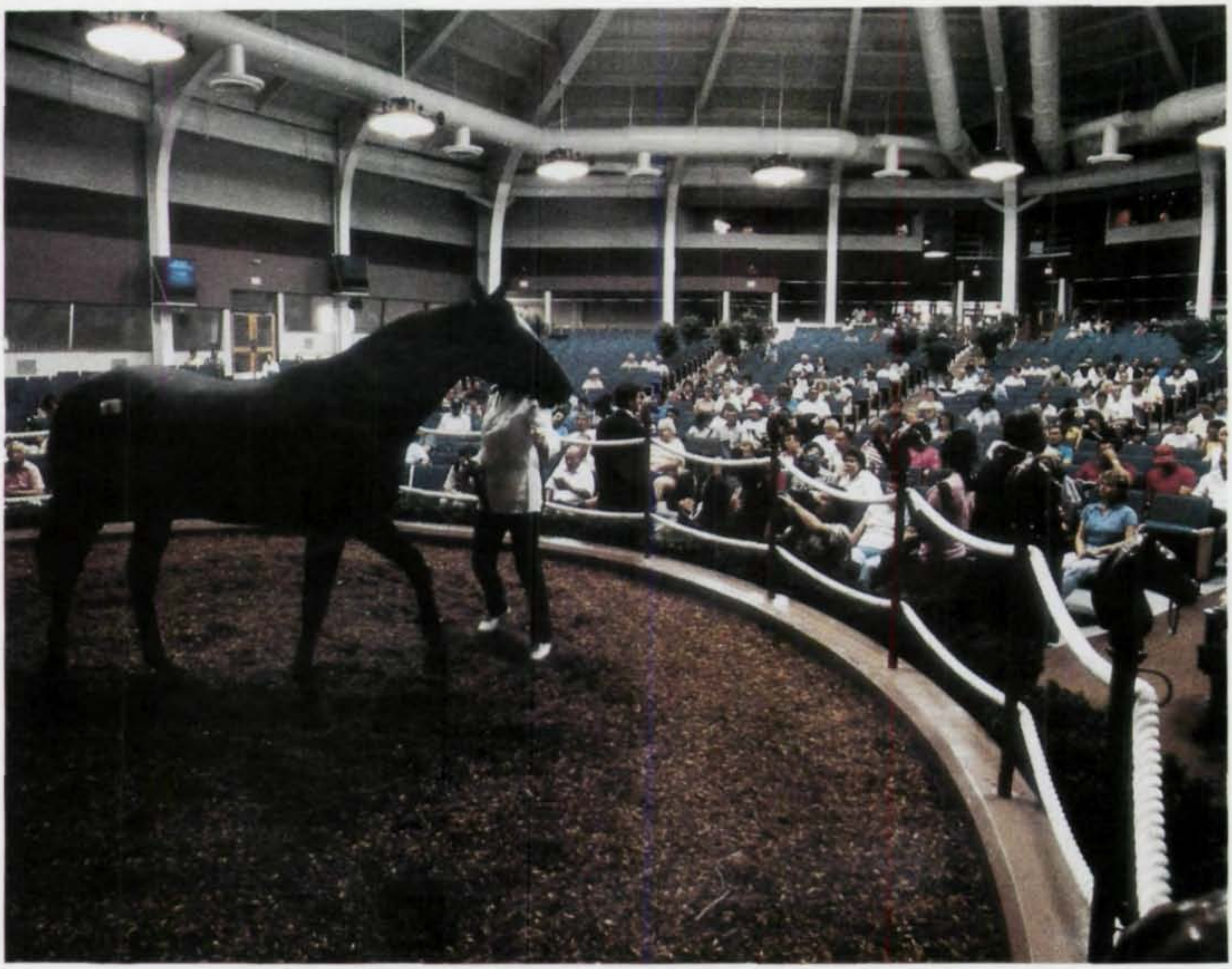
by Pradeep R. Patel

A hexagonal-shaped auditorium, the Garden State Park Sales Pavilion is built adjacent to the Crystal Paddock at Garden State Park, Cherry Hill, N.J. This racetrack, destroyed by fire in 1977, was rebuilt and expanded by a Philadelphia-based architectural/engineering/interior design firm, for a grand re-opening in 1985.

Phase 2 expansion included the 53,000-sq. ft Sales Pavilion, completed in the spring of 1986. This clear-span auditorium, 150 ft in diameter and about 70 ft high, has a seating capacity of 1,500. Its two large television screens allow patrons to watch live Garden State races, simulcast races from other tracks and other sporting events. The auditorium also functions several times a year as a sales facility for both thoroughbred and standardbred horses, which are brought on stage from "backstage" stalls and a walking ring.

The structural system for this building is a three-way, rigid frame which spans 150 ft. The three frames at the crest are connected to a hexagonal compression ring box, consisting of top and bottom plates, as well as side plates and stiffeners. The design concept was to allow stresses to flow from each half of the rigid frame through the central compression ring box to the other half of the rigid frame. The depth of the rigid frame at the knee is 9 ft, and 5 ft at the compression ring. The horizontal forces at the base of the rigid frame are resisted by tie rods 3 in. in diameter.

Exterior of new Garden State Sales Pavilion, Cherry Hill, N.J.



Clear-span (150-ft) auditorium seating 1,500 permits indoor race viewing and serves as walking ring and sales facility for thoroughbreds. Below (l.) shows interior seating arrangements. Below (r.) details steel framing of large pavilion.

The compression ring elements were given careful consideration in detailing the welded joints involving thicker plate material. It was essential to maintain continuity of frames within the compression ring. However, this meant the stresses at the top and bottom flanges had to be transferred by welding each rigid frame to the compression ring. Knowing that welding must be avoided in tensile stress zones involving thicker plate material, our goal was to solve lamellar tearing-prone details on paper.

After exploring several ideas suggested by the fabricator and the architect/engineer, the engineer developed a practical approach using bolted connections at tensile zones and welded connections in compression zones. This is a good example of the shop drawing design/detail process, where complex and critical detailing problems can be solved with teamwork between fabricator and design professionals working in the best interest of a project.

The foundation for the structure was constructed by placing pile caps on cast-in-place concrete piles. The erection procedure was kept in mind while designing the foundations, since an erection tower would be used at the center of the compression ring. A special pile cap on the pile foundation was designed to hold the erection tower.

This tower was critical for controlling alignment and erection of all components of the structure. The frames were also designed and detailed to accommodate the splice at the knee area of the rigid frames. This splice would be made at the top of the column section, rather than at the horizontal beam section.

A three-dimensional computer study

helped analyze the behavior of this unusual structure with hybrid design elements (i.e., three-way rigid frames, hexagonal compression ring, and spandrel framing elements at hexagonal perimeter acting as partial tension ring). This analysis included several symmetrical as well as asymmetrical loading conditions. The connections were then designed for critical load conditions.

The data for theoretical frame deformations were also given to the structural steel contractor, who, in turn, made practical adjustments for erection of the structural elements in alignment. These adjustments took into account the desired final alignment with complete dead load on the structural frame upon removal of the erection tower.

Strain gauges were used to monitor simultaneously the tension at the tie rods connecting the three rigid frames. A uniform minimum tension of 20 kips was recorded in all tie rods by tightening the turnbuckles and accurately monitoring the sequence prior to removal of the tower. The tension was applied in 5-kip increments. The tower was removed gradually, after all the dead load was in place. All secondary framing connections for the structures outside the main auditorium framing were bolted later, to avoid stressing those connections through rigid frame deflection upon release of the tower.

The design and construction of this structure proved that a complex design can be built with success if cooperation exists among the general contractor, the fabricator and the design professionals. □

Architects/Structural Engineer
Ewing Cole Krause
The sports facilities division of
Ewing Cole Cherry Parsky
Philadelphia, Pennsylvania

Construction Managers
Torcon, Inc.
Westfield, New Jersey
Costanza Contracting Co.
Pennsauken, New Jersey

Steel Fabricator
Lehigh Structural Steel Company
Allentown, Pennsylvania

Owner
International Thoroughbred Breeders, Inc.
East Windsor, New Jersey

Pradeep R. Patel is a principal and director of structural engineering at Ewing Cole Cherry Parsky, architects, engineers and interior designers, with offices in Philadelphia, Pennsylvania and Haddonfield, New Jersey.

"ONLY VULCRAFT COULD HANDLE A JOB THIS BIG THIS FAST."

The Levi Strauss & Co. Jeanswear Distribution Center is big: 720,000 square feet at a cost of $19 million and capable of shipping 50 million units a year.

What's more, it had to go up fast. In time for Levi Strauss & Co. to use it to fill summer orders. So a job that typically would be scheduled for 15 months had to be completed in 11 months or less from the time design work started.

The Haskell Company, which designed and constructed the building at Gluckstadt, Mississippi, relished the challenge. And they knew who to turn to for steel joists and joist girders. In their words, "Only Vulcraft could handle a job this big this fast."

Because Haskell used fast track methods, they specified a precisely timed sequence of deliveries. Vulcraft followed through to perfection. Which didn't surprise Haskell. They had worked with Vulcraft for 15 years, including 20 projects in the previous 30 months.

For more information concerning Vulcraft steel joists and joist girders; or, for copies of our joist and girder catalog, contact the nearest Vulcraft plant listed below. Or see Sweet's 05100/VUL.

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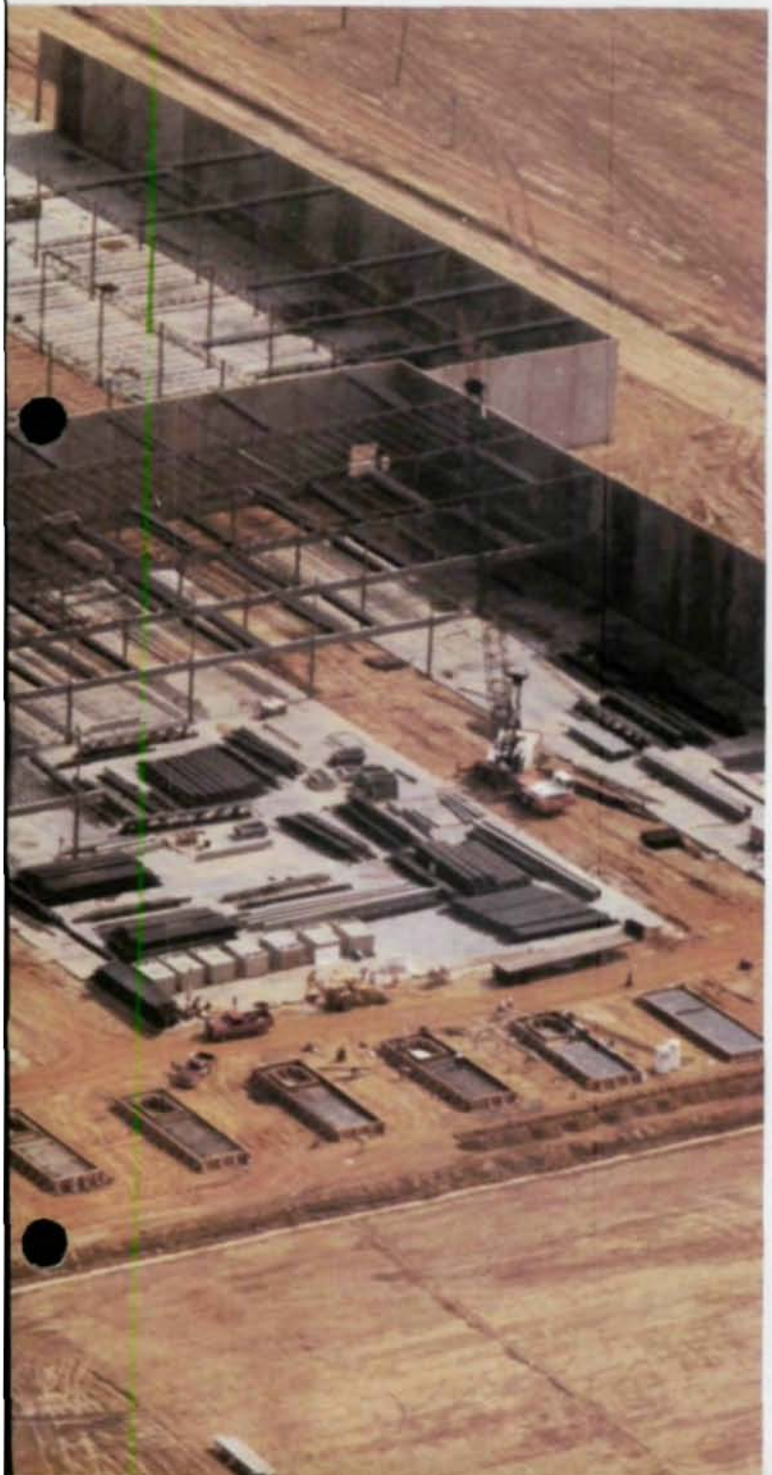

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General Contractor: The Haskell Company
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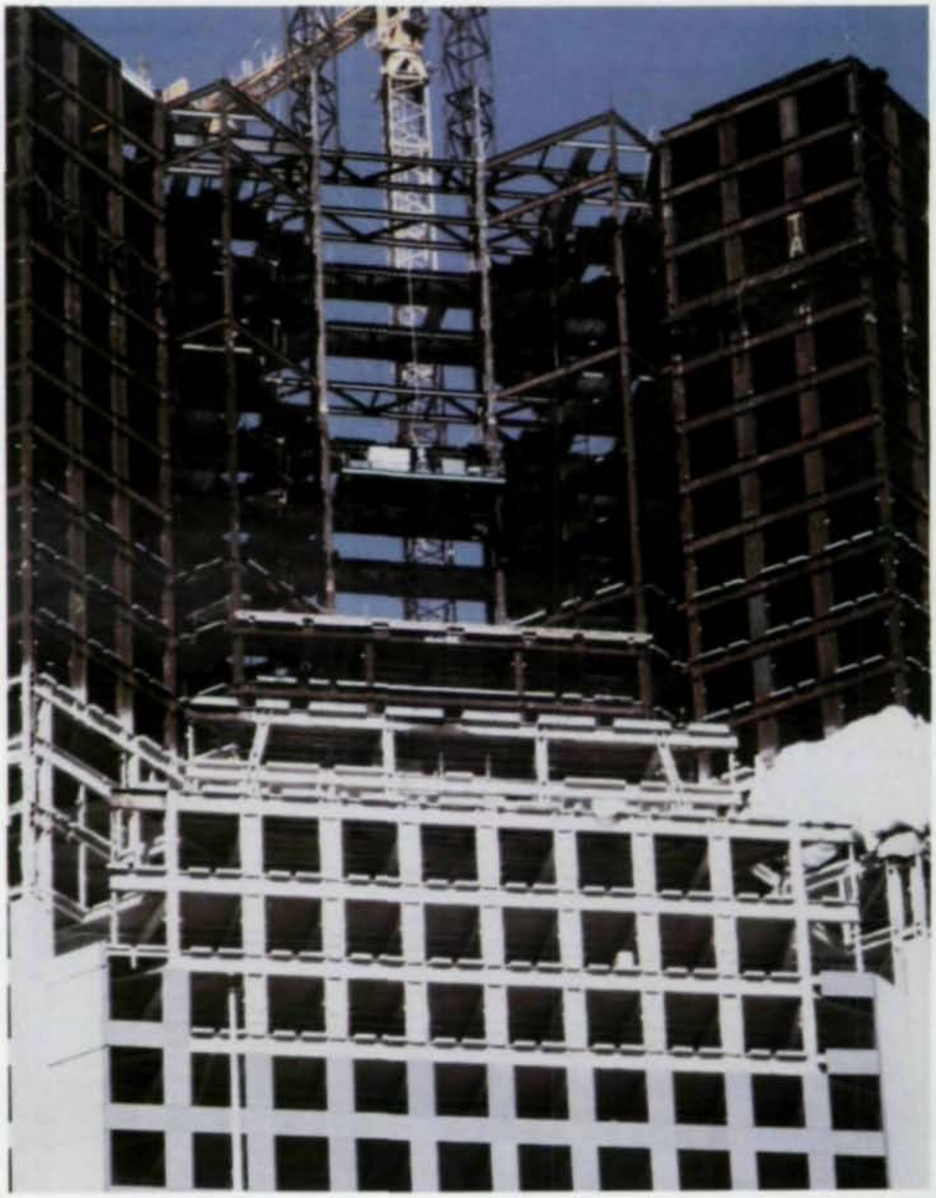
Vulcraft supplied 1,100 tons of joists and 207 tons of joist girders for this Levi Strauss & Co. Distribution Center at Gluckstadt, Mississippi.

CALIFORNIA CENTER
Big, Bold, Ambitious— and Last of Its Kind

by David Moulton and Navin Amin

When the California Center project began, John Harris, from the developer's firm, went to the mayor and asked, "What would you like to see here?" She responded she would like to see a mixed-use building, a combination of residential or hotel use, retail and office space. When the architect got involved soon after that meeting, they were instructed to develop this project on the basis of mixed use. It turned out that the Inner Planning Code at that time permitted office space to be built at a certain floor/area ratio. The code also permitted a bonus system, which would permit development of more area in a building as compensation for doing certain things, such as creating plazas, setbacks, etc. But the increased area could be devoted to residential uses, either hotels or residences.

The problem was to develop as much floor area on the site as possible, devoting that portion generated by bonuses to condominiums. As the project progressed through the planning stage, the condominiums became hotel rooms. The final result is a building with 47 stories divided up into 35 floors of office, plus the lobby, 11 hotel floors and one mechanical floor. In addition, there are two basement parking levels. Total square footage in the building is 885,000 sq. ft. The center, located in the heart of downtown San Francisco, touches four streets—but fronts on none of them. The building is a fairly long, thin hexagonal shape in the center of the site.



A Site—an Opportunity

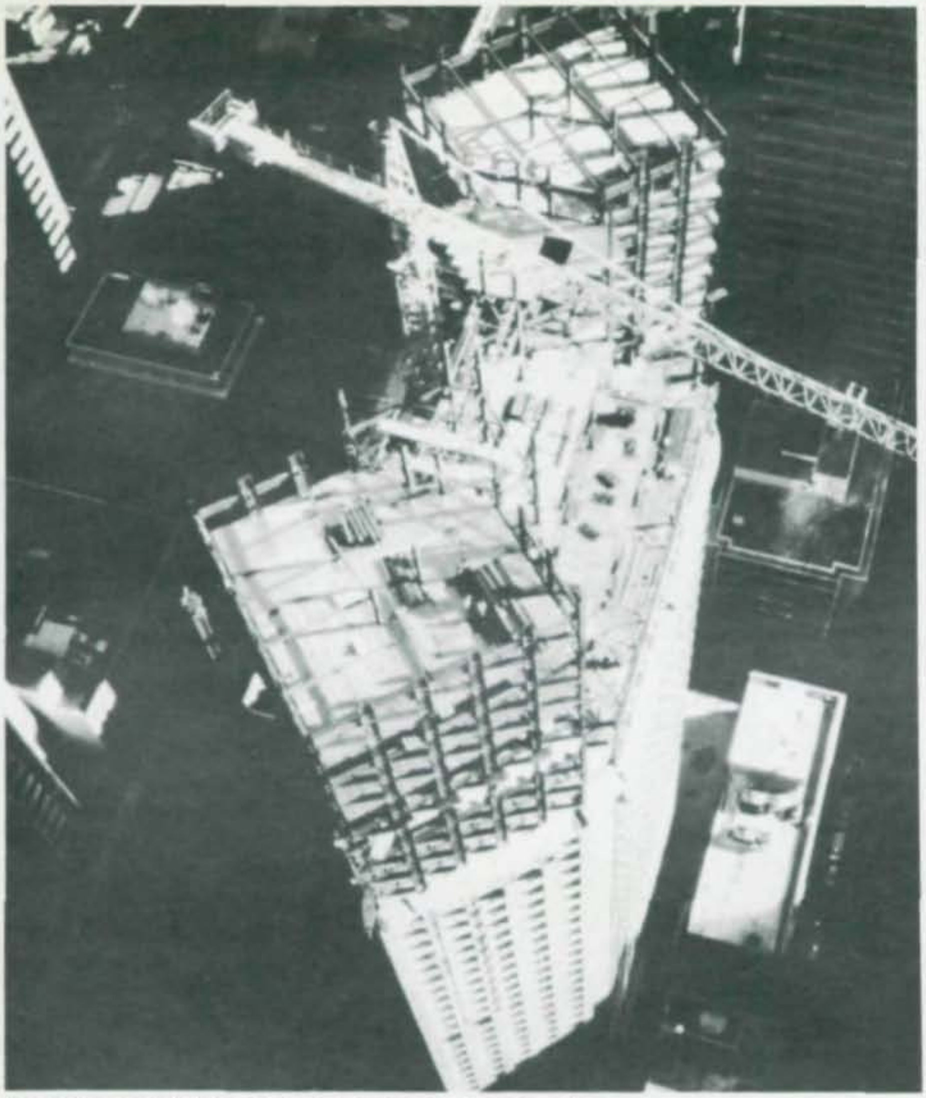
This, along with the mixed use of the building function, was the greatest challenge

Steel-framed California Center was challenge in both design and site conditions.

and opportunity of the site. It makes the building unique. Many design studies were done to discover the potential of the site and to tie the various functions of the building together under one roof. Some of these design studies were so unusual the engineers started to get nervous. But they stayed on that track, and found forms which chamfer the end of the building worked very well. They opened up areas at the bottom of the tower where it is surrounded by buildings on all four corners. The engineer found that splitting the top of the building into two segments or towers expressed the fact the residential areas reduced the bulk impact of the structure. A lot of detailed study went into choosing the final configuration. In doing the studies, a lot of little white models, boxes and boxes of them, were built.

At this point in the process, two things happened. First, the developer of the building sold the project, essentially to their own management people. A new group was formed with outside investors. Thus, many of the same people continued to be involved in the planning. The other thing that happened, the new client—in evaluating the scheme and looking at the lease spaces and the efficiency of the building—determined the split towers and the very small retail or rental floors (only about 15,000 sq. ft) worked against the project's economics. Two instructions to the engineer resulted. One was to find a way to make the split upper floors in the building work as one floor, with one elevator system and one stair system and to find a way to increase the square footage on the main tower floors. So back to the drawing boards and the models again. The main problem was the bulk limits of the site had been reached, as had also the city's maximum 200-ft diagonal dimension measuring from tip to tip. The architect looked at a number of schemes which would not go beyond the limits. The final determination was the best approach which would be to simply extend the building, or spread it out laterally. Then, the two towers were quite far apart. But by pulling the cores in toward each other and connecting them by glazed bridges, the architect could maintain the basic theme ideas of the building.

It had already been decided the structure was to be clad with monolithic green, partially reflective glass to stress as strongly as possible the form of the building. But the city decided they did not want any more reflective glass in San Francisco. So the final decision was to clad the building in polished stone and clear glass. Also, they proceeded to develop the building with a three bank elevator system plus a



Spectacular aerial shows tight siting and geometry of structure.

separate bank for the residential part. The building plan is a chamfered, hexagonal-shaped building, with floor plans that work quite well and permit spectacular views from the upper floors. The building is very thin in one direction—actually only 100 ft wide—which was dictated by the code requiring exactly 20 ft from the interior property lines.

Architectural Interest with Steel

The part of the building which was of terrific interest to the design group was the base. It must be more than just an office building to be used by the people leasing space. It must have something for the city. The main entrance is through an opening from California Street into the lobby. A system was developed to direct the public sidewalk access straight through the block as well as to connect to California Street. The main lobby accommodates retail spaces as well as the hotel lobby. The architect would have to develop a podium for the building. An enormous amount of study went into it to determine the character of it. And there was much negotiating with the city about color, finishes and the actual design. Out of the meetings came the scheme of a building entirely clad in stone, except for the structural steel decoration. Part of the architectural treatment was to use exposed painted structural steel in the shape of wide-flange beams above the four entrances to the building. These beams have no structural function, only architectural, but they are quite impressive.

In addition to using structural steel over the entrances, the design team used it in a decorative fashion in the lobby—such as for columns to support light fixtures and as supports for the building directory.

Real Challenge in Structure

The real challenge was to come up with a structural system to incorporate the requirements of the hotel, office, retail and parking, and one that would be economical in terms of the least structural steel weight to resist seismic and wind loads for this narrow, slender tower configuration. One of the important criteria of the design also included the aspects of human perception due to wind induced accelerations.

Several structural ideas and concepts were evaluated. The selected concept was multiple tube frames with an eccentric bracing at the building core. This was the most effective system to resist the seismic and wind loads with the least amount of steel weight. The analysis and design of the main frame was done using three-dimensional modelling techniques. The complete frame analysis and design was done with in-house computers. The frame was analyzed and designed to resist gravity loads in combination with code seismic loads and wind loads. It was also analyzed and designed to resist forces due to maximum probable and maximum credible seismic response spectra. The response spectrum analysis was done with dynamic analysis techniques.

The floor system is a composite metal deck slab supported compositely on standard rolled beams. The bridges at the hotel levels are designed so that in a seismic event there is no relative movement between each of the hotel towers. The bridge floor structure is a horizontal truss in plan. The slenderness of the tower and the hotel type occupancy led to a complete wind study for the project. The wind tunnel studies were performed to predict the wind pressure and suction for the design of the external wall and the glass, as well as the structural loads for the frame- and wind-induced accelerations.

One very important consideration was the behavior of the moment connected 36-in. wide column and 36-in. deep beam under a seismic event. A detailed test program was undertaken at U.C. Berkeley under Prof. E. Popov to study the behavior of a large column-beam joint under cyclic loading. The results of the tests confirmed the connection details used on the project would perform satisfactorily under a seismic load.

California Center—big, bold, ambitious—and the last of its kind in San Francisco. Never again will the Downtown Plan permit another skyscraper so tall and bold to rise in so crowded and historic a cityscape.

Architect/Engineer

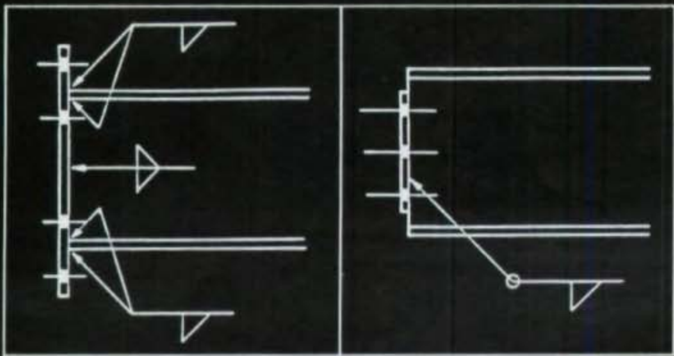
Skidmore, Owings and Merrill
San Francisco, California

General Contractor

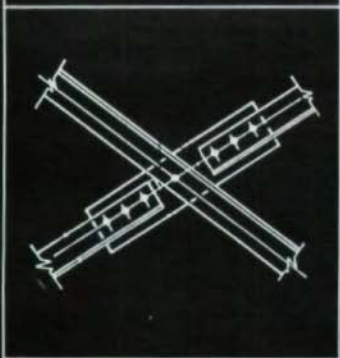
Dinwiddie Construction Co.
San Francisco, California

David Moulton is an architect with Skidmore, Owings and Merrill, San Francisco, California.
Navin Amin is an associate partner with Skidmore, Owings and Merrill.

Structural Software Co.



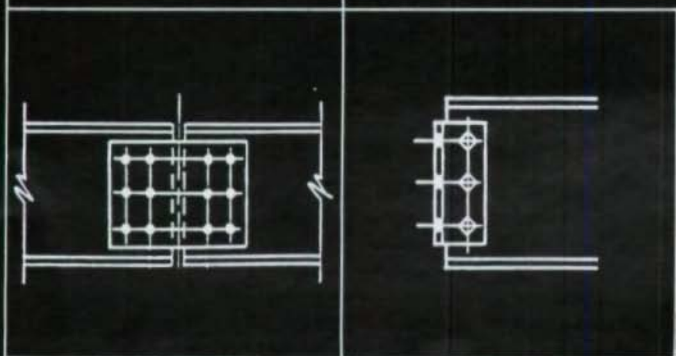
Structural Software's Computerized Detailing



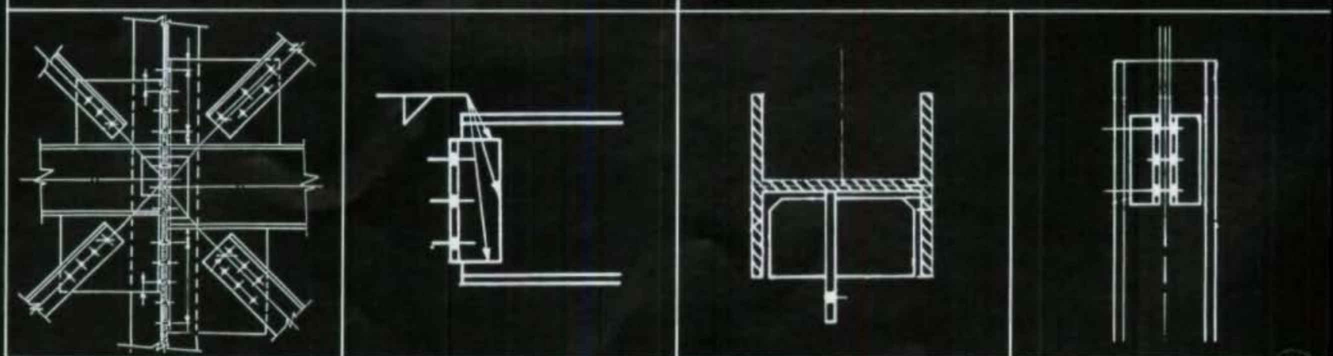
Computerized Detailing is a relatively inexpensive structural steel detailing program designed to run on the IBM-AT compatible computer systems. The system not only completes details, but also designs connections according to the A.I.S.C. specifications.

Computerized Detailing allows a multiple member input of material using a grid plan thereby allowing the operator to easily make global assignments of connections. **Computerized Detailing** can then produce an erection plan as well as detail the project as a total integrated unit.

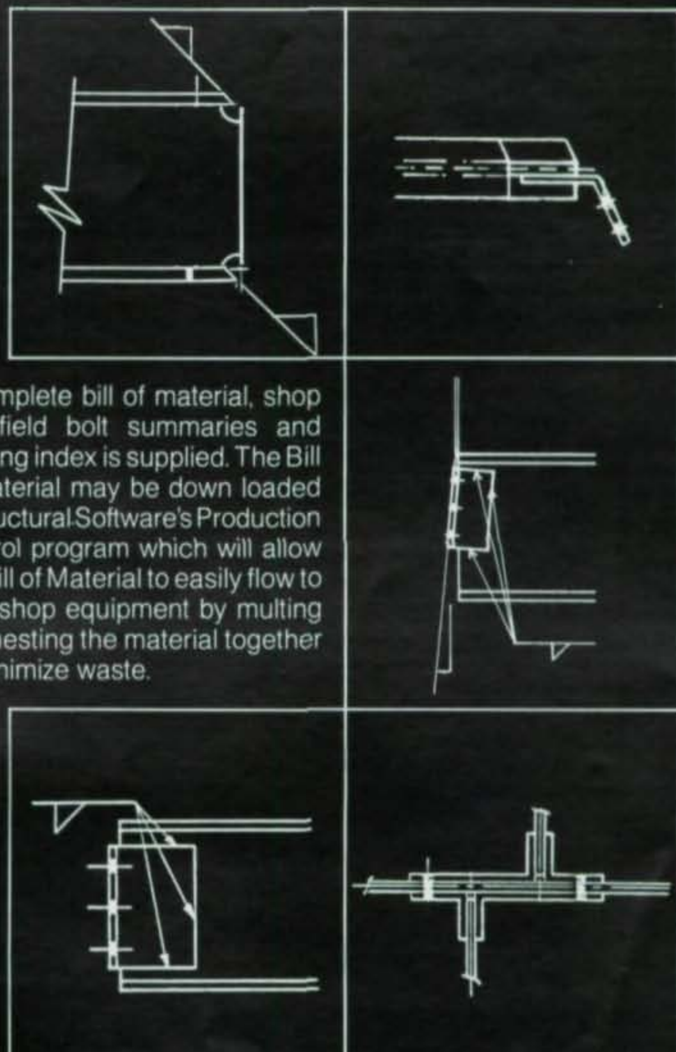
Computerized Detailing will handle wide flanges, channel, tube, pipe, angles, and plates. The design portion of the program investigates a wide variety of shop and/or field connections, that in-



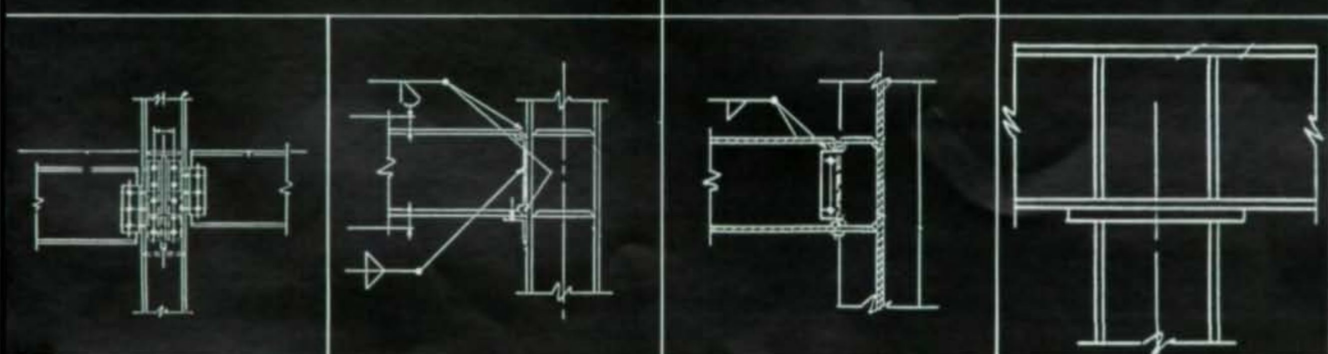
Structural Software Co.
2839 Peters Creek Rd.
Roanoke, VA 24019
(703) 362-9118



clude combinations of bolted and/or welded, framed, cantilevers, knifed, end plate, stiffener and shear plates, moment connections, splice plates, tee connections, joist, joist girder, and one-sided connections. The design routines will also accommodate non-flush top beam framing, sloping beams, bracing, skewed beams, off column centerline framing (within limitations) and offset as well as opposite beam framing. An interface to Autocad graphics is available for additional special input.



A complete bill of material, shop and field bolt summaries and drawing index is supplied. The Bill of Material may be down loaded to Structural Software's Production Control program which will allow the Bill of Material to easily flow to your shop equipment by multing and nesting the material together to minimize waste.



Steel Notes

OOPS, SORRY DEPT.

In the last issue of *Modern Steel Construction* (No. 2-1987) in Clellon Loveall's feature, "Advances in Bridge Design and Construction," we incorrectly pictured the Whitechuck River Bridge. Here's the correct photo.

AISC-KLINGELHOFER SCHOLARSHIP ANNOUNCED

The first AISC/Klingelhofer Scholarship Award will be available to undergraduates in civil or architectural engineering in the Pacific Northwest states of Washington, Oregon, Idaho and Alaska.

This award, in the amount of $5,000 for the school year, commencing in the fall 1987, brings to three the number of annual scholarships being offered by the AISC Education Foundation. For 1987, the Stupp Brothers Scholarship will be available in Texas and New Mexico, while the USS Steel Award is offered in Upstate New York schools.

1987 NEC/COP PROCEEDINGS AVAILABLE

The *Proceedings* for the 1987 National Engineering Conference/Conference of Operating Personnel (NEC/COP) is now available for purchase. the 820-pg. book contains 47 complete technical papers presented by recognized authorities at the 1987 conference held in New Orleans April 29-May 2. Papers cover a broad range of the latest developments in building and bridge design, fabrication and construction. Illustrations, tables and photographs are also included.

The cost for the *Proceedings* (G448) is $26.25 for members, $35 for non-members. To order, send a check, money order or Visa/MasterCard information (state type of card, number and expiration date) to AISC Publications Dept., P.O. Box 4588, Chicago, IL 60680-4588.

LIMITED NUMBER OF 6TH EDITION MANUALS AVAILABLE

A number of copies of the *Manual of Steel Construction*, 6th Ed. (1967) are now available for $10 each (no discount). This 768-page manual is based on the provisions of the April 1963 AISC Specification. To order the Manual (M009), send check, money order, Visa/MasterCard information (state type of card, number and expiration date) to AISC Publications Dept., P.O. Box 4588, Chicago, IL 60680-4588.

1987 FELLOWSHIP AWARD WINNERS NAMED

Eight winners of AISC's 1987 Fellowship Award competition have recently been named. Each winner receives a $4,250 study fellowship, with another $750 going to the academic department heads for admiistering the awards. Students ae judged by an outstanding award jury on the basis of grade point averages, faculty recommendations and contributions their expected programs will make to the engineering profession and the structural steel industry as a whole.

The 1987 winners are:

Douglas J. Ammerman, University of Minnesota
Gus Bergsma, University of California-Berkeley
Christopher M. Foley, Marquette University
Galal A. Korayem, Georgia Institute of Technology
Brian D. Peck, Syracuse University
Carl D. Petty, University of Maryland-College Park
Gregory P. Schleusner, Marquette University
Gregory L. Tucker, University of Tennessee

**1988 NATIONAL ENGINEERING CONFERENCE/
CONFERENCE OF OPERATING PERSONNEL**

Hilton Fontainebleau

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June 8-11, 1988

WASHINGTON-BATTERY BUILDING

Steel Solves a Tight Site Problem

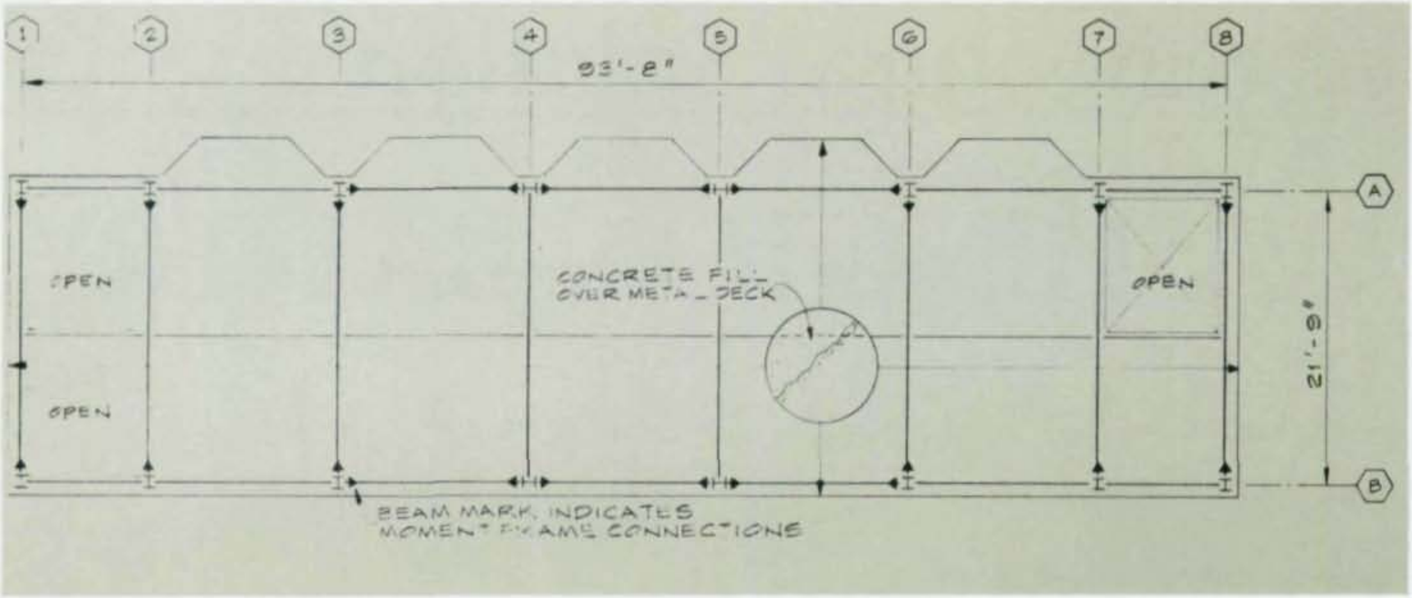
by John D. Meyer and Deborah Michels

The southwest corner of Washington and Battery Streets in San Francisco's Financial District was once a small embayment of San Francisco Bay called Yerba Buena Cove. Today, this narrow, problem-ridden lot is the home of the Washington-Battery Office Building, a tribute to the merits of modern steel construction and a winner of an CCAIA Honor Award.

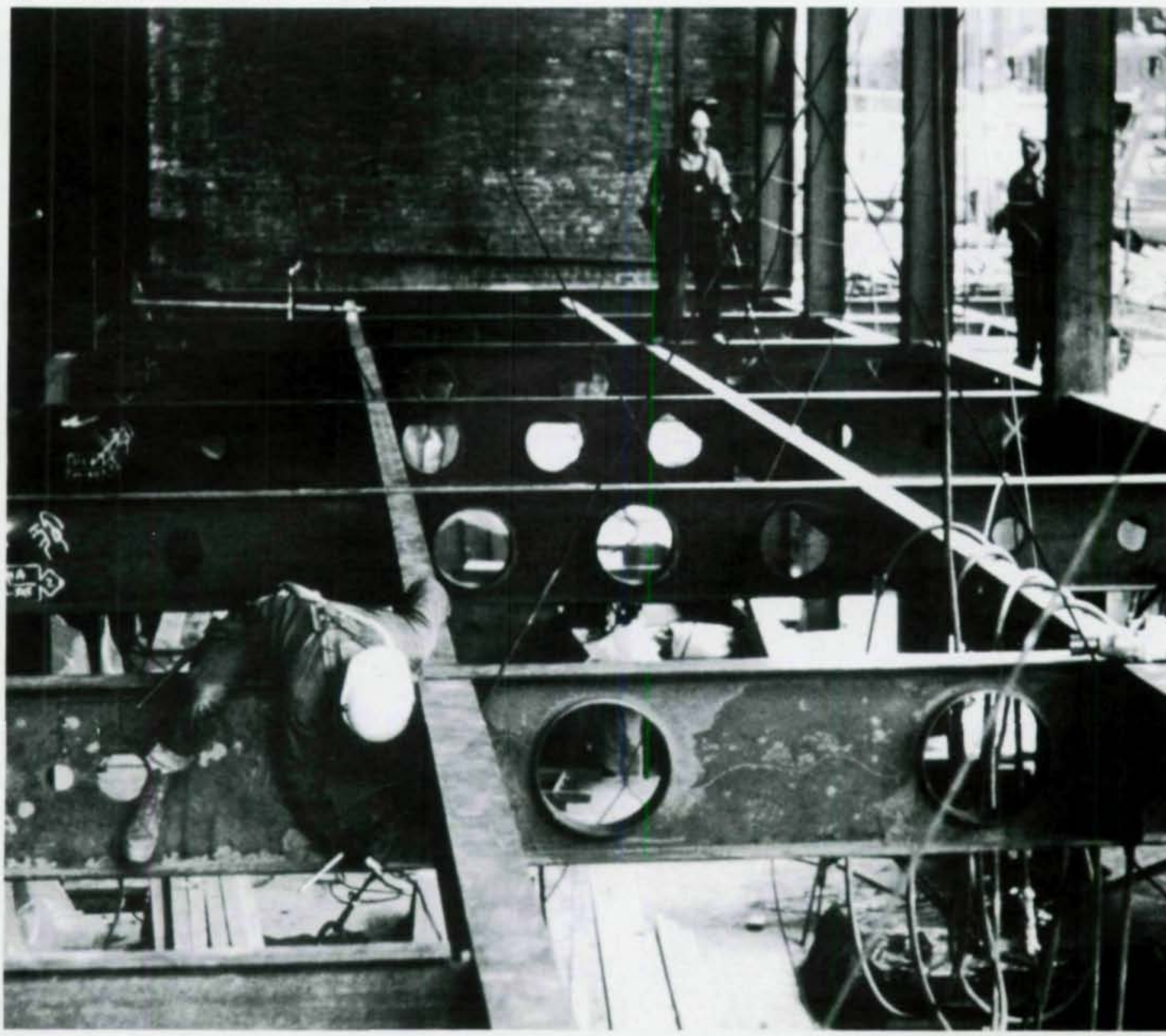
Previous attempts to develop an office building on the site succumbed to the harsh rules of economics. The location was prime, but the 25 × 97-ft lot size had always proven uneconomical. The site shares common property lines on two sides with historic 3-story brick buildings with full basements. One of these, the Burroughs Building, houses highly sensitive equipment. Existing concrete perimeter basement walls, buried slabs and foundations and wood piles from a previous structure would have to be removed with extreme care because of the zero property line. The architecture of any proposed building would have to be in keeping with its prestigious neighbors—most notably, the TransAmerica Pyramid, Embarcadero Center and One Maritime Plaza (the Alcoa Building). Directly across the street is the U.S. Customs Building with its rusticated granite facade. This then, was the architectural challenge to be met: to conceive on this small, but highly visible, site a major office building that would complement but not distract from the impressive neighbors.

Architectural Solution

The architects designed a 7-story, 22,000 sf office building with full basement. After consulting the San Francisco Planning Code for requirements of bay windows,



Typical floor framing plan (above). Photo shows framing with large beam penetrations for ducting.



cantilevered industrial, sash-bay windows were used to add space. The corner location required fire walls at property lines and fire-rated towers. For the stair towers on each end, precast concrete panels were massed and rusticated to relate to the U.S. Customs Building reflected in the Washington-Battery's glass front. The floor plan was designed to accommodate a prestigious single user.

Structural Solution

The structural engineer was challenged by the many difficulties of the site. The subsurface consists of Bay mud over a sand lens 50 ft thick, 25 ft below grade, with older clay-like Bay mud beneath and then weathered bedrock. The most economical and practical foundation system would be piles driven into the sand lens, but the subsurface conditions created problems here too:

- Using the sand lens for pile support resulted in limited capacity.
- Precise location of piles was difficult to achieve. Existing wood piles were encountered which could not be removed.
- It was necessary to drive 2½ ft inside of the property to avoid disturbing the Burroughs Building footings.

All of these factors made placement complicated and expensive. The structure would need to resist San Francisco's severe seismic forces. This fact, in addition to the limited pile capacity, indicated the need for the lightest, most flexible structural solution. With this in mind, the engineer used a structural steel moment frame with a concrete grade beam system over the 50-ton capacity, 12-in. × 12-in. prestressed concrete piles. At the property line, the grade beams were cantilevered over the piles to pick up the steel columns. The advantages of using the steel moment frame as opposed to shear walls and/or braces are many. The steel moment frame offers proven reliable resistance to seismic forces without the architectural constraints imposed by shear walls and braces. Moment frames have lower overturning forces than shear walls or braces. This, in turn, was in keeping with the need to minimize the load imposed on the piles. Also, the moment frames allowed a symmetric lateral force resisting system. Symmetric systems avoid torsion problems and are therefore very desirable for earthquake resistance. The design of the ductile, moment-resisting space frame was controlled principally by drift limitations. Limiting building drift during moderate earthquakes minimizes the non-structural damage to the building.

Six of the eight transverse lines are moment resisting. Transverse moment frames were required on both sides of the stair towers because of the lack of horizontal diaphragm within the towers. The transverse moment frame girders range from W24x62's at the first floor to W18 × 35's at the roof. The non-moment transverse floor girders are W18 × 40's. Bay framing in the longitudinal direction is about 15-ft column-to-column. The three middle bays of the exterior longitudinal column lines are designed as moment frames. Girder sizes range from W24 × 55 at the first floor to W18 × 35 at the roof. In the transverse direction, the building is one bay wide. The bay width is 21 ft-9 in. The floor deck spans transversely over a W12 × 15 filler beam and cantilevers 4 ft-3 in. at the bay windows. Maximum deck span is slightly less than 11 ft. The non-moment frame floor beams are designed for composite action with the 3-in. deep metal deck and 2½ in. 3,000 psi lightweight concrete fill.

Savings in the cost of the building's fire safety system were possible by limiting the height from the first to seventh floors to 75 ft. To meet the height limitations, mechanical ducts penetrate the transverse floor girders. Typical floor to floor heights are 11 ft-8 in. Penetrations were coordinated and shown on the contract documents. This allowed shop fabrications of the penetrations and resulted in both a cost savings to the owner and better quality control.

The girders have both reinforced and unreinforced penetrations. Reinforcing the penetrations was done where required because of their large size and/or high beam shear. Round penetration sizes ranged up to 14 in. in the 18-in. deep girders. Reinforced and unreinforced penetrations were designed using the procedures published in the AISC *Engineering Journal*.

Moment frame connections are made by full penetration welds of the girder flanges to the columns and with A325F bolted web connections for the girder gravity and seismic shear forces. The frame was designed in accordance with the ductile provisions of the Uniform Building Code and the San Francisco Building Code.

Summary

The steel system proved simple and efficient. Erection took only 16 days. Steel solved the difficult foundations problems with a minimal amount of disruption and inconvenience to the neighboring buildings. And the building height limitation was met by coordinating the HVAC system with the structure during the design phase. □

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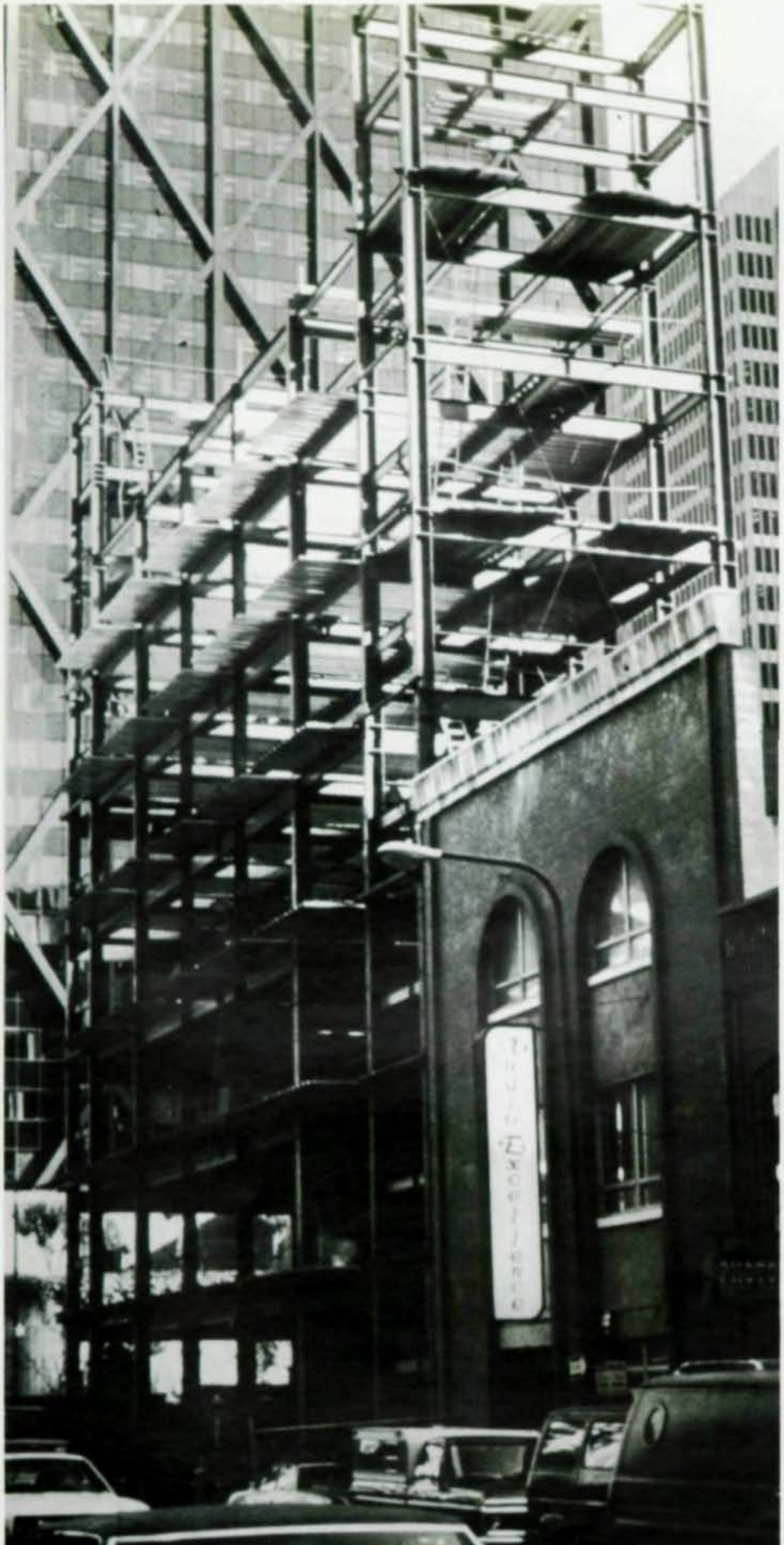


Open steel frame highlights challenge of erecting major office building on 25-ft lot.

SAN ANTONIO MUNICIPAL AUDITORIUM

New Era for a Fire-ravaged Landmark

by J. David Mack

New dome under construction, with stage-ouse framing and temporary shoring on left.

Dedicated to those from Bexar County, Tex. who gave their lives in WWI, the San Antonio Municipal Auditorium opened on Sept. 3, 1926. The building received the gold medal award from the AIA and quickly became a well known landmark on the edge of the city's central business district. *The building, conceived as a multi-purpose facility,* served the city well for over 50 years of housing civic events ranging from opera performances to high school graduations. But its present restoration was not brought about by economic factors or even by a growing city's needs, as might be expected for a building of this age. Instead, it was the result of a catastrophic fire on Jan. 6, 1979. The memories of many of the city's citizens appeared to rise with the smoke and flames that severely damaged *the building.*

The Spanish-styled building with Moorish influences is clad with Indiana limestone and ornate cast stone accents. The main roof is covered with standing seam metal panels, while the towers on either side of the entrance are topped with domes finished with brightly-colored tile work. Inside, the building contains much detailed plaster work as was the style of this period, with the focal point being the intricate plaster ornamentation outlining one of the largest proscenium arches, or stage openings, existing in this country. The building includes a kitchen, meeting rooms and basement exhibition space. A unique feature is the tilting floor at the center of the main auditorium seating area. It can be set in a sloped position for auditorium seating or in a flat position with the seating removed for a wide variety of events.

The auditorium is oval-shaped. Along one long side are the lobby, meeting rooms and the main entrance with the stage house along the other. Connecting these elements to each other at the ends of the oval are covered arcades. The stage floor is very large, 90 ft wide and 40 ft deep. The 193 × 142-ft domed roof over the auditorium is formed by two halves of a circular dome 142 ft in diameter with a rise of 14 ft, separated by a 51-ft long barrel vaulted section. The two half domes consist of four-ft deep trusses spaced radially at 30° angles. The center barrel vault is formed by two 4-ft pinned arch trusses spaced 25 feet apart. The radial and vault trusses support 3-ft deep cross trusses and I-beam purlins. The compression ring sections at the top of the half domes are 28 ft in diameter and connected to each other by three trusses running through the top of the vault. The half compression rings are prevented from collapsing inwardly by another truss between the end of each half dome.

The entire compression oval is topped by a steel-framed gabled cupola that originally provided ventilation and lighting at the center of the auditorium ceiling. The original roof deck was a 2-in. concrete slab cast-in-place on a heavy duty T-ribbed lath that served as a form as well as reinforcement.

The radial trusses are supported on steel columns at the perimeter of the dome, while the vault trusses are supported by additional trusses spanning across the 89-ft wide proscenium opening and the opposing rear balcony seating area. Twelve-ft deep vertical trusses carry the load of the barrel vault and 12-ft deep horizontal trusses carry the thrust. At the stage side, the vertical proscenium truss also supports the 40-ft high clay tile stage house wall, its roof, and the attendant grid system below that supports the stage backdrop rigging. The thrust from the barrel vault is transmitted through the horizontal trusses to two extremely large flat C-shaped trusses that also support the roofs over the arcades at the ends of the oval. The inside chords of these trusses also serve as the tension ring for the dome.

The balcony inside the auditorium is steel-framed and consists of triangular-shaped trusses supporting concrete tread slabs and channels forming the risers. The first floor is framed in concrete, except for the tilting section that measures 80 × 100 ft. The tilting floor originally had a hardwood floor supported on purlins spanning between trusses at 20 ft o.c. The trusses were inverted triangles in shape, 80 ft long across the top and 12 ft deep. They are supported on a single pivot point at their bottoms. The stage house is a rigid steel frame that rises to a height of 88 ft above the stage. On either side of the stage are the dressing room areas which are framed in concrete. The lobby roof is framed with steel purlins and beams while the main entrance and adjacent towers are cast-in-place concrete.

Fire Damage

The fire started in a storage area beneath the tilting floor adjacent to the stage. From there, it spread to the wood flooring on the tilting trusses and into the main auditorium. Consequently, the tilting floor structure was completely destroyed. Since the building was not sprinklered and the steel was not protected with fireproofing, the heat of the fire caused the dome to buckle and deflect. The worst damage occurred primarily in the section adjacent to the stage house. The extreme heat also affected the vertical and horizontal proscenium trusses, causing large inelastic deflections in them. The movements of the dome also caused stress reversals in the radial trusses on the entry side of the dome resulting in buckled chord members in several of the radial trusses. The ends of the flat C-shaped trusses over the arcades were twisted and warped where they connect to the built-up columns supporting the proscenium truss. The clay tile wall of the stage house supported by the proscenium truss was also severely cracked.

Fortunately, the roof did not collapse before the fire was extinguished. Its survival can be attributed to two factors. First, the inherent stability of the dome and the ductility of the steel members allowed it to continue to carry loads even when flattened and buckled. Secondly, the vertical proscenium truss, which also acted as the perimeter tension ring, did not fail. Its survival can be attributed to the plaster arch of the stage opening that acted as fireproofing. A large ventilating duct in front of the truss also shielded it from the heat.

Devastating damage (r.) to stagehouse wall. Below, exterior of completed mosque-like auditorium.

The stage house virtually survived intact since there were minimal amounts of combustible material at the stage level. Also, the roof structure was high enough above the main floor so as not to be affected by the heat. The balcony also suffered virtually no damage since its structure was protected by a 1-in. plaster soffit. Unfortunately, all of the interior finishes and furnishing were lost either to fire, smoke or water damage. But exterior damage was minimal, primarily cracked facing stone and smoke damage.

Design Sequence

Two days after the fire, the structural engineers made an initial visit to the building. At this time, only a partial set of plans had been located and no one fully understood the framing system yet or knew the extent of the damage. Therefore, it was decided that scaffolding towers should be installed at each end of the top oval compression ring to support the roof from possible collapse. On going architectural and engineering assessments and cost estimates were begun for the restoration at the request of the city, whose citizens later authorized a bond issue to finance restoration of this historic structure. The design team then produced plans to restore the building based on a program that included restoring the facility in its original style, while upgrading it to comply with present building and safety codes. Completely new electrical, plumbing and mechanical systems were also to be installed. The city also requested that a new tilting floor system be designed so that instead of housing floor trusses, the space below could be used to expand the basement exhibition space. This would enable the facility to house small conventions as well as the functions for which it was originally intended.

The design team chose to complete the restoration in two phases. Phase One would include all demolition work, roof repair, exterior repair work and other work necessary to put the building in a weatherproof condition. Phase Two would then include all of the mechanical, electrical and plumbing installations, the new tilting floor structure, new stairs, elevators and all interior finish work.

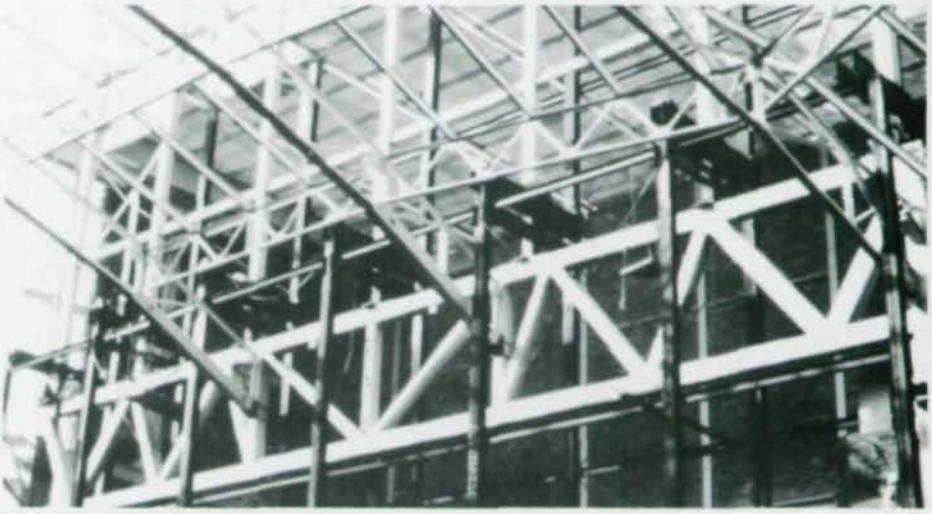
Fortunately, at the beginning of the design work, a complete set of structural plans was located at the architectural archives of the University of Texas, Austin. These plans included roof design loads and truss elevations showing member stresses. This design information allowed the structural engineers to fully understand the original design, thereby reducing the unknowns that would have otherwise been present in the restoration. When the detailed investigation of the structure was completed, it was found that the plans and as-built conditions generally agreed, except that some bracing trusses inside the compression ring were found that had not been shown on the plans. Even though the plans did not contain many framing or connection details, they proved to be of immeasurable value during the restoration. The design live load shown on the plans was 25 psf which is about twice what is required by today's codes, especially in a region where snow loadings are not a factor.

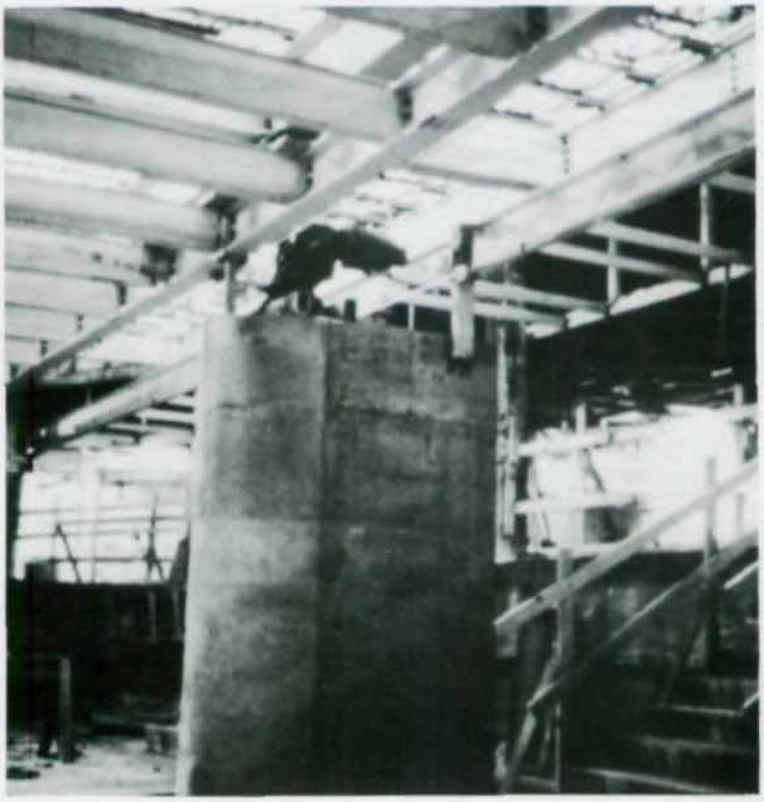
Preliminary Investigation

When work began on Phase I documents, it quickly became apparent that some demolition work needed to be done prior to its completion. Many structural members were obstructed, and those that were not could not be seen clearly in the dark caverns of the auditorium. It was decided that one-third of the concrete roof deck should be removed from the stage house side of the dome. This would allow enough natural light to enter to permit investigations of major connections and steel framing details, and still leave the remainder of the building with some protection from the weather. It would also allow the design team to determine how easily this roof deck could be removed from the original framing, an issue that had raised some concern. It had the added benefit, as well, of removing a significant portion of the dead load from the most severely damaged section of the roof. This work was completed successfully and a detailed structural investigation of the entire roof structure, member by member, performed.

The investigation verified that the majority of the damage was to the stage house side of the dome, except for the buckled radial truss chord members previously mentioned. It also revealed that the top

Interior (above) of restored auditorium. Vertical 12-in. deep proscenium truss (below). Two barrel vault trusses also erected.





New tilting floor framing

compression oval ring survived with only minor damage due to the stiff bracing trusses at each end. The ring though was 12 to 14 in. lower than the plan elevations and had shifted 2 to 4 in. toward the stage house in a horizontal direction. The columns supporting the proscenium trusses were twisted at their tops as were two adjacent columns supporting the dome. The vertical proscenium truss itself was bowed about 18 in. out of line horizontally. The ends of the flat roof C-shaped trusses attached to the proscenium columns had been twisted and warped.

Steel samples from both the damaged areas and the remainder of the structure were taken and used to determine yield strengths. Since welded connections were essential to the repair of the structure, weld tests were performed to verify the weldability of the steel. The original connections were made with rivets, but it would be impractical to detail new bolted connections to replace the rivets without numerous field fit-up problems. The yield strength was determined to be 30,000 ksi and the steel was found to be weldable with current techniques.

Phase I

After the detailed structural investigation had been completed, several major design decisions had to be made. It was decided the entire concrete roof deck should be removed and replaced with a metal deck and rigid insulation board system. The concrete deck had been badly cracked during the fire. Also, the rib lath on which it was cast was beginning to rust and it was impractical to clean and paint it. An added benefit of removing the concrete deck was that the roof dead load would be reduced significantly, thereby decreasing the working stresses in the original structure that would remain.

The next decision to be made was whether or not to replace the entire domed roof or only replace the damaged members. The argument for a total replacement of the roof was that there would be no alignment or connection problems between new and existing members to contend with. There would also be no risk that some undiscovered damage had weakened an existing member still in place. The argument for a partial roof replacement was based primarily on the results of cost estimates that showed it would cost less than a full replacement. In countering the questions concerning connection alignment, it was argued there was enough flexibility in the steel framing to allow it to be jacked from the supporting scaffolding back to its proper position. In its proper position, the alignment of the connections to the new steel could then be made. Since the original steel was weldable, it was maintained new members could be connected to existing members with welded connections that would not require extremely close tolerances. Since the dead load was being reduced and the live load was overly conservative, any risk of problems due to uncovered damage would be minimal.

With these factors in mind, it was decided to produce plans for the partial replacement of the domed roof. It was recognized actual costs would be determined by the successful competitive bidder and that due to the unusual nature of this project and the accompanying risks involved, the design team's estimates may not be entirely reliable. Therefore, a bid alternate of replacing the entire roof structure was included in the documents. When the bids were received, the low bidder determined the partial replacement scheme to be less costly.

In developing the partial replacement plan, it was decided that approximately two thirds of the roof structure should be replaced. The entire compression oval was also replaced, since it was felt a new one erected in the proper position would be essential to properly align both old and new members. Additional shoring towers with screw jacks were to be installed to support and raise the ends of the remaining trusses to their proper position. Also, a specific sequence for the removal of major damaged members was given so the remaining structure would not suffer further damage while it stood in a partially completed state.

The new dead loads, which were somewhat less than the original dead load were used for the design of the new members. The original live load of 25 psf was used again, even though it was overly conservative. It was decided the original live load should be used to restore the building to its original load carrying condition. In doing so, working stresses in the members would also be kept at a low, giving an added measure of safety in the remaining original framing. Also, the final ceiling and roof mechanical loads for the main auditorium were not known at the time of the Phase I construction. It was felt the conservative live load would act as a buffer in case the final Phase II loads were higher than anticipated.

In designing the replacement framing, it was decided to use ASTM A-36 steel and the current AISC code. All new trusses and beams were designed to essentially the same dimensions and configurations of the original ones. The members of the new radial trusses, as well as vertical and horizontal proscenium trusses, were redesigned since a significant weight savings could be acquired. The members of the cross trusses were left virtually unchanged since low stress levels caused their sizes to be governed by stability criteria which resulted in practically no weight savings. All new trusses were of welded steel construction.

Sections of the C-shaped trusses replaced were also redesigned to save weight. Even though these C-shaped trusses were designed to resist thrust from the vault, it is doubted the entire structural system performed in this manner. If the C-shaped trusses did indeed resist the thrusts, the deflections in these large trusses would have been significant. Since no evidence of such deflections was found, the thrusts were instead transferred into the stiff framing of the stage house and opposing entry area. It was evident though, that during the fire, these trusses contributed significantly in bracing the proscenium columns and trusses. With this in mind, and to restore the structure to its original design, the original assumptions were used in sizing members of these truss replacement sections.

Connections between existing trusses and new trusses were generally made with shear plates welded to existing verticals and gusset plates, with care being taken to avoid the rivets on the original trusses. There were four existing built-up columns consisting of angles and plates riveted together that had to be replaced because of twisting during the fire. These columns were cut off at a level below the damage. A plate was then welded onto the existing stub, and a new column section welded to the plate. This technique also allowed the connection of the horizontal and vertical proscenium trusses to be made to the new stub instead of having to work around the numerous rivets in these heavy built-up columns.

The remaining significant problem facing designers was how to replace the vertical proscenium truss supporting both the dome and the stage house roof. It was considered too costly to completely disassemble the undamaged stage house roof, so a system of shoring columns, one row on either side of the existing truss was devised with screw jacks and needle beams across the top of this truss to support the stage house columns. The domed roof and the original truss were then disassembled and removed. The new proscenium truss, which weighed 36 tons, was shop-fabricated in three parts and lifted up between the two rows of shoring columns and welded in place. The stage house columns were then jacked up to their proper location and welded to the proscenium truss. Phase One construction proceeded smoothly and the

Detail of new column stub welded to existing column, which supports proscenium trusses to be attached on left side.

designer's decisions were justified in that no major alignment problems during reconstruction occurred.

Phase II

The unique structural feature included in Phase II was the new tilting floor design. To utilize the space occupied by the original 80-ft × 100-ft floor trusses, the new floor would be supported on nine new concrete columns placed in a three-by-three grid. The floor was divided into two 40-ft × 100-ft sections with a pivot point at the middle row of columns. Screw jacks were located at the other column rows to either raise or lower the floor. The columns also housed air conditioning ducts running from chases below the floor to the attic space above. Besides a new floor system, a separate structural frame was required below the floor to support a fixed plaster ceiling to act as a fire barrier between basement and auditorium. Since this frame was a fire barrier, it could not be connected to the floor above.

The floor was designed for a live load of 100 psf to accommodate a wide variety of loading conditions. Since the movable floor needed to be as light as possible, and because a limited attic cavity restricted the structural depth of this system, a steel framing system was deemed the only feasible solution. Each section of floor consists of open-web bar joists resting on wide-flange beams supported on the jacks and pivots on top of new concrete columns. The entire floor received a 2½ in. concrete slab on metal deck. Close attention was paid to deflection and vibration parameters in the design of this floor. Since the auditorium floor space was an open area with heavy foot traffic in the aisles, vibration could be a potential problem. This problem was aggravated by the requirement to keep the dead loads to a minimum. The ceiling structure below consisted of light wide-flange purlins supported by wide-flange beams attached to the columns.

As part of the upgrading of the structure, new elevators and stairs were installed in the auditorium to serve the balconies. Since the balconies were framed in steel, it was a simple matter to rework the framing and add additional members as required to form the openings for the elevators and stairs. Other new features added during Phase II included a steel gondola on screw jacks hung from the center of the roof. This gondola houses ceiling lights on the bottom of a catwalk which also supports spotlights above. It can be lowered to a position so the spotlights may be used for stage productions. New platforms and catwalks in the stage house required for the new modern stage rigging equipment were also added.

A unique construction technique used by the contractor on the Phase II project was a working platform out of scaffolding sections suspended from the domed roof. This eliminated floor scaffolding and allowed work on the ceiling to be done concurrently with the installation of the tilting floor and finish work done at the main floor and balcony levels. The engineers reviewed the contractor's plans and decided that with the 25 psf design live-load capacity, the platform could be used safely during construction. This scheme reduced significantly construction time, and therefore cost, by permitting the contractor to work in two areas at once.

Summary

Two factors contributed to the successful restoration of this facility. The first was the manner in which the design team addressed its unique problems. The search for and location of the original structural plans, along with the thorough early investigation in the design phase, enabled engineers to produce complete and detailed plans. This was followed by frequent site visits during construction, allowing the design team to work closely with the contractors on this unusual project. The end result was a restoration project moving smoothly with no major unforeseen conditions arising.

Secondly, the qualities of structural steel, the primary building material in this project, were a significant factor. The ductility of the steel dome enabled the structure to survive the fire, leaving enough of the structure undamaged so as to make restoration feasible. Since it is relatively easy to repair structural steel members either by replacement or strengthening, the cost of the restoration were affordable. Also, for the same reasons, it was relatively easy to modify the structure during remodeling and upgrading processes, a significant part of Phase II.

The grand-reopening of the San Antonio Municipal Auditorium was held on Feb. 24, 1985. The facility was open to the citizens of the city with civic and cultural programs being held within the hall. This opening marked the end of a successful restoration project and the beginning of a new era for this treasured historic landmark. □

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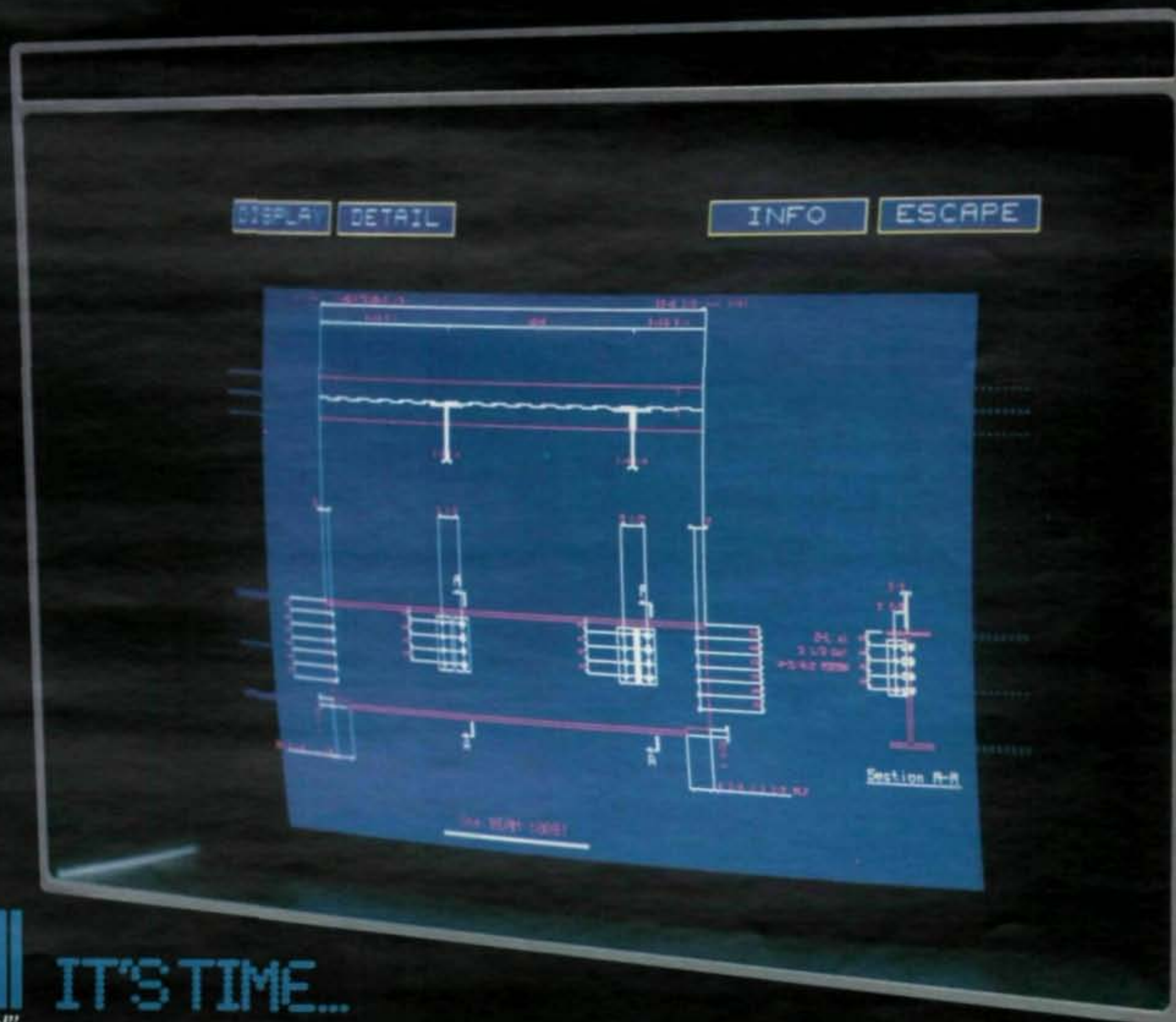

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