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<th>12 GAGE TOP HAT/16 GAGE BOTTOM PLATE HEAVY SLAB FORM.</th>
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<th>3/4&quot; FORMED PLATE—HEAT SHIELD.</th>
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<th>METAL THICKNESS</th>
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MODERN STEEL CONSTRUCTION
VOLUME XXVII • NUMBER 3
MAY-JUNE 1987

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So we called the service center. Eventually, they sent us a test report that did match our beam: it said heat number 34.005, from JMA in Madrid, Spain. That’s Madrid, Spain.

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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 Ellifrit 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...
“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.

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Alternate Design

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.
Number 3 / 1987
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
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(See Reverse Side for Abstract Guidelines, Preparation of Final Papers, etc.)

Return this form **before August 31, 1987** to:

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Attention: Lona Babbington (Phone: 312-670-5432)
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.

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* 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.

**Topics of Particular Interest**

* Practical application of research results

* Advances in steel building and bridge design and construction

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* Partially restrained connections and frames

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* 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.
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 space.

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 dis-

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

Fig. 3. Upper exhibition level showing tapered column capitals

plays. 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 Information mart 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
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 3½-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.](image-url)
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 6/4-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 Informan 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."

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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.
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.

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The sports facilities division of
Ewing Cole Cherry Parsky
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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
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.

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Skidmore, Owings and Merril
San Francisco, California

General Contractor
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David Moulton is an architect with Skidmore, Owings and Merrill. San Francisco, California.

Navin Arni is an associate partner with Skidmore, Owings and Merrill.
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OOPS, SORRY DEPT.

In the last issue of Modern Steel Construction (No. 2-1987) in Clel­ lon 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 pur­ chase. 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/Master­ Card 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 administering the awards. Students are 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
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Architectural Solution

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 x 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 x 35's at the roof. The non-moment transverse floor girders are W18 x 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 x 55 at the first floor to W18 x 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 x 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.

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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 un-reinforced 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.

Architect
Fee & Munson, in association with Sid Hoover

Structural Engineer
Vogel & Meyer Partnership

General Contractor
Herrero Brothers

Owner
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John D. Meyer is a partner in the structural engineering firm of Vogel & Meyer, Walnut Creek, California. Deborah Michels was the marketing coordinator for Vogel & Meyer.
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Landmark Restoration

SAN ANTONIO MUNICIPAL AUDITORIUM
New Era for a Fire-ravaged Landmark
by J. David Mack

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-style 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 x 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.
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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-ripped 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 x 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.

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 re-
pair, 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...
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 col-
umns supporting the proscenium trusses
were twisted at their tops as were two adja-
cent 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 at-
tached 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 weldabil-
ity of the steel. The original connections
were made with rivets, but it would be imprac-
tical to detail new bolted connections to
replace the rivets without numerous field
fit-up problems. The yield strength was de-
termined to be 30,000 ksi and the steel
was found to be weldable with current tech-
niques.

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 re-
moved and replaced with a metal deck and
rigid insulation board system. The con-
crete deck had been badly cracked during
the fire. Also, the rib lath on which it was
cast was beginning to rust and it was imprac-
tical to clean and paint it. An added ben-
et of removing the concrete deck was
that the roof dead load would be reduced
significantly, thereby decreasing the work-
ing 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 mem-
bers. The argument for a total replacement
of the roof was that there would be no
alignment or connection problems be-
tween new and existing members to con-
tend with. There would also be no risk that
some undiscovered damage had weak-
ened 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 align-
ment, 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 con-
ected to existing members with welded
connections that would not require ex-
tremely close tolerances. Since the dead
load was being reduced and the live load
was overly conservative, any risk of prob-
lems due to uncovered damage would be
minimal.

With these factors in mind, it was decid-
ed to proceed plans for the partial replace-
ment 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 reli-
able. Therefore, a bid alternate of replac-
ing the entire roof structure was included in
the documents. When the bids were re-
ceived, the low bidder determined the par-
tial 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 re-
placed. The entire compression oval was
also replaced, since it was felt a new one
erected in the proper position would be es-
sential 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 se-
quence 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 some-
what 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 conserva-
tive. 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 mea-
sure of safety in the remaining original
framing. Also, the final ceiling and roof me-
chanical 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 antici-
pated.

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 hori-
zontal proscenium trusses, were rede-
signed 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 con-
struction.

Sections of the C-shaped trusses re-
placed 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 truss-
es 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 contribut-
ed significantly in braking 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 re-
placement 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 to-
gether 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 con-
sidered too costly to completely disassem-
ble the undamaged stage house roof, so a
system of shoring columns, one row on ei-
ther 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-fabricat-
ed 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 weld-
ed to the proscenium truss. Phase One
construction proceeded smoothly and the
designer's decisions were justified in that no major alignment problems during re-
construction 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 \times 100-ft floor trusses, the new floor would be supported on nine new con-
crete columns placed in a three-by-three grid. The floor was divided into two 40-ft \times
100-ft sections with a pivot point at the middle row of columns. Screw jacks were lo-
cated at the other column rows to either raise or lower the floor. The columns also hous-
ed air conditioning ducts running from chases below the floor to the attic space above. Besides a new floor system, a sepa-
rate 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 bar-
rier, 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 fea-
sible solution. Each section of floor con-
ists 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 2\frac{1}{2} in. concrete slab on metal deck. Close attention was
paid to deflection and vibration parameters in the design of this floor. Since the audito-
rium 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 bot-
tom of a catwalk which also supports spot-
lights 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 sec-
tions 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 bal-
cony 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 con-
struction. This scheme reduced signifi-
cantly 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 ad-
dressed its unique problems. The search
for and location of the original structural
plans, along with the thorough early inves-
tigation in the design phase, enabled engi-
nieurs to produce complete and detailed
plans. This was followed by frequent site
visits during construction, allowing the de-
sign team to work closely with the contrac-
tors 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 duct-
ility of the steel dome enabled the struc-
ture 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 ei-
ther 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 re-
modeling and upgrading processes, a sig-
ificant 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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San Antonio, Texas

Mechanical/Electrical Engineer
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