



Arizona Diamondbacks Bank One Ballpark

Phoenix, Arizona

Kicking off a new trend in stadia design, the Bank One Ballpark, home of the Arizona Diamondbacks, features a moving roof. The 49,500-seat stadium, which rests on 24 acres of land in downtown Phoenix, has become both a tourist attraction and landmark for the city of Phoenix.

The owner's fundamental objective with Bank One Ballpark was to create a dynamic and unique home for the Arizona Diamondbacks baseball franchise. Not only did the stadium need to have its own special characteristics (including a swimming pool behind center-field), but also it had to have natural grass and be air-condi-



tioned. In order to accomplish this, a stadium with a moving roof was designed and built.

The objective of the design team was to meet the requirements listed above and also to:

- Design a modern major league quality baseball stadium;
- Exceed the requirements for disabled access of the Arizona Disability Act Guidelines;
- Provide opportunities for revenue;
- Provide a facility that can be used for events other than baseball;
- Open for the 1998 baseball season;
- Incorporate the latest technology where appropriate.

STRUCTURAL CONSIDERATIONS

A combination of steel and concrete was used to maximize the efficiency of the structure. While the overall frame was cast-in-place concrete supported on spread footings, the roof, roof support structure, cantilevered seating and exterior walls all were steel members.

In keeping with current stadia design trends, the Bank One Ballpark was designed to be

reminiscent of an “old-time” ballpark. To capture the desired look, much of the steel—including the 40’ cantilevered trusses—was left exposed. Additionally, the structure was designed to allow for large open concourses for fan comfort while also providing opportunities for revenue locations.

To satisfy the seemingly conflicting need for both natural grass and air conditioning, the structural engineering design team developed a moving roof stadium capable of closing during an event. However, the tight site prevented the design team from moving the roof completely off the stadium. This presented the design team with the challenge of combating “sunshadowing” on the natural grass playing field. This, in turn, created the design challenge of somehow reducing the height of the structure to compensate for the “sunshadowing.” This was done by developing a telescoping and stacking structural system. This unique design is driven by a very economical and safe cable and winch drive system using gantry crane technology. The structural

engineer of record worked very closely with the owner, architect, and construction manager to integrate the compressed design and construction schedule on the tight downtown site.

UNIQUE STRUCTURAL FEATURES

To allow for maximum flexibility and sunlight control, each half of the roof can move independently. The roof is cable driven using gantry crane technology. Six telescoping panels are retracted in 4-1/2 minutes, revealing a 5.3-acre opening to the sky. Each panel is supported by the panel beneath it and moves on a steel wheel guide roller system. This piggyback arrangement reduces the structure height and allows the most sunlight to reach the natural grass field given the tight site limitations. A computer control system adjusts the roof opening similar to a camera aperture for the angle of the sun, allowing the maximum amount of light on the field and the least on the seats to reduce heat gain.

The moving panel system was also used as a construction aid, substantially reducing the need



for shoring. Fixed panels were fabricated on the ground and lifted into place. The moving panels were then erected from this elevated "platform." The stadium bogie and rail systems were also used during construction to facilitate positioning and storing of panel sections. The six 800-ton panels are pulled by 4 miles of steel cable attached to two gearboxes and two 200-horsepower electric motors. To reduce the weight of the roof and its effect on the moving system, high strength steel (65 ksi) was used in the roof trusses. The low roof profile also reduced the size of the drive motors required to "drive" the roof into the wind.

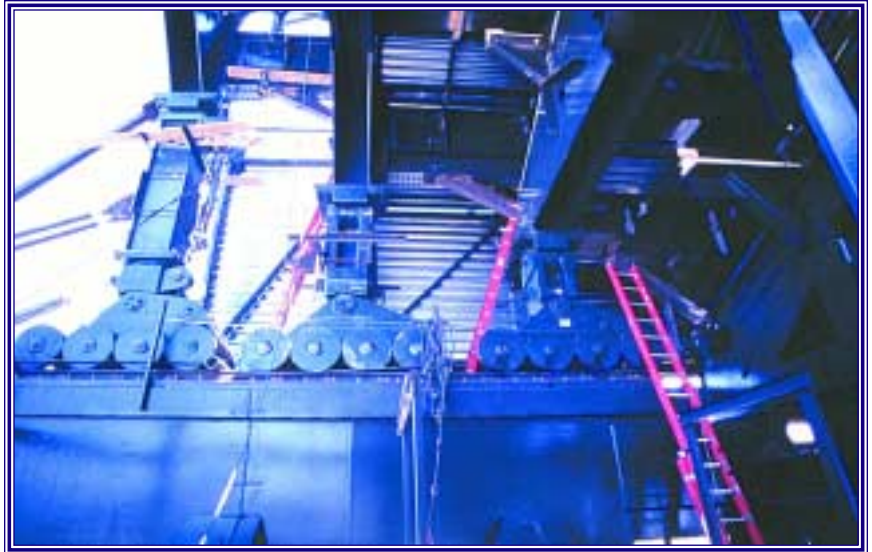
Integrating the design and construction process was a huge challenge. The structural engineers spent the equivalent of 15 man-years on the design of the roof portion of the stadium. The compressed schedule was dictated by lack of funding to start design and construction until the baseball team was awarded to Phoenix and the inflexibility of the completion set for opening day. Design and design packaging decisions were dictated by the construction methods and construction schedule. To allow for safety and sequencing of concrete work, and, to eliminate the need to shore the overhead steel, the fixed steel roof trusses at the ends of the stadium were accelerated in design and erected before the concrete work was started. The stadium substructure (seating, concourses, etc.) was built after the roof sections were in place - this is the opposite of normal construction sequencing. Adding to the complexity was the tight site, bordered by streets on three sides and railroad tracks on the fourth.

As design started, it was determined an outfield wall would need to be constructed to support the roof and close the building for air conditioning. The roof's height would also need to be reduced as much as possible to compensate for "sunshad-



owing.” The structural engineer reduced the height of the structure, compressing the roof to a total stacked height of 243’ while achieving 200’ clear above the playing field. Multiple loading analysis were performed on the loading conditions that change with the slightest movement in the roof. The number of load cases was magnified because of the desire to be able to position the roof at any location regardless of the weather conditions. The intense Phoenix summer heat added to the difficulty during construction by causing the steel trusses to expand and contract daily by several inches. The stadium roof successfully withstands thermal, wind and seismic forces; as well as the dynamic loads of the moving structure. Because the roof was designed to resist all of the required code loads at any position, opening the roof has become a game event.

Intricate computer modeling, well beyond the usual applied to building structure and accounting for temperature variation, moving loads, dynamic effects and changing geometries, became a vital tool throughout design and construction. The modeling included a nonlinear P-Delta buckling analysis linking the independent moving panels.



Jurors' Comments

“The use of a moving, long-span structure in a harsh environment is a significant achievement.”

“A simple and elegant solution to a complex problem.”

“An engineering marvel, considering the configuration and space requirements together with the design tolerances and environmental conditions.”



ENGINEERING SIGNIFICANCE

The \$350 million Bank One Ballpark opened on time and met the total goals and concepts of the client. Meeting the opening day completion date was a huge accomplishment given the schedule and the fact that this was the first of its kind. This was a result of the extraordinary efforts of the design and construction team. The engineering that was done at the ballpark has provided baseball fans with a modern venue to watch major league baseball games. It is a very "user-friendly" stadium, and the engineering played a very significant role in that fact. Concourses are easily accessible, seating is comfortable, and the structure itself is very safe and able to provide numerous amenities.

Bank One Ballpark not only accommodated the specific parameters of a moveable roof that allows for natural grass and air conditioning; it exceeded all anticipated possibilities. The 40,000+ baseball fans that flood the downtown streets 80 nights during the summer months have revitalized the nightlife of Phoenix. This "Economic Engine," as the ballpark has been referred to, is estimated to have an annual economic impact of over \$300 million to the downtown Phoenix area.

The scope of the project is demonstrated by the magnitude of construction requirements.

- The 49,500-seat ballpark was designed and constructed in 36 months;
- 1,250,000-sq. ft.
- Structure height: 250';

- Field dimensions: 335' right field; 328' left field; 376' power alleys; 407' center field;
- 69 luxury boxes;
- 6 party boxes;
- 4,400 club level seats;
- 1,200 field level club seats;
- 18,000 tons of steel;
- 60,000 cubic yards of concrete;
- 5.5 acre opening over the field;
- Roof Size: 376,000-sq. ft.;
- Roof Weight: 7,600 tons (40.5 psf);
- Roof opens or closes in 4½ minutes;
- The structural engineer of record logged approximately 69,000 total engineering hours on the ballpark, 30,000 of which were dedicated to the roof.

Project Team

Owner:

Maricopa County Stadium District, Phoenix

Structural Engineer:

Martin/Martin, Inc.
Wheat Ridge, CO

Architect:

Ellerbe Becket, Inc.
Phoenix

General Contractor (joint venture):

Perini McCarthy & Perini/Tutor-Saliba

Steel Fabricators:

Schuff Steel, Phoenix
Zimmerman Metals, Denver
Mountain State Steel, London, UT

Steel Detailer:

Schuff Steel, Phoenix

Joist Manufacturer:

Vulcraft



Golden State Warriors Training Facility

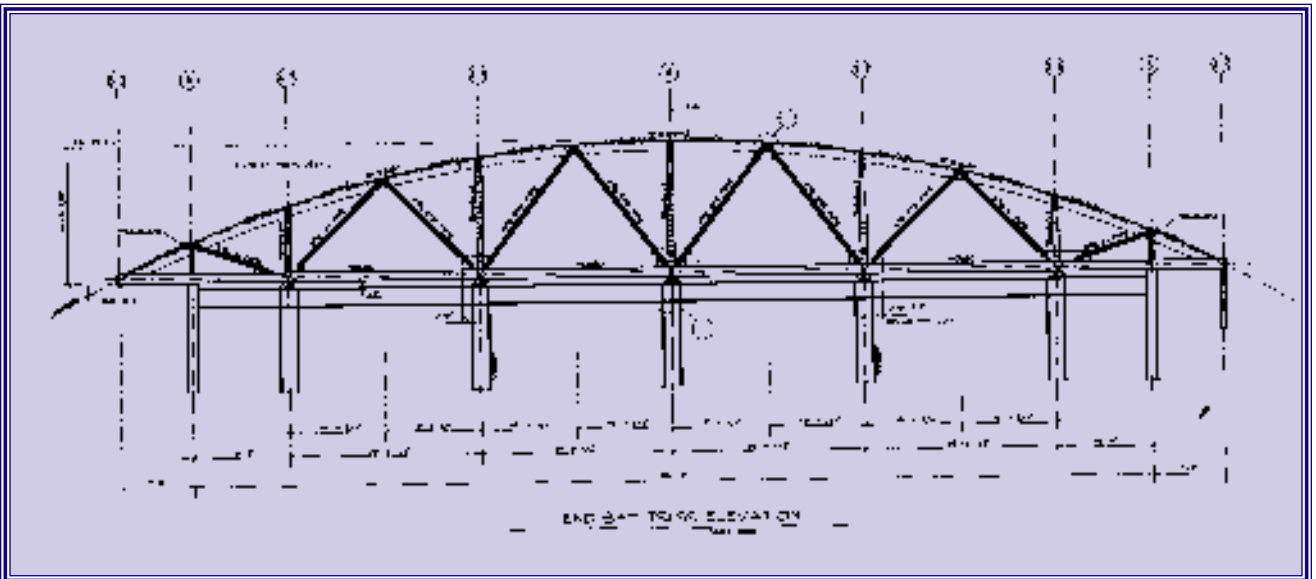
Oakland, California

After the National Basketball Association Golden State Warriors relocated from San Jose to a new arena in Oakland, the owners decided to consolidate their offices with a new training facility. The city of Oakland offered them the roof of their existing

convention center building—then an automobile parking level—as the new site. The Warriors' program called for practice courts with a 100' clear span and 30' clear ceiling height, corporate offices with coaches' offices overlooking the courts and locker rooms, health spa and

storage facilities.

The four-story steel-framed convention center was built in the early 1980s. The ground floor, with a clear span of 156', is used as a convention space. The existing structure uses a 10' deep plate transfer girder to support the three levels of parking



above to provide the 156' clear span. San Francisco-based Structural Design Engineers' investigation of the existing structure showed that it had excess capacity in its lateral two-way steel moment frame system, as well as the major column foundations supporting the convention center transfer girders. The excess capacity in the lateral frame was a result of the 1980s design practice of using non-standardized response spectra, which resulted in a higher level of seismic excitation than the

1994 Uniform Building Code criteria. Since the original construction complied with the ductility requirements of the current code, the lowered base shear for the building allowed for a light structure of a limited weight (not exceeding 11% of the existing building mass) to be placed upon the roof of the existing building. The existing concrete pile foundations for the major columns supporting the transfer girder had been conservatively designed such that a limited capacity for additional new loads

was available for the proposed addition. The transfer girder could take a small additional load provided the loading was close to its supports. Finally, the columns and moment frame girders of the convention center were sized for seismic drift for a force level larger than the current code standards.

In summary, the challenges to the design team were:

- Vertical addition of a light two-story volume structure on an existing occupied structure with minimal or no disruption



in operations.

- Meet the Warriors' program requirements for the facility.

The structural engineers, in conjunction with Oakland-area architect Charles F. Jennings Architects, achieved the program requirements within the structural constraints of the existing building. They used structural steel bowstring trusses spanning 104' to provide the clear spans for the courts. Lightweight steel decking, insulation and metal roofing were used to keep the weight to the required minimum loads. The partial upper 6th floor for the corporate offices was hung from the roof trusses. By so doing, all gravity loads were placed near the transfer girder supports and on the heavy existing building columns.

To match the lateral system characteristics, the new structure utilized perimeter moment frames. Two innovative concepts were utilized to meet these requirements. The first was the use of the truss bottom chord as a collector of seismic loads to be delivered to the steel moment frame through a pin connection. The second innovation was the use of a coupled girder at the 6th floor to reduce weight of girders and columns. Because the new columns could not be rigidly connected to the top of the existing building box columns, the tall, 17.5'- height of the addition posed a stiffness problem. This was solved by the use of two girders at second floor and by coupling of the girders to create a stiffer structure. The coupling of the girders was achieved by vertical links at one-third of the girder spans. The coupled girders formed a Vierendeel truss, providing higher stiffness and reducing the effective height of the 17.5' tall first story columns. At 9', the door and window head allowed more than 7' for the depth of the coupled girders. A plate girder of that depth not only would have been expensive, but also would have violated the concept of strong column - weak beam, a ductility concept essen-

tial to seismic performance of structures.

In summary, design solutions to the challenges include:

- 1) Use of steel trusses and metal decking to deliver the loads close to existing columns, which carry the transfer girder to the heavy foundations.
- 2) Innovative use of a coupled-girder moment frame system to reduce steel weight and increase efficiency of the moment frame system.
- 3) Reduction of total weight of the addition by using steel roof trusses and metal decking. Also, by using 2" metal deck and 2 1/2" lightweight concrete for the partial second floor of the addition, thus keeping the added weight of the 56,000 sq. ft. addition within the required 11% excess capacity of the existing structure.
- 4) Innovative combination of trusses and moment frames at the roof, where the diagonals and bottom chord of the truss were used to deliver the seismic and wind loads to the moment frame through a pin connection.
- 5) Special erection techniques were developed to manage the placing of trusses, which were shipped in two segments and assembled on the roof. The erection sequencing allowed the dead loads to be transferred to the 104' span before welding of the moment frames. The transverse moment frames do not impose significant gravity loads on the transfer girder below even though the moment frame column spacing is approximately 21'.

The project was successfully completed in 1998, and is currently occupied by the Golden State Warriors.

Project Team

Owner:

The Golden State Warriors

Structural Engineer:

*Structural Design
Engineers, San Francisco*

Architect:

*Charles F. Jennings
Architects, Oakland*

General Contractor:

*Dinwiddie Construction Co.,
San Francisco*



Swiss Bank Corporate Headquarters

Stamford, Connecticut

The new Swiss Bank Headquarters Complex, located in Stamford, CT, has been designed to accommodate the relocation and centralization of the majority of bank's office and trading operations from three buildings in New York City. The Phase I project, constructed on a 12-acre site,

consists of a 15-story tower adjoining an 8-story parking, technology and trading facility.

A unique feature of the project is the trading arena sitting atop a seven-story base, which, at 144' long and 240' wide, is the world's largest clear-span trading area. The roof framing consists of exposed king-post trusses constructed of a curved box gird-



er with two underslung cables held apart by a central post. The cables, requiring exact construction tolerances, were prestressed without the use of turnbacks or other mechanical hardware. A small section of the final post was slid over a narrower guide post, the cables were set in their groves and, like an arrow being drawn, the post with cables was pulled down the narrow guide post until the required stress was obtained. Half pipes, field cut to exact lengths, were then welded in place over the narrow post to complete the final post section. At the north end of the girder span, the king-post-trusses are supported on a 30' high clerestory truss, which provides the space with indirect natural daylight. The ceiling configuration of the trading room curves from 35' high at the south end to 50' high at the north end.

Other major features of the

Jurors' Comments

"Very creative in meeting the owner's requirements for a large clear-span area"

"Bottom chord cable trusses with mechanical fastening devices simplified construction."

"Very efficient structural solution related to the lateral loads at trusses."

project include the seismic separation of the 8-story parking/trading facility from the 15-story office structure (which

required the design of unique details for the post tensioned portions of the structure); and the design of the large, structurally reinforced window wall of the grand entrance lobby together with its suspended staircase and mezzanines.

The six-story-high, 250'-long window wall of the main entrance lobby required special design consideration. Unbraced circular composite concrete and steel columns, rising the six stories, support cantilevered steel outriggers, which support a horizontal, and vertical tube system that carries the glass sections. In addition, the lobby contains a five-story staircase connecting four stories of mezzanines. Both the staircase and mezzanines are suspended by hanger rods and cables from the sixth floor framing.

Through use of different structural systems and materials, the client's diverse space needs were accommodated and integrated within a unified complex. Unlike working with a developer on a speculative office building, working hand in hand with the bank's internal corporate real estate, information technology, and construction management teams allowed the design team to create a complex uniquely tailored to the specific needs of the bank.

The office tower accommodates 40' and 60' core-to-wall lease-spans and 9' floor-to-ceiling heights with a 6" raised floor throughout. Accommodating this floor to ceiling height, while minimizing the building height and matching the parking floor levels, required increased coordination efforts by the design team. In addition to normal office functions, the tower also provides space for technology and cafeteria functions as well as the Center for Learning and Development located on the top two floors of the tower. The Center for Learning and Development provides space for employee training and development in the auditorium and conference facilities.



The parking facility, which is located beneath the technology and trading centers, is a post-tensioned concrete structure. Combining post-tensioning with high quality concrete, appropriate admixtures, and rigorous concrete quality control has provided the owner with a durable maintenance-free parking garage with only a slight increase in construction cost.

The technology center located at the fifth floor of the parking and trading building, houses the entire complex's state of the art information and communications systems. All of the utilities required for business operation have multiple degrees of redundancy of M.E.P. and communication distribution systems.

The Phase I project provides 594,000-sq.-ft. of rentable space and 377,000-sq.-ft. of enclosed parking. Two future phases of the project include two additional office towers and the expansion of the parking and trading facility by 50%. Full build-out of the development will contain up to 1.7 million-sq.-ft. of office space plus 1.2-million-sq.-ft. of enclosed parking. The design team considered the impact of future phases in the design and construction of Phase I.

While the budgeted cost for the structural portion of the project was \$19,993,000, the actual cost of construction of the structure is \$24, 997,000. The variation in cost is due to the addition of 21,000-sq. ft. to the trading facility and a redesign of the office tower to accommodate owner requirements.

Project Team

Owner:

Swiss Bank Corporation,
Stamford, CT

Structural Engineer:

Thornton-Tomasetti
Engineers, New York City

Architect:

Skidmore, Owings & Merrill,
New York City

General Contractor:

Turner Construction Co.,
New York City

Steel Fabricator:

Cives Steel Co., Gouverneur,
NY



Hawaii Convention Center

Honolulu, Hawaii

The \$200 million, 1.1-million-sq.-ft. Hawaii Convention Center in Honolulu encompasses all the elements of a tropical paradise. The imagery of palm trees, sails, sunlight, warm breezes, waterfalls, and exotic flora can all be found within the design of the convention center.

Completed in October 1997, the facility was on budget and 1 month ahead of schedule. The center is expected to attract

400,000 additional visitors to Hawaii each year, generating \$3.5 billion in new annual tourist expenditures and \$178 million in new tax revenues.

A one-of-a-kind convention facility in a one-of-a-kind place, the Hawaii Convention Center captures the friendly and exciting spirit of "Aloha."

An international competition with world-class participants was the method used to solicit designs for the facility. The four

Jurors' Comments

"An outstanding design, especially given the site and loading constraints. This structure is a work of art—from the design complexity to the detailing and fabrication."

"A very innovative design with ground level convention space. The development of a supertruss helped create a very efficient structural system."

"The development of special software was critical in the design of these unique connections."

competing teams faced a very challenging building program:

- The owner's program did not fit on the site without stacking the functions on many levels.
- The water table was just 4' below grade.
- The potential existed for flooding hydrostatic pressure.
- The soil was deep, soft lagoonal deposits.
- The design, permitting, and construction time totaled only 750 days.
- The construction site was a very tight urban block located directly on the busiest intersection in Hawaii.
- The contractor had to guarantee the price based on only the competition submittal drawings, which had to be completely developed in just a little over two months.

The winning design was the only one submitted that was able to have setbacks and terraces, due, in large part, to the struc-





tural system developed. It was also the only competition entry that achieved a ground-floor exhibition hall and provided ideal structural modules for each usage area.

A three-dimensional “super-truss” structural system reduced building height by 50’ and allowed the terracing and setbacks. A double-pitched Hawaiian roof, architecturally exposed steel “tree” columns, fabric rooftop “sails,” tapa-patterned concourse roof framing, and an underground utility corridor are other unique structural elements incorporated into the successful design.

INNOVATIVE TECHNIQUES

The structural system was key to the design’s choice for this project. It allowed the development of a terraced structure capped with a native double-pitched roof, instead of the tall, imposing, boxy solutions proposed by other teams.

Another key was the use of “super-trusses.” In normal building design, rooms requiring wider column spacing are typically stacked above rooms with tighter column spacing. For a convention center, that typically means locating the exhibition hall above meeting rooms and parking—essentially the design proposed by the other competing teams. This may be best for the structure, but an on-grade exhibition hall works best for facility operations and was accomplished through the development of a “super-truss” system. The Hawaii system consists of two-story-deep trusses at 90’ on center with perpendicular single-story-deep trusses suspended below at 55’ on center. This solution:

- Captures otherwise-unusable interstitial truss space for parking and meeting rooms;
- Reduces building height by 50’;
- Allows the exhibition hall to be located on the ground-floor;
- Provides ideal structural modules for each usage area, with 90’ x 118’ column-free spans in

the exhibition hall, 55' column-free parking bays, and 90' x 330' of column-free space for meeting areas.

A major architectural feature are 14 exposed steel tree columns rising up to 108' in the lobby area and echoing the mature palm trees planted alongside and soaring to a lobby "sky" of sails and light. The columns appear visually free-standing, yet support the tallest tension glass wall system in the Western Hemisphere. Columns were designed to allow pre-fabrication in large, easily erectable pieces. To preserve the architectural expression of the trees, they were filled with reinforced concrete. This step met fire code requirements without unsightly fireproofing and costly cladding. A king post truss system atop each tree serves triple duty - supporting the skylight roof, offering stability against the wind, and providing stiffness for the glass system. The lobby is light, dramatic, and open to the elements; in fact, the skylights and sails were oriented to minimize the amount of wind-driven rain brought in with the trade winds. The complex geometry of the roof sails was developed using soap film models.

FUTURE VERTICAL EXPANSION

The structure design not only provides optimum facility configuration, it also allows for easy and economical vertical expansion, stacking future areas on the existing top level. Expansion is accomplished by simply adding new two-story space-capturing trusses onto the existing two-story trusses. These super-economical add-on trusses will provide 100,000 additional square feet of interstitial meeting rooms with 100,000 additional square feet of exhibit space above.

Because the project was bid under a design/build contract, the contractor had to commit to costs with only competition drawings, leaving the design team to face the challenge of



meeting owner requirements while working within predetermined financial commitments on this \$200 million project.

Finally, the schedule allowed only 750 working days for design, permitting, and construction of a \$200 million, 1.1-million-square-foot facility.

Despite the tight schedule, the facility was turned over one month ahead of schedule and on-budget.

Project Team

Owner:
State of Hawaii

Structural Engineer:
Skilling Ward Magnusson
Barkshire Inc., Seattle,
Washington

Architect:
LMN Architects (Seattle),
in association with
Wimberly Allison Tong & Goo
(Honolulu)

General Contractor:
Nordic/PCL, A Joint
Venture, Bellevue,
Washington

Steel Fabricator:
Canron Construction Corp.
(Vancouver, BC and
Portland) and Herrick
Corporation (Pleasanton,
CA)



Nancy Lee and Perry R. Bass Performance Hall

Fort Worth, Texas

The Fort Worth cultural district is the third largest arts district in the nation. Acknowledged as one of the finest museum districts in the world, Fort Worth houses an elaborate collection of artwork in the internationally known Kimbell Art Museum, the Modern Art Museum of Fort Worth, the Fort Worth Museum of Science and History, and the Amon Carter Museum. At the same time, five world-famous performing arts organizations – the Fort Worth Symphony Orchestra, Fort Worth Dallas Ballet, Fort Worth Opera, Van Cliburn International Piano Competition and Concerts, and the Casa Manana Theater – also reside in Fort Worth. What the city lacked, however, was a first-rate performance hall – a home – for these organizations.

The future of the performing arts in Fort Worth had long been

a concern for area leaders. Conditions were so dire that the orchestra rehearsed at one facility and then traveled to another to perform. In 1992, Performing Arts Fort Worth, Inc. (PAFW) was formed to manage the design, construction and operation of a world-class multi-use performance hall in Fort Worth. PAFW hoped to build one facility that would meet the needs of each of the resident performing arts organizations and successfully host various traveling performances. PAFW established four goals for the performance hall: to provide a home for the major performing arts organizations of Fort Worth, to provide a world-class venue for touring artists and attractions, to serve as a catalyst in the economic revitalization of downtown Fort Worth, and to be a driving force in the integration of the performing arts into the curriculum of

the public schools.

PAFW retained David M. Schwarz/Architectural Services to design a performance hall that would meet each of these goals. The firm has built a strong reputation in the Dallas/Fort Worth community based on a variety of distinctive projects, including the Ballpark in Arlington; Cook-Fort Worth Children's Medical Center, and the mixed-use Sundance West apartment/entertainment complex in Fort Worth. The design-oriented firm believes that a healthy respect for the past is a key to understanding the present and helps define directions for the future. Schwarz's goal for Fort Worth's performance hall was to design a state-of-the-art, multi-use facility by returning to traditional design and planning concepts which have not been used for performing arts centers in decades.

Jurors' Comments

"The designers utilized several innovative solutions, such as the use of a torsion tube, to a very complex design problem."

"The use of 3'x3' box girders to provide unobstructed views along the horseshoe-shaped seating was an outstanding design concept."

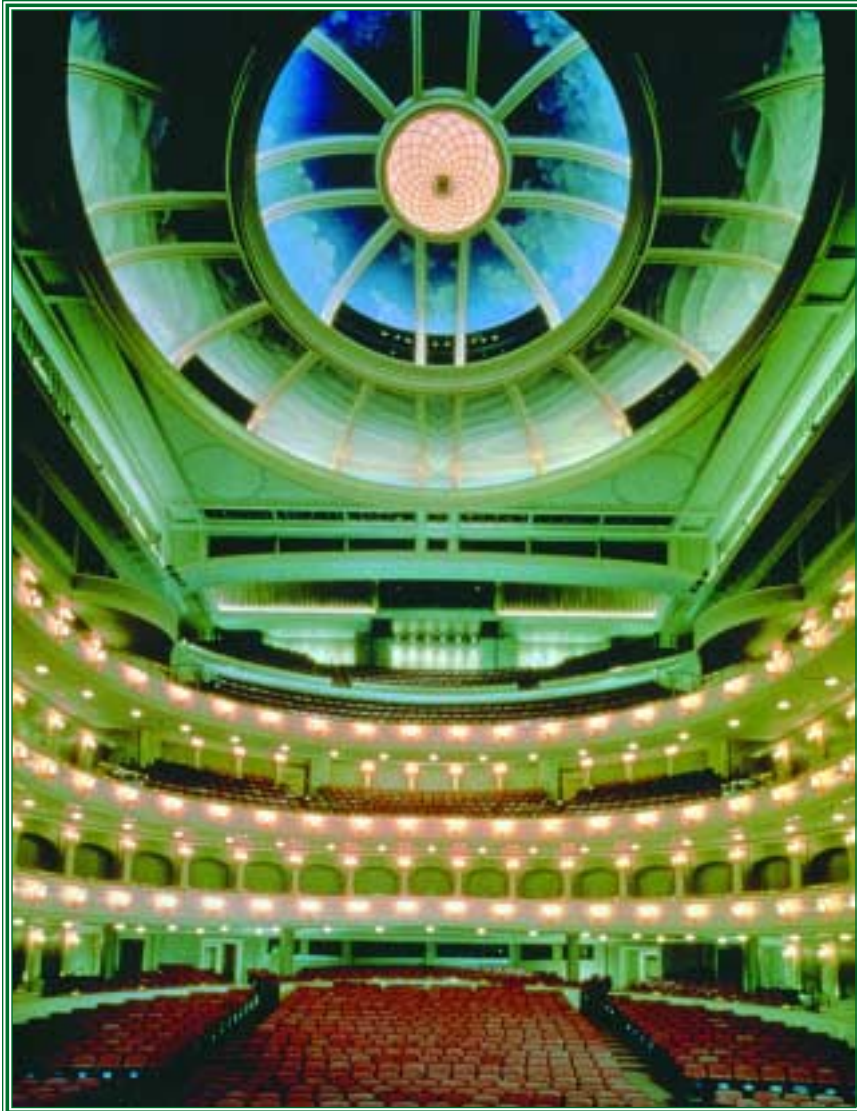
"The vibration analysis and design considerations to meet the building's acoustic needs were impressive."

The goals of both the owner and architect were met with the Nancy Lee and Perry R. Bass Performance Hall, a 2,056-seat multi-use performance theater located in Fort Worth's downtown Sundance Square. Opened in May 1998, the \$65 million Bass Hall was funded by private donations contributed by individuals, corporations, and foundations. Bass Hall has been described by the Toronto Star as "one of the great concert halls of this century." The structural engineer's use of structural steel as the primary construction material is a key to the project's success.

OWNER'S PROGRAM

Walter P. Moore and Associates, Inc., the structural engineering firm on the project, utilized a variety of engineering skills to design the world-class performance hall. In spite of the



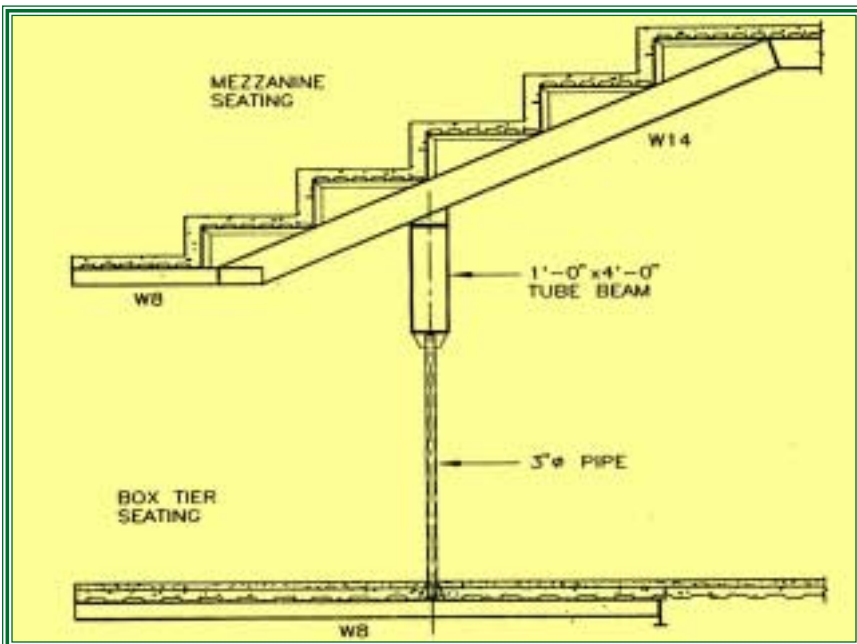


complex design challenges presented by the traditional 19th century opera house seating design, the project was completed within budget and in accordance with the owner's schedule requirements.

STEEL FRAMING SYSTEM

The first crucial engineering decision was the selection of an appropriate material for construction. Engineers compared cast-in-place concrete versus structural steel systems and performed various analysis of both systems. Although cast-in-place concrete offered some advantages, steel offered two primary strengths. First, steel framing eliminated the need for geometrically complex and expensive concrete formwork. The hall required more than 84,000 pieces of steel, an unusually high number that reflects the intricate complexity of the performance hall. Second, designing with steel allowed engineers to resolve any complications in the draft stage rather than in the field, which could have delayed the relatively fast moving, four-year project.

Careful engineering and collaboration assured that a steel frame supported the desired acoustically pure environment just as efficiently as a cast-in-place concrete frame. Working with the acoustician to ensure that the structural frame would minimize vibrations and noise generated by the audience, the engineers carefully analyzed the structural frame, focusing particularly on the seating cantilevers. Additional mass in the walls, floors and ceiling, as well as careful jointing throughout the structure, was specified to help maintain the sound environment.



STRUCTURAL ROOF FRAMING

Acoustical considerations also played a major role in the design of the structural roof framing. The thick plaster ceiling used to isolate the audience chamber acoustically required more struc-

tural support than conventional ceiling construction. Engineers developed structural roof framing that used a series of 12'-deep trusses spaced at approximately 40' on center. The framing spans the 92' width of the audience chamber within the available vertical plenum space. The basic roof construction consists of conventional metal deck on steel beams, but the acoustician required special provisions to create an adequate sound barrier: a 48" air space below the roof, enclosed by a 100 psf slab. This mass was achieved with a 9½"-thick slab consisting of 6 ½" of normal-weight concrete on a 3" metal deck. A level of composite steel beams located within the depth of the main roof trusses supports this slab.

Ductwork was placed immediately below the acoustical roof slab, within the remaining depth of the roof trusses. Because the ducts for incoming air supply took up all the available space in the ceiling plenum chamber, engineers routed the return air from the audience chamber through tunnels beneath the lowest seating level and into the mechanical rooms.

The ductwork also prevented ceiling and catwalk hangers from reaching the acoustical slab framing, so we added another level of framing at the roof truss bottom chord elevation. This framing consists only of a grid of beams, arranged in plan to support hangers from suspended catwalks and plaster ceilings below.

RIGGING SYSTEM

The design called for an extensive counterweight rigging system for the curtains, scenery, lighting and other staging components that the hall uses to accommodate various stage shows. Operators can deploy a secondary, movable ceiling to achieve the appropriate acoustical environment in the audience chamber. To accomplish this, our engineers specified a system of rigging lines spaced at 6" centers

over the full depth of the stage. The rigging loads are suspended from pulleys attached to steel roof beams that are spaced across the stage width. The pulleys direct the rigging lines horizontally to another large series of pulleys at one end of the stage. These master pulleys are supported by headblock steel beams, which span 60', unsupported, from the front to the rear of the stage. The rigging lines then turn down at the headblock beams to the counterweight zone.

Operators can add counterweights from upper and lower loading galleries as required to support the rigging loads. The loading generated by the turned rigging lines is significant - 2,200 pounds at 6" centers both horizontally and vertically. Heavy, W36 sections welded into a single headblock beam resist these design loadings.

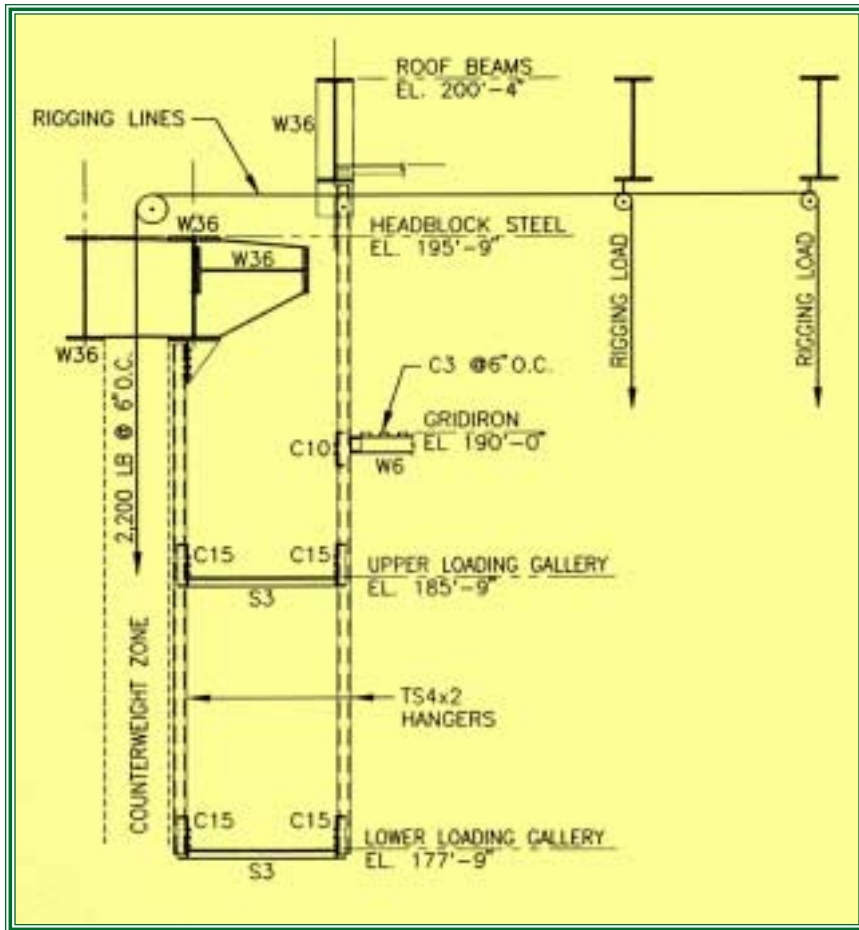
STRUCTURAL EFFICIENCY

The steel structural system proved to be highly efficient. During schematic design, the engineer estimated the weight of the structural frame at 1800 tons; the final tonnage was 1805 tons, a minuscule .28 percent over the initial estimate. As a result, cost "creep" from schematic design to construction was eliminated and the budget was achieved. Creativity, thoroughness and accuracy of structural documentation allowed the structural frame to be completed on schedule and within budget. The frame was topped out only ten months after the start of construction, and structural change orders amounted to less than one percent of the frame cost. The steel framing system also offered solutions for the challenges presented by the horseshoe seating design.

INNOVATIVE TECHNOLOGY

The success of public assembly facilities greatly depends on the seating design, which is complex because of the various seating levels and the demand for

unobstructed sight lines. Designed with a classic horseshoe shape in the style of a 19th century opera house, the audience chamber of Bass Hall provides five levels. Each of the five seating levels presented a different set of structural concerns. The second level (box tier) demanded the most creativity. The intimate box tier level features seating boxes that measure 8' by 10', each with a private entry from an anteroom off the main public corridor. Because the box tier level cantilevers over the orchestra level, engineers could not use supporting columns that would have obstructed the views of those in the orchestra level below. The horseshoe configuration also complicated the framing, limiting structural depths to eight inches if those seated below were to have unobstructed sight lines. The design team elected to frame the box tier level using a shallow and economical system of 8" deep, wide-flange steel beams. This system hangs from the mezzanine level above using 3" diameter pipe hangers, effectively eliminating all concerns about columns that might block the view. Of course, this moved much of the structural challenge to the mezzanine level above, demanding an innovative engineering answer. To position the seats for that level properly, the three lowest rows of the mezzanine level cantilever approximately 10'. Although the structural depth was not overly restricted, architectural considerations dictated that only two columns be used in the box tier back wall, severely restricting choices for support locations for the mezzanine above. The engineer designed a deep, curved steel tube beam to span between two columns hidden in the box tier back wall, at the rear of the parterre seating area below. Fabricated from A572 grade 50 steel plate, the 12" by 48" tube gracefully curves between the columns, following the geometry of the seating horseshoe. The



out at the mezzanine level to accommodate a public lobby below.

For more information on this project, please see the article "Steel Horseshoe" in the December issue of Modern Steel Construction.

Project Team

Owner:

Performing Arts Fort Worth

Structural Engineer:

Walter P. Moore and Associates, Inc., Irving, TX

Architect:

David M. Schwarz/
Architectural Services

General Contractor:

Linbeck Construction Co.,
Fort Worth, TX

Steel Fabricator:

Steel Service Corp.,
Jackson, MS

mezzanine level cantilevers directly over the top of the tube to create the desired vertical seating alignment without compromising the view from below.

The second level seating design demanded engineering creativity and prepared engineers for an even greater structural framing challenge: the upper and lower gallery seating levels.

The challenge of the upper and lower gallery seating levels was met with an innovative solution. Due to architectural constraints, the lower portion of the balcony seating, which cantilevers over the mezzanine seating, could not have a back span. The structural engineer developed a unique torque-tube concept to allow the structural framing to work within the architectural constraints.

At the lower gallery, the seating rows were intended to cantilever well into the main audi-

ence chamber, which is common for a performance center. However, at this level, the back span depth was limited to 8" by the ceiling heights in the public lobby behind and below the upper gallery. Since that wasn't enough to provide a conventional cantilevered solution, a torsionally stiff support beam offered the best answer.

Engineers designed a square beam with 36" sides to support the cantilevered loads in pure torque. Referred to as a torque tube, it curves to follow the horseshoe seating geometry, much like the tube beam at the mezzanine level below. Engineers specified A572 grade 50 plate for the tube. Two pairs of columns located at the back of the mezzanine seating resist the torsion accumulated in the torque tube. As if the lower gallery hadn't produced enough structural challenges, these column pairs had to be transferred



Sophia M. Sachs Butterfly House & Education Center

*Faust County Park,
Chesterfield, Missouri*

Sunlight is an essential source of energy on which butterflies rely for survival. So with this in mind, building transparency was the key design issue of the Butterfly House and Education Center in Faust County Park in St. Louis. The \$3.4 million new construction consists of an exhibit hall, a gift shop, a theater, and an

8,000-sq.-ft. glass conservatory.

The conservatory houses 60 exotic species of living butterflies imported from four tropical countries. To compliment the home for the tropical butterflies, brightly-colored plants and flowers were transplanted from the southern Florida. A tropical ecosystem was called for with constant air temperature of 82

Jurors' Comments

"An impressive and innovative structural design that maximized structural efficiency. It was a simple solution that saved significant costs."

"This design solved owner requirements with design techniques that lowered costs dramatically."

"Building aesthetics were critically important and this efficient structural system met that need."

degrees, 74% humidity, and sufficient sun light. The requirements for constant air temperature and humidity were met by using underground HVAC system and laminated insulating glass panels. However, the need for natural sunlight demanded a creative structural system with minimum shadow-casting effect so as to collect as much solar energy as possible to maintain plant growth for the butterflies to thrive.

A five-span vaulted skylight structure was conceived for the conservatory to evoke and resemble the curved shape of butterfly wings. The skylight system covers a 107' by 72' footprint with a 37' high center vault. Four steel truss frames are the main gravity load-carrying elements supporting five vaults of aluminum skylight rafters. The narrow curved rafters serve as secondary load-carrying members, as well as glass mullions in order to minimize shadow-casting effects. Vertical truss columns have rigid



connections with horizontal truss girders to form moment frames. This provides the conservatory with a lateral force resisting system in the truss frame direction. The combination of lateral bracing for the truss chords, unbalanced horizontal forces from the skylights due to gravity loads, and the wind and seismic forces called for a special structural system in the direction perpendicular to the trusses. The conventional solution was a compression bracing system with curved steel trusses parallel with vaulted skylights. However, the compression thrust caused lateral stability problems for curved truss members. The connections between curved and horizontal trusses became extremely difficult due to stringent requirements for the butterflies to have natural flight paths.

A multi-span tension rod tie-down system with elastic supports was proposed for the required lateral bracing system. The new system replaced conventional curved trusses with 1½" diameter tension rods and reduced bracing steel from 27 tons to only 2 tons. All curved steel members and their expensive fabricating process were completely eliminated. The new design saved construction cost of \$11.50 per sq. ft. Since the tie-rods provided compression thrust, the aluminum-arched rafters were able to be designed for minimum sections and were able to span up to 30'. A bright open space was created and the project benefited from simpler construction with a shorter schedule. Truss columns were designed to have certain fixity at base so that progressive collapse is minimized should tension rods suffer severe damage for any reason. This design consideration also eliminated temporary bracing for the truss frames during erection.

Tension rods are large deformation members. Conventional linear elastic analysis does not apply to the structural analysis. A special computer program was

developed to model the true curved rod length at each loading stage. The challenge the structural engineer faced was that the tension rod tie-down system is not as stable as tension rod suspension system as used in cable-stayed bridge. Due to the unique geometric setting, many factors tended to reduce the rod tightness. Among these factors were rod elastic elongation due to axial load increase and rod relaxation due to steel support vertical deflections. Extreme care was used to make this delicate structure strong and stable by optimizing and balancing the stiffness between tension rods and supporting steel frames. ASTM A449, 90-ksi high strength steel was specified for the rods so that inelastic deformation will not occur even under factored load condition. The design also ensured that steel frame deflections were within strict skylight tolerances. ASTM A519, 90-ksi high strength steel pipe was used for turnbuckles to keep the size to a minimum and to provide a clean look. Rod installation and setup were assisted with a torque wrench calibrated with strain gauges.

In order to reduce field welds, truss members were fabricated in the shop to a maximum extent. A special retractable backing tube detail was developed for full penetration welds at main truss chords. All welds were ground smooth after welding and received special inspections which included ultrasonic and magnetic field tests. The structure and connection details were designed simple, practicable and thoughtful, construction process was smooth and straightforward. No change orders were issued for the structure.

This new facility, which opened on September 18, 1998, presents an unparalleled opportunity to foster a better understanding of butterflies, to increase public awareness of our natural world, and to provide family recreation. The creative

structural design made the Butterfly House simultaneously strong and elegant. Coupled with excellent architectural, landscaping design and quality field execution, it placed the Butterfly House among the most notable landmarks in the St. Louis area.

Project Team

Owner:

The Butterfly House
St. Louis

Structural Engineer:

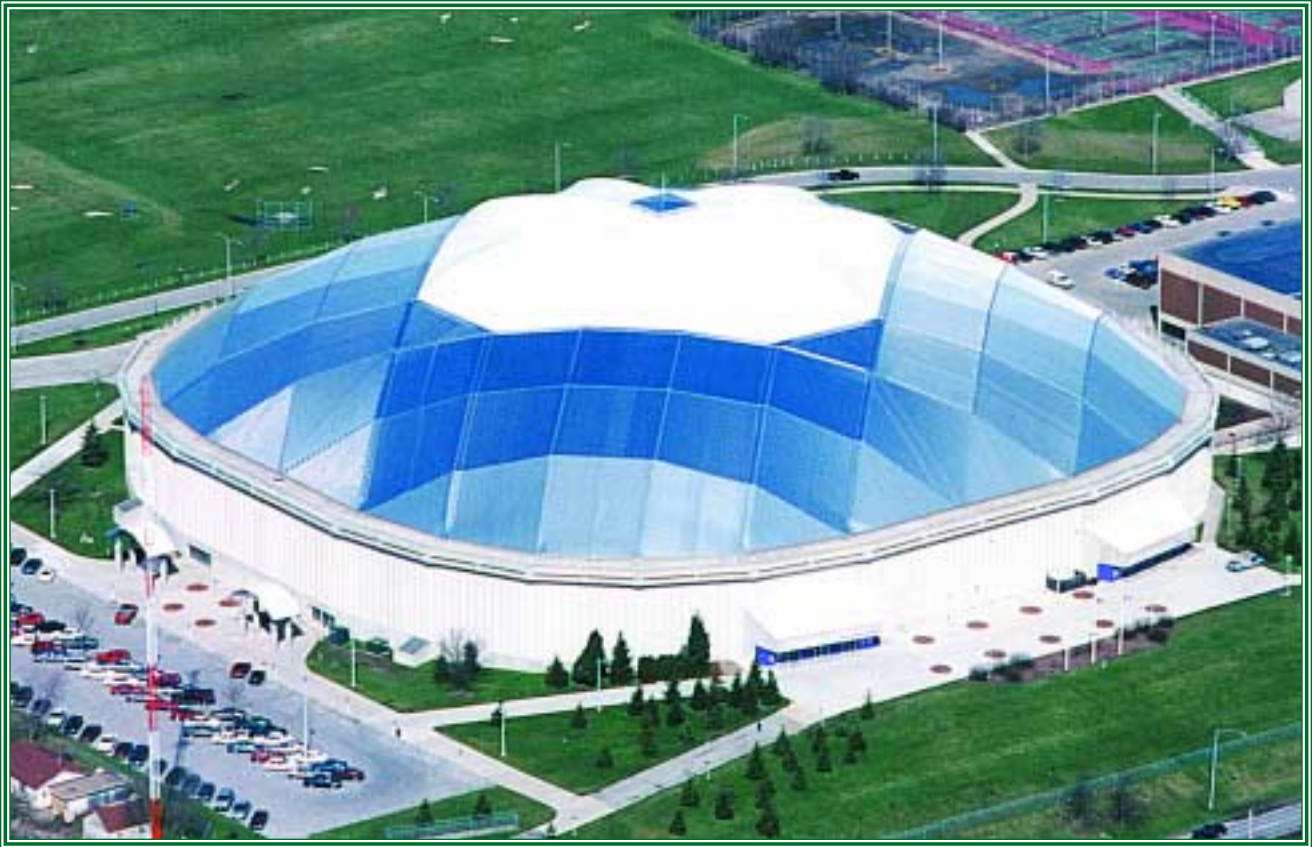
Ibrahim Engineering Corp.
St. Louis

Architect:

Christner Inc.,
St. Louis

General Contractor:

BSI Constructors Inc., St.
Louis

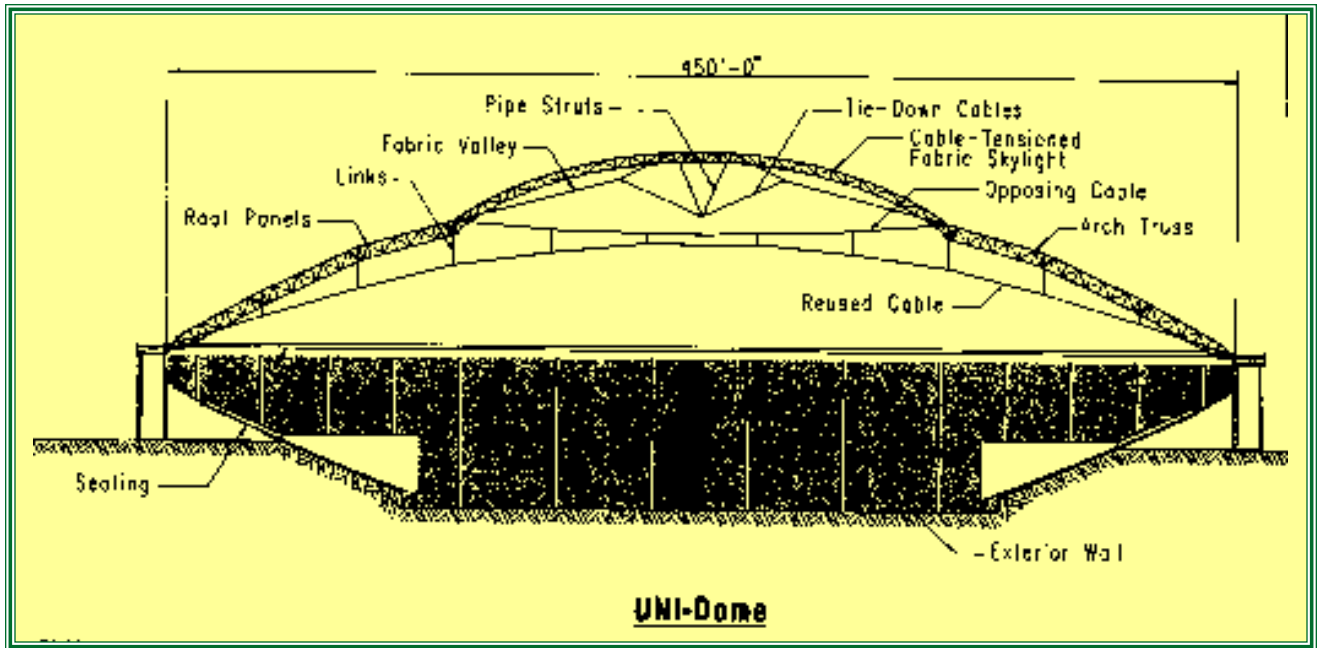


University of Northern Iowa Air Dome Retrofit

Cedar Falls, Iowa

The UNI-Dome at the University of Northern Iowa, completed in 1975, was the first indoor stadium with a full-size, air-supported fabric roof system in the US. Using concepts developed by the late David Geiger of Geiger Berger Associates, PC, this technology drastically reduced roof construction costs and, in some cases, reduced the construction schedule. Not incidentally, the infusion of natural light through

the translucent canopy overhead further adds an extremely appealing architectural and aesthetic element, creating a sense of light and space not present in a conventional, rigid-roofed structure. After the completion of the UNI-Dome, air-supported roof structures began to replace conventional, rigid roof structures entirely for stadium size covers, and were subsequently constructed for eight other indoor stadiums.



Despite the advantages, there will still be some drawbacks. Snow removal is a major problem for air-supported domes. The stability of the roof depends on the maintenance of an interior pressure larger than the exterior load. In the original UNI-Dome roof, the 450'-diameter dome was kept inflated by two 125-hp fans operating 24 hours a day to create air pressure of 4.5 lb. per sq. ft. (the interior design pressure is limited to not more than 5 lb. per sq. ft.). However, in northern Iowa, snow loads may be as high as 40 lb. per sq. ft. Snow can be melted by hot air directed to the roof surface, but with a heavy snowfall this may be insufficient to reduce the exterior load. In the winter of 1994, the UNI-Dome, like other facilities in colder climates, resorted to manual snow removal to prevent deflation. This led to a rip in the fabric and deflation of the roof. As it was approaching the end of its life span, the university decided to replace the original roof, rather than repair it.

The replacement of an existing air-supported dome roof with an aesthetically appealing, cost-effective, low-maintenance alternative is a sizable challenge. The University of Northern Iowa achieved these goals with the

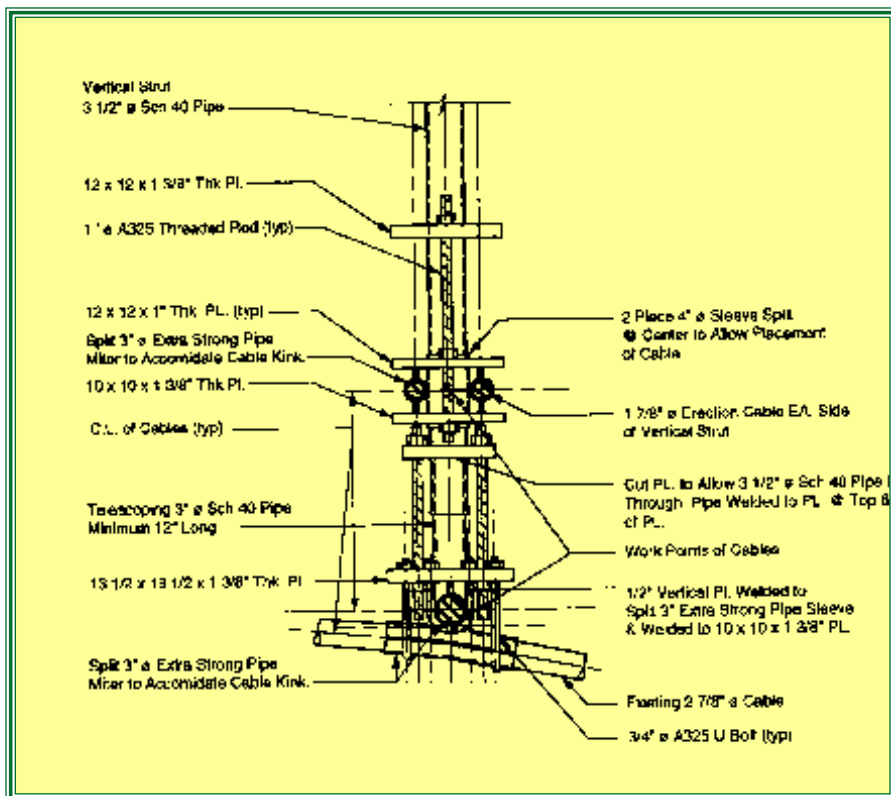


Jurors' Comments

"A very innovative solution to a complex problem."

"The solution of adding a new stadium roof with minimal changes to the existing building is a concept that can be used for other similar buildings."

"Retrofitting without a major interruption to the existing structure simplified construction and proved cost effective."



assistance of Light Structures Design Consultants of White Plains, NY (a subsidiary of DeNardis Associates). For the past 5 years, LSDC has developed alternative designs for the replacement of air-supported domes at several facilities, including Syracuse University and the University of South Dakota-Vermillion. For the UNI-Dome replacement, LSDC developed a hybrid cable-arch scheme, the first of its kind, which offers both functionality and aesthetic value.

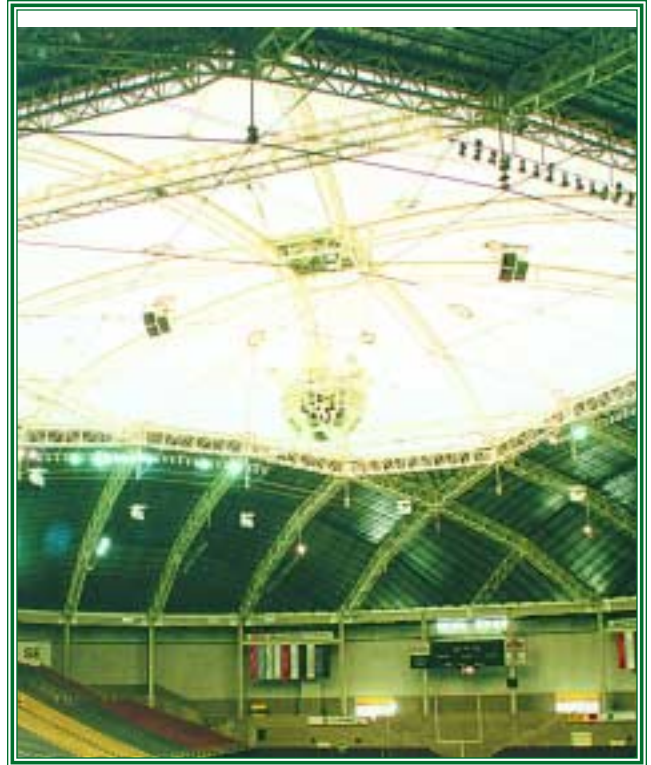
The hybrid design utilizes the ingenuity of the existing roof geometry and maximizes use of the existing structural components (the cable net, columns, and a reinforced concrete circumferential girder). The structure's periphery was prestressed with a post-tensioning system of tendons, converting the existing concrete compression ring into a tension ring. The existing cable net, connected and stressed against the arches, gives the arches stability and allows them to be slender and lightweight. All structural components were shop fabricated in segments and field bolted. The translucent center skylight, 45,000 sq. ft in area, is enclosed with an arch supported (PTFE) fabric tensile roof. To optimize energy efficiency and lower heating costs, stain-

less steel, and standing-seam insulated roof panels on metal deck cover 75% of the roof.

The main support for the replacement roof is a 6'-deep, 4'-wide steel box-truss arch system. There are four main cross arches, 2" in each direction, each 400'-long and 220' apart. Between these are sixteen secondary arches (four per side), each 107'-long, spaced every 44', which span from the structure's perimeter to the middle third of the main arches. The center skylight is a cable-supported irregular polygon, with a crown supported by pipe struts and cables. The reused cables (twelve 2 7/8"-diameter cables) are linked to the arches or cables above by rigid vertical members. In this hybrid roof design, the under-slung linked secondary cable system is located below and along the plan centerline of the arches. In addition to resisting uplift, this causes the crossed arch system to act as a prestressed shell. The cable system effectively diffuses the loads applied to the arches, creating a structure that can be considered fully utilized in terms of strength.

In the U.S., there are now 20 air-supported sports facilities in structural distress. Roof replacement or even construction of new facilities using a hybrid cable-arch design offers unique possibilities. As a symbiosis of conventional roof technologies and contemporary lightweight, long span technologies, it unites the better of these two schools. In particular, the "skylight" section is both architecturally appealing and cost-effective. While replacement of the UNI-Dome roof utilized the existing cable geometry, design of a new structure allows the design of cable geometry for a specific application. This could allow both the magnitude and spatial distribution of the pre-stress to vary. It would then be possible to use an initial state of flexural pre-stress on the arches rather than simple axial load as was done for the

UNI-Dome. It would also be possible to investigate the effect of cables going slack in conditions of extreme loading, giving a structure whose behavior would change in response to the type and magnitude of load.



Project Team

Owner:

University of Northern Iowa,
Cedar Falls, IA

Structural Engineer and
Architect:

Light Structures Design
Consultants (a division of
De Nardis Associates, Inc.),
White Plains, NY

General Contractor:

Penn Co. Construction, Inc.,
Eagan, MN