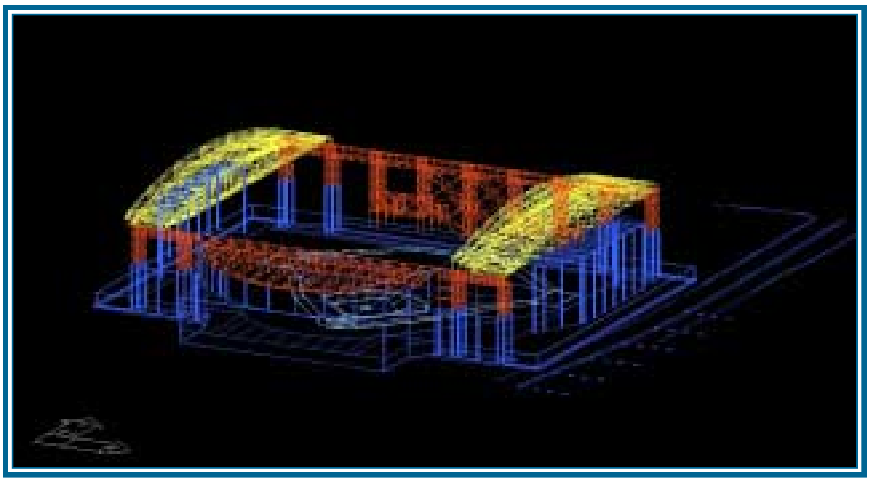
MOVABLE ROOF

The design for the Bank One Ballpark in Phoenix incorporates a movable roof that allows for natural grass while still maintaining stadium comfort

Photo by J. C. Prator Jr.



By Stanley Welton, P.E., S.E., and Brad Lueger, P.Eng.

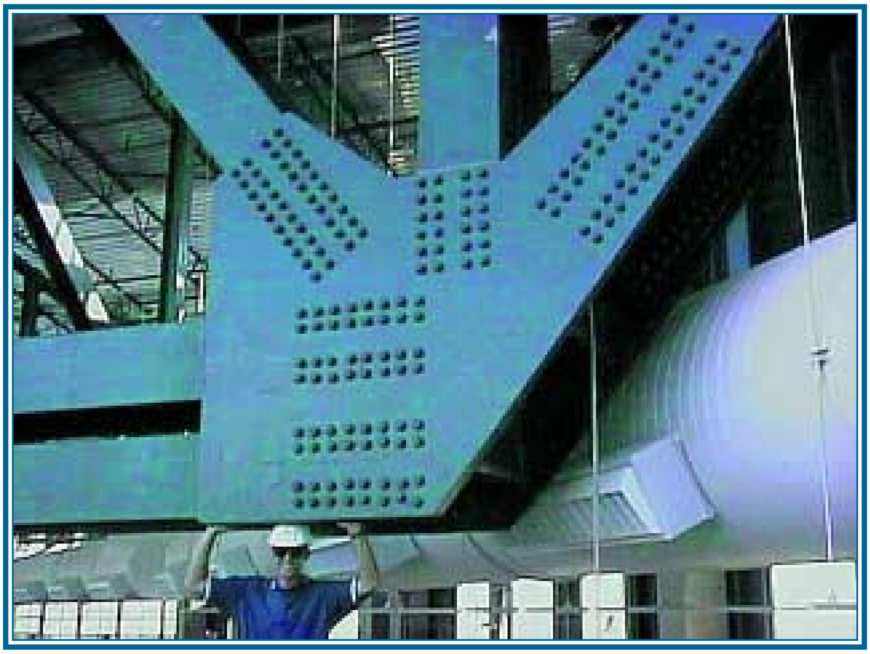
THE DESIGN OF A STADIUM BEGINS WITH AN UNDERSTANDING of the owners' goals. For a new stadium for a new baseball team in Phoenix, those goals included a stadium with natural grass, air conditioning and a moving roof—a design never before accomplished.

In Phoenix—where daytime temperatures during the baseball season regularly exceed 105 degrees F—air conditioning is a must to attract fans. Combine this with the teams' desire for a natural grass field and an operable stadium roof was a logical choice. The roof can be opened each day to allow sun to reach the field, and then closed to air condition the space prior to the start of a game.

In 1994 a design team, headed up by architect Ellerbe-Beckett with Martin/Martin as the structural engineer, was selected for the new Bank One Ballpark. During the design competition, it was established that the ballpark must be completed in time for the 1998 season, which therefore required a fast-track schedule. The roof, which can be opened and closed in a manner similar to that of a camera aperture, is clearly the most interesting—and most complicated—part of the project, both in design and erection. Due to the tight schedule, the roof erection needed to proceed at the same time as the construction of the lower concrete structure. This often produced conflicts and interferences between the concrete and steel trades. The sun also proved to be a problem for the erector during the erection of the movable roof panels. The steel expanded and contracted several inches over the course of the day, making it difficult to control erection tolerances.

The moving system is a combination of existing technologies used in a unique situation. The design made use of crane technology using rails, cables and sheaves. It was the intent of the designer to make a maintenance free, simple-to-use system that could be controlled by a single person. For this to happen, a lot of debugging and fine tuning occurred during the erection and commissioning process. This process, while standard for machine work, is unusual in building construction and steel erection. The process took several months and culminated in requiring as many as 30 people to coordinate and observe the first few roof moves.

The limited space available on the site, combined with the angle of the sun, required the roof to have a low profile and to stack within the limits of the seating bowl. To accomplish this, a roof system was selected that would open symmetrically about center field and stack over the left and right field seating bowls. Due to area constraints on site, the distance the roof could move off the field was limited. To maximize the amount of sunshine to reach the grass, the roof was reduced to the lowest possible height—205' of clear space directly above home plate. Each moving panel has a down-turned truss along its leading edge and is supported by the panel below along its trailing edge. This concept allows for both a low profile roof and short travel distance. At approximately 60' per minute for the fastest panel, the closing or opening time is approximately 4½ minutes. This compares favorably to 20 minutes for the Toronto Skydome and other stadiums currently under design



and construction.

Because the panels telescope while moving, the number of design load cases was very high. Since each panel is supported by the panel below, as the panels move the reactions from one panel applied to another continuously change location. The load magnitude also changes. This combination made it difficult to determine the controlling load combination and location for each member design. Different positions controlled the design of the trusses as well as the upper bogie beams and supporting structures. Over 100 load cases and load combinations were used in the analysis process to determine the controlling member forces.

A major design consideration in long span trusses is chord bracing. This becomes even more critical when it involves moving trusses. In addition to bracing forces, horizontal forces due to accelerations and decelerations from starting and stopping also need to be considered. In most long-span roofs, it is normal to use bracing trusses perpendicular to the main truss to provide stability and redundancy. With tight stacking distance (9' center-to-center spacing of the trusses) the use of perpendicular bracing trusses was not an option. In lieu of this, careful bracing analyses were made during design to study local and global truss member buckling issues.

Roof System

The roof consists of eight panels, four on each side of the field centerline. With three moving panels stacked on top of one fixed panel, each half of the roof operates independently of the other and closes in a telescoping manner. The speed that each panel moves is dependent on the distance it must travel since all of the moving panels start and stop at the same time.

The fixed panel has two trusses spanning 517' with a depth of 51'-8". The inner truss (field side) supports 35,000 sq. ft. of roof in the closed position and contains approximately 530 kips of rolled shapes (W14x30 through W14x428 sizes) fabricated from 50 ksi steel. Due to availability and accelerated schedule, 65 ksi steel was not considered for the steel in the first bid package, which contained the fixed panels.

The outer truss supports 100,000 sq. ft. of roof in the closed (stacked) position and has approximately 800 kips of rolled shapes (W14x30 through W14x605) fabricated from 50 ksi steel. In the stacked position, it supports half of each of the moving panels stacked above it in addition to half of the fixed panel.

The trusses for the fixed panels are bow trusses tied together with horizontal diaphragm bracing in the plane of the top and bottom chords. The fixed panels, trusses and support towers were designed to be stable prior to the completion of the remainder of the roof and support structure and also to be stable during the jacking process used to erect the fixed panels.

The roof deck is 3” Type N acoustical deck supported on 32” LH bar joists. Bar joists were used due to the long spans and the light weight-to-span ratio. They do, however, make the panel diaphragm bracing more difficult. WT shapes were used for the diaphragm bracing by stripping the tee stem where they crossed over the joist top chord. The deck was used as a diaphragm to provide local bracing for the joists but was not relied upon as a diaphragm for the entire panel or for bracing of the truss compression chords.

Two months into the construction document phase, the owner and construction manager revised the design schedule to further accelerate the roof design since the fixed roof panels needed to be constructed ahead of the concrete bowl construction. The drawings for the fixed panels and supporting towers were issued three-and-one-half months ahead of the original schedule and before the moving panels’ analysis and final loads from above were completed.

The moving roof panels and the rest of the steel structure was bid separately from the fixed roof. However, the same fabricator/erector, AISC-member Schuff Steel, won both packages. Engineer for the moving system was Hatch Associates, Mississauga, Ontario and onstruction manager was the Phoenix office of Huber, Hunt & Nichols, Inc. Among the several detailers on the project were AISC members B.D.S. Steel Detailers and Schuff Steel.

Trusses spanning 517’ along the leading edge (field side) and 13 bogies along the trailing edge support the moving panels. The moving trusses support approximately 50,000 sq. ft. of roof in the closed position and have 645 kips of rolled shapes (W14x30 to W14x426 sizes) fabricated from 50 ksi and 65 ksi steel. The trusses are bow shaped with a double trussed top chord. The trussed top chord allowed the truss panel points to be spaced at 90’ centers and thus provide a lighter looking truss. It also minimized the number of field connections and shored support points, reducing erection time and cost.

To reduce member weight and simplify the connection design, the WF top chords were oriented with the webs horizontally, which allowed for a more efficient use of the member’s unbraced length during design. The KL/r ratio for both x and y plane were approximately equal, allowing for a greater spacing between bracing points.

In order for the panels to nest tightly together on top of each other in the open position, bottom chord bracing was not achievable. To provide lateral support for the truss web compression members and to resist horizontal accelerations/decelerations from seismic forces and emergency stopping forces, the bottom chord was horizontally trussed. The bottom chord has a 6’ stance for the 517’ span. In addition, the vertical web members were fixed at the top to the W40 upper bogie support beams. The bottom chord truss as well as the truss vertical web members were designed to resist bracing forces as well as meet minimum bracing stiffness criteria.

Rails mounted on the panel below support the 13 bogies along the trailing edge. The rails are elevated above the roofing to allow clearance for roof flashing and unobstructed flow of water to the gutters. Elevating the rails posed a unique design consideration. While most rails in use have continuous support along their length, these rails were supported on posts spaced at 4’ to 6’ on center. Due to the high bending stress and unique support conditions, ASTM A572-Gr. 50 steel was used for the rail material. This material is more ductile than normal hardened rail material, but still provided the hardness properties required.

The rails and rail support posts are supported on W40 Gr. 65 steel beams (upper bogies support beams). These beams have spans up to 88’ and are used to support the vertical loads from the panels above, as well as the horizontal component of the bogie force due to differential movement between the panels. These horizontal components are generated when the upper bogie pivot arms lean due to the differential panel movement. Leans of up to 6 degrees are expected in the upper bogies, which produce horizontal loads up to 10% of the vertical reaction.

To reduce the building height and maximize the sun angle to the field, the panels were stacked vertically as tightly as possible. Approximately 12” is all that separates the bottom of the W40 upper bogie support beam for the panel above and the top of the rail on the panel below. To allow for differential movement between the panels and reduce the horizontal forces, the upper bogie pivot arms needed to be as long as possible and articulated (hinged top and bottom). They also provide for greater flexibility in the construction tolerance by reducing the differential movement from horizontal forces, which allows for increased tolerance for rail locations.

The trusses for the moving panels are supported by the lower bogies. In addition to providing gravity support, these bogies along with the cable provide the drive forces as well as the braking forces for the moving panels. The bogies are attached via cables to a winch and motor system located under the bottom chord of the fixed panels. Holes are provided through the lower bogies for the cables to pass. This is a result of all the bogies being in line with each other and using the same rails. There is one winch platform for each side of the building, suspended at approximately mid-span of the fixed panels.

Because the panels stack and move relative to each other, providing lateral bracing was difficult. The lower bogies are used to provide the lateral support for the panels. In the east-west direction, lateral loads are resisted by the drive cables attached to the north and south lower bogies. In the north-south direction, resistance to lateral loads is provided by horizontal guide wheels in the north lower bogie along the leading edge of the panels and the horizontal guide roller at the panel mid-span along the trailing edge. The south lower bogie is articulated to allow for free thermal and gravity load expansion and contraction of the truss.

The lower bogies travel on rails placed on a trolley box along the north and south edges of the field. The north end is supported on a colonnade while the south rails are supported on a 422' header truss that spans over the main seating bowl behind home plate. The trolley box is located 170' above the playing field.

The north side trolley box is on a trussed colonnade. This colonnade cantilevers from the ground and provides most of the lateral support for the roof. In addition to the colonnade, four towers located in the four corners of the building also provide the lateral and gravity support for the roof.

To reduce the span of the moving panels and trusses, a header truss was provided that supports the south track of the moving trusses. This truss, referred to on site as "The Mother of All Trusses," is located behind home plate and provides unobstructed view for fans seated behind home plate. It has a 422' span and weighs approximately 1,200 tons. Due to height limitations, the truss is 38' in depth from center line of the top chord to center line of the bottom chord. The top chord elevation was kept as low as possible to reduce shading from the sun from the south, while the bottom chord was raised so as not to block the view of the field form the upper deck section located behind home plate.

The top chord is a built-up box section (trolley box) to provide a runway for the moving roof rails. It is 5' deep and 4'-6" wide and was fabricated from plates up to 4" in thickness. The box was shop welded and spliced in the field with finished end bearings and welded lap plates. Welded lap plates were used for the top chord splice so all of the connections could be made from the exterior of the trolley box.

The bottom chords are double W36 Gr. 50 and 65 steel. The largest rolled shape used was a W36x798. Two W36x798s make up the center section bottom chord. The maximum length of these members was 39', which was limited by the maximum ingot size. These members are oriented with the webs horizontal to allow bolted side plates to be used at the connections. Because the double chords limit connection access for bolting, access holes were provided through the webs. The connections were bolted using lap plates and 1-1/8" diameter A490 X-bolts.

Erection

Steel erection for the Bank One Ballpark began in early 1996 and continued through 1997. The first steel bid package included construction of four, 170'-high towers and two roof panels (east and west fixed panel) with heights of 170' to 230' above the playing field. The design and construction of these fixed sections were accelerated four months so the erection could take place ahead of the concrete bowl construction.

The east and west roof panels are the fixed sections that support the moving panels. These panels, consisting of two trusses, top and bottom chord diaphragm bracing, steel joist and eck, were designed to be erected on the ground and lifted into place or erected in the air on shoring towers. Schuff Steel chose to erect the panels on the ground, then lift them into place using an innovative jacking system mounted on the towers.

Because the trusses are supported by the towers, they were too long to be fully fabricated on the ground with the towers in place. Instead, a section of each truss was left off until the panel was lifted clear of the towers. The panel was suspended by the jacking system until the end sections could be attached and the panel lowered onto the towers.

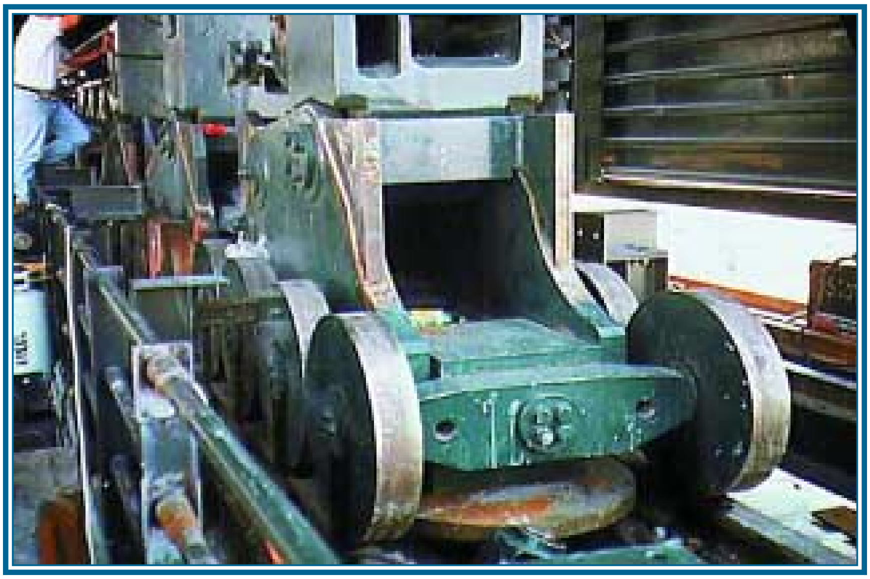
Four Elgood-Mayo 400-ton hydraulic cable jacks were employed to lift the panels. One was placed at each end of each truss. A laser controlled system was used with the jacks to ensure the panel was kept level and to prevent over-loading any of the jacks.

Each of the two panels lifted weighed approximately 1,200 tons and measured 477' long, 90' wide and 52' tall at the highest point. Because the panel geometry and weight distribution was not symmetrical, two of the jacks supported approximately 60% of the load.

The jacking process for each panel took several days. Due to the size, complexity and concern for safety, extreme caution was exhibited during the lift. Initially, the panels were raised and lowered from 1' to 6' approximately five times to verify the behavior during the lift. Only after the performance of the lift was verified were the panels slowly lifted into place. The total lifting time to raise the panels was approximately 16 hours. The jacks could make a 6½' lift per stroke. At the end of each stroke, cable wedges were used to hold the panel while the jacks reset. Each stroke took approximately 45 minutes.

Once the panels were lifted, the end sections were attached and the panels were lowered onto the supporting towers. Because the length of the fixed panels and the distance the towers are apart were critical to the future moving panels, the bearing locations were set with as much precision as possible when handling a 517' truss. First one end of the panel was lowered and connected to the supporting tower, and then the other end was lowered and jacks were used to spread the towers apart.

Spreading the towers was required for two reasons. The first was due to lateral deflections in the towers caused by lifting the panel. The unsymmetrical load on the towers caused them to move together approximately 2". The second was due to thermal expansion of the steel. The design distance between bearing points was based upon 85 degree F. The steel temperature at the time the connections were made was above the design temperature; getting the towers the correct distance apart was important for proper location of the moving truss work points and rails.



Geometry change of the trusses under their self-weight was accounted for during erection by specifying member lengths and truss cambers. The half-open position was used during design to determine the camber corrections. This position was chosen to minimize the distance any member location would be from its horizontal and vertical position throughout its moving positions. Truss 1, for example, moves vertically plus/minus 8" from its neutral position.

The moving roof panel erection also was unique. Each panel was built upon 15 rolling bogies. The upper 13 bogies sit on rails mounted to the panel below it, while the lower two bogies sit on rails mounted on the trolley boxes. This posed several problems for the erector: Imagine building a 55,000-sq.-ft. roof on 15 roller skates! Without fixed support points, stability and member placement becomes a critical erection concern.

The erector chose to build the panels in the approximately open (stacked) position. It was decided to build all three panels in the same location and then roll the panels to the fully open position. This allowed one set of five shoring towers to be used for all three of the moving panels. This decision allowed the construction of the bowl to proceed with minimal impact from the shoring towers and steel erection.

Due to the L-shaped configuration made by the panels and trusses, only the trusses needed to be shored. The trailing edge of the panel was supported upon the panel below; this also allowed access to the majority of the panel being built relatively easy. Each panel, though over 200' in the air, had a natural working platform for access approximately 6' below.

Temperature movement proved to be one of the most difficult problems to overcome. Martin/Martin worked closely with Schuff Steel to develop a procedure that would allow for free thermal expansion/contraction while providing the necessary horizontal and vertical stability during the erection. Movements of more than 2" were experienced during the day, which made squaring of the panels and erection tolerances difficult.

After several trials, however, a system was developed. Once the moving panels were completed and moved into the stacked position, they were secured by temporary bracing attached to the panels below. At this time the panels were surveyed to determine if any corrections were needed prior to moving the panels for the first time. Likewise, during the first move, additional surveys were taken to determine if any other structural corrections were required.

While the moving roof posed unique erection problems, the trusses—though of a more typical type of construction—also presented challenges. In addition to limited site access, the contractor had to deal with very large members, such as the 422'-long, 1,200-ton south truss. The truss was erected on shoring towers with the largest crane pick being 230 tons. The truss was shop assembled then reassembled on the ground, in sections, as close to the final position as site access allowed. The sections were then lifted into place and spliced. The bottom chord had four field splices that required up to 768 $1\frac{1}{8}$"-diameter A490 X-bolts each. In all, up to 40 tons of bolts were used in the construction of this truss. A 450-ton American 11320 crane was used in the erection.

This article was adapted from a paper presented at the 1998 National Steel Construction Conference by Stanley L. Welton, P.E., S.E. (shown above) and Brad Lueger, P.Eng.

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