Special Delivery

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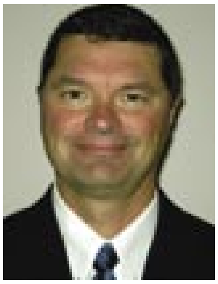
Not even a violent wind storm could stop the progress of steel erection for Memphis's FedExForum.



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It stands to reason that a building with "FedEx" in its name would be delivered on time. Despite the challenge of a major wind storm during steel erection, Memphis, Tenn.'s FedExForum opened in fall of 2004 on schedule and within budget.

The 800,000-sq.-ft multipurpose arena is the new home of the NBA's Memphis Grizzlies and the NCAA's Memphis University Tigers, as well as hockey games, concerts, and other events. Along with the arena, the complex also includes a practice basketball court, a four-story administration building that will house the Memphis Rock 'n' Soul Museum, a 1,500-space parking garage, a team store, and a sports bar.

At $250 million, FedExForum is the largest public works project in Memphis history. A unique combination of structural steel and reinforced concrete was used to optimize the structural design for economy and the project's fast-track schedule. Reinforced concrete construction was used for the foundation walls and lower seating bowl up to the balcony (upper concourse) level. For its numerous advantages in economy and speed of erection, structural steel was chosen for the upper seating bowl framing, the main entry lobby, the administration building, and for the key elements of the arena structure, the seismic resisting system, and the long span roof structure.

Roof Structure Concept

Circular plans are not the norm for arena roofs, which are usually oval or pill shaped in plan. For FedExForum's roof, a circular plan was architecturally driven by the form of the round seating bowl. As with the lateral system, this 436′-diameter circular plan was an opportunity to investigate a variety of framing systems, including radial systems. Eliminated were two early architectural exterior concepts for the building; one with an exterior exposed arching box truss spanning down the center of the arena and landing on "super towers" at each end, and the other with a fairly flat roof.

The box truss/super tower concept was ruled out after a preliminary analysis due to the relatively heavy weight of the structure and coordination problems with the location of the super tower columns and bracing.

For the shallow sloped option, lenticular shaped radial trusses, approximately 40′-deep at the center with distributed bracing rings, were laid out to create what was called the "bicycle wheel" concept. The design featured a hub at the center with radial trusses in tension at the bottom and compression at the top—basically horizontal beam-type behavior, without much contribution from the distributed bracing rings. However, the weight and detailing complexity of the bicycle wheel solution made it impractical.


The steel erection process was analyzed to account for construction and environmental loads for all phases of steel erection.

A dome structure seemed to be an obvious way to make the structure lighter in weight. A spherical dome with arched, radial shallow truss ribs with distributed tension/compression rings at the top and bottom chord levels was then investigated. Truss depths were reduced to a constant 12′, which allowed sections of the trusses to be shop-assembled, welded, and shipped to the job site. The resulting weight was 20% to 25% less than the other systems, which provided a significant material savings and the advantage of quicker field assembly with 40′ shop-assembled truss sections.

The final system includes 36 radial trusses: 12 "A" main trusses that span from the perimeter to the center hub and 24 "B" secondary trusses that span from the perimeter to a header truss approximately 2/3 up the slope. Top chords of the A trusses varied from W14x132 up to W14x233, while the bottom chords varied from W14x61 up to W14x109. The B trusses feature top and bottom chords that vary in size from W14x43 up to W14x99. This configuration saved weight and reduced congestion at the hub by maintaining more uniform spacing of the radial trusses. A 30′-diameter compression truss at the center is the connection hub for the "A" trusses. The combination of top and bottom chord perimeter rings and four intermediate rings of "sway" trusses, spaced at approximately 40′, serve as distributed tension and compression rings for the dome in the top and bottom chord plane.

Roof Loading

In addition to the code-required roof live, wind, and snow loads, there are numerous special loading conditions for the arena roof structure:

- Event rigging loads up to 150 kips applied in a multitude of different configurations for end stage or center stage events can be accommodated. Rigging loads are allowed to be applied at truss bottom chords and infill steel, including a grillage of rigging beams spaced at 15′ to 20′ on center above the event floor in the plane of the truss bottom chords.
- A 60,000 lb center-hung scoreboard, complete with large state-of-the-art video displays, is hung from a steel-framed hoisting platform hung below the center hub. This platform also supports approximately 40,000 lb of hoisting equipment for the scoreboard and the four major speaker clusters that surround the scoreboard.
- A network of horizontal catwalks hung below the dome framing provides mounting points and access for sports and house lighting and also includes platforms for event spotlights and electrical and sound equipment racks.
- Other hanging loads including end video boards, house reduction curtain trusses, and hoisting and mechanical ductwork and equipment.

Roof Structure Analysis

The entire superstructure, including the roof, was modeled in SAP2000 to analyze the interaction of the bowl and roof for seismic design. The roof was also modeled alone in STAAD.Pro for analysis and design of roof members in a smaller, more manageable model. Results from both models were compared and reconciled in the end as a quality cross-check. Radial trusses are generally in compression at the top and bottom chord, but also have bending capacity for the many concentrated and unsymmetrical loading conditions. The distributed circumferential rings resist the thrust forces normally resisted by a single large tension ring at the perimeter. Rings are in compression in the upper half and in tension in the lower half of the roof.

Connection Design

The roof system offered the advantages of a lightweight, symmetrical, and repetitive system. The challenge then became to configure the connections, which included numerous two-way connections at the truss/ring intersections, in a simple and economical manner. It was critical that the connection design and detailing be developed in conjunction with the design of the members. In such a complex geometrical and highly redundant structural framing system, the designers elected to design and detail the connections fully, with the following concerns in mind:

1. The member types and depths had to be coordinated with the connection configurations. Every connection type in the roof was a custom detail for this project, not a typical detail that could be referred to.
2. The scheme for the connections had to allow for a practical and speedy erection sequence that could be planned for and analyzed during the design phase.
3. The size of the members had to satisfy the net section requirements, especially in the high-tensioned members at the outer rings. With forces of more than 2,000 kips in tension and four rows of bolts at each flange, the governing design was based on the net section capacity.
4. The large number of bolts and complex details would have made the estimation and bidding of the connections very difficult for the fabricator without the connections being fully detailed on the contract documents.
5. Having the connections fully detailed on the drawings allowed the shop drawing detailers to complete their work in a much shorter period because they were not required to design these complex connections.
6. All major connections were detailed to be field-bolted. Field-welded connections were kept to an absolute minimum.

The radial arched trusses were detailed as shop-assembled and welded straight segments. The main "A" trusses consist of five straight segments. The secondary "B" trusses, which terminate at the fourth tension ring, consist of three straight segments. The splice locations of the segments occur at the same joints where the tension rings are located, so the splice connections are two-directional splices.

For the truss top chords, a two-plate sandwich was used to solve the two-way splice detail. For the tension circumferential members, plates were used at the top and bottom flanges of the top chords of the radial trusses. The truss vertical members at the splices were shipped loose with the bottom plates welded to their ends. This detail allowed circumferential forces to be transmitted in a direct load path above and below the chord members, avoiding force transfer through the truss members.

Splice connection end plates were used for the bottom chord splices and for the bottom circumferential tension members for simple fabrication and quick erection. The forces at the bottom tension members were much less than at the top members, except for the outer ring.

The outer ring members at the supporting columns behaved as the main tension rings, with very large forces at the bottom ring compared to the top ring. The tension forces, however, changed as each member's axial area changed. Therefore, the top tension ring members were oversized to reduce the bottom chord forces.

The connections at the end of the trusses were detailed with sandwich plates at both the top and bottom chords, with plates being shop-welded to the truss vertical.

Seismic/Lateral System

Located in the vicinity of the New Madrid Fault, Memphis is in a relatively high seismic zone. Designed per the 1999 *Southern Building Code* (SBC), the building falls into seismic performance category D, the most severe. The seating bowl was "sculpted" to shift seats from the ends of the arena to the more desirable sideline positions. As a result, the main arena's footprint is a 436′-diameter circle, segmented into 36 facets. The structural engineering team, led by Ellerbe Becket, took full advantage of this feature when designing the lateral system and roof structure.

Circumferential, concentrically braced steel frames are located in 12 of the 36 bays of the exterior circular column grid creating a rigid "can" effect. The bracing was configured as double bay chevron braces in six pairs of adjacent bays with field-welded steel pipes, 14″ and 16″ in diameter. By the time the concrete upper concourse level was reached, structural steel had been detailed, fabricated, and was being delivered sequentially for the remainder of the structure. Steel was also chosen for the upper seating bowl raker beams, which saved money and one month from the schedule when compared to forming and placing sloping elevated concrete raker beams.

A variable width slab transition zone exists between the back of the lower concrete seating bowl and the steel "can" grid. This transition zone was framed with steel floor beams and composite deck. The connection to the concrete bowl was made through steel embedment plates, making the connection of the floor diaphragms to the "can" grid standard steel construction. The majority of the lateral load is taken through the donut shaped floor diaphragms into the "can" bracing. The rigidity of the concrete lower bowl frames was also taken advantage of to contribute to lateral resistance. The resulting lateral resisting system became a combined system of circumferential concentrically braced steel frames and rigid concrete frames, a system permitted by the 1999 SBC.

Keys to Fabrication

One of the biggest advantages of the dome framing system compared to other long span roof systems is that the truss depths are much smaller—only 12′ compared to 30′ to 35′ for the typical arena roof, which also requires complete field assembly of the trusses. The working line for the trusses is a 500′ radius; however, the truss is segmented into approximately 40′ straight sections to form the overall curve. The shallow truss depth allowed the 40′ sections to be shop-welded and shipped to the job site as one piece. All sections of the trusses were aligned and assembled in the shop to ensure that each 500′-radius truss was properly fitted and allowed for full bearing at each splice connection.

Another advantage of this system was repetition. According to AISC member fabricator W & W Steel, because the A and B trusses had numerous pieces that were the same, the shop had various jigs set up where repetition reduced man hours significantly.

Welded trusses tend to grow dimensionally in length during the fit-up/welding process, so overall dimensions were monitored in the fabrication shop. Shims were provided in the splice joints to allow the erector to either use them or drop them out as each segment was erected toward the compression ring. Oversized holes, used at all splice plates for the ring framing, employed Class B slip critical connections to allow for field tolerance and adjustment.

Critical Schedule

The 26 month construction schedule included a seven month window for steel erection, following work for the concrete superstructure and preceding the enclosure of the building's skeleton. This ag-

The bottom chord central hub was shop-assembled to ensure proper fit-up.

gressive and unyielding time frame was critical to the success of the project. W & W Steel and AISC member erector LPR Construction Company were brought on board through a negotiated, yet competitively-awarded, proposal process early in the design phase, affording the owner the opportunity to benefit economically from the collaboration of the design and construction teams.

Roof Steel Erection

The erection of the arena's steel, including the roof, was performed by LPR. Ellerbe Becket shared their SAP2000 3D structural model with LPR to help with development of the erection planning, shoring loads, and wind loads during the various phases of construction. The model was also used to help determine temporary stability requirements for the trusses during intermediate stages of construction, including applied wind loads on the trusses before being stabilized by adjacent framing. Once the shoring loads were determined, LPR designed and fabricated approximately 220 tons of shoring for the project, consisting of 12 towers approximately 145′-tall plus a center cluster of four towers approximately 154′ in height. The key to the design of the 145′-tall shores was that they span the entire height with no intermediate lateral supports or guys. Individual shore design loads for the project were in the range of 400,000 lb. The base of each shore was designed with an adjustable shim support system to hold the shores in place while allowing hydraulic jacks to be inserted to raise or lower the entire shore while fully loaded.

A Manitowoc Model 2250 Crawler Crane was chosen for hoisting the 95,000 lb, 210′-long arched trusses. Each primary truss was delivered by W & W Steel in five segments. The lower three-segment portion of the truss (61,000 lb and 128′ in length) was assembled on the arena floor and hoisted into position between the perimeter column and one of the outer ring shores.

The main crane was configured using a short luffing boom to enable it to work in a relatively small area within the arena floor while retaining the ability to reach up and over the primary trusses to set the lighter 128′-long secondary trusses.

The erector chose to erect the first quarter of the roof structure between the perimeter and the first ring of shores to establish stability within the erected framework. This also conserved the limited working space inside the arena prior to installing the center shoring cluster. After the first outer quarter section was erected, the center cluster of shoring towers was installed with stabilizing guys anchored to the surrounding cast in place arena structure. The entire 30′-diameter, 115,000 lb compression ring was assembled on the ground and hoisted atop the center shoring cluster using two cranes.

After the compression ring was in place, the upper two segments of the first four trusses were hoisted into position to complete the first quarter of the roof structure. The remainder of the structure was then built "one slice at a time," erecting the outer 128′ truss section onto a shore and then dropping in the inner 82′ section.

Once the roof structure was complete, and designated portions of the roof deck were in place, the 16 shores were lowered in stages by the hydraulic jacks at the base of the shores.

Weathering the Storm

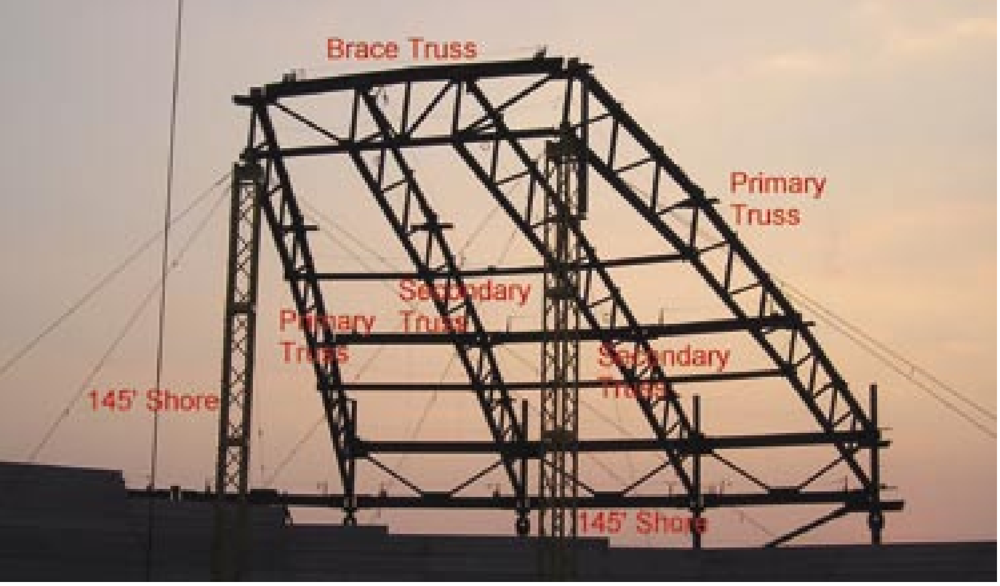
On July 22, 2003, primary truss erection was just taking shape and the first two temporary shores were in place. Two of the twelve outer primary truss sections had been erected on top of the 145′-tall shoring towers. Guy cables were installed and carefully tensioned to stabilize the tops of the towers, and a single bracing truss had been installed between the two primary trusses between the shores.

The work day was just beginning when an approaching storm brought tornado-like winds. As it turned out, there was no tornado. The storm was officially called a "Bow Echo," a weather phenomenon capable of causing straight-line, sustained, and widespread winds in excess of 100 mph. News reports indicated straight-line winds up to 120 mph for this storm.

LPR had planned for a 70 mph wind speed during all critical stages of construction, yet the system held its position. An extra pair of guy cables had been installed on the first two shoring towers for the purposes of redundancy. The loss of an single guy cable could have caused a catastrophic failure at this stage of construction, so the erection plan called for duplicate guy cables to be installed roughly parallel, yet spaced a safe distance apart, from its twin cable. As it turned out, the redundant guy cables were just what were needed to resist wind pressures more than twice the anticipated design load.

The cranes did not fare as well. All three of the tower cranes on the project were permanently damaged and distorted after the storm. One of the tower cranes suffered total failure as the bolts in two corners of the main tower stretched and broke under the wind load. Ultimately, all three of the tower cranes were removed from the project and very little remedial action was required for repair of the structure.

With the sudden absence of the tower cranes, the erection plan was quickly revised to erect the remainder of the lighter roof steel from the ground using conven-



View of a segment of the dome during erection.

tional and luffing boom cranes. After a multiple-week delay, an aggressive acceleration schedule was implemented. The steel roof structure was completed ahead of the accelerated schedule and the shores were promptly removed, coinciding with the date originally planned. ★

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Fabricator
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Erector
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