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The Florida Suncoast Dome

Describes the conceptual and structural design, production and erection of a $100 million multipurpose stadium in St. Petersburg, Florida. This all precast prestressed concrete structure utilizes more than 7000 precast components weighing over 50,000 tons (45350 t). A large variety of precast products were used including columns, beams, raker beams, double tees, hollow-core slabs, ramp elements and stadium seats. The first part of the article discusses the structural and construction aspects of the ring beam framing while the second part discusses the concourse and seating area framing.

While many domed stadiums have used various portions of their structure or seating using precast prestressed concrete, few if any have employed a completely precast structural system. Such is the case, however, for the Florida Suncoast Dome, in which all of the structural elements and elevated seating are completely precast concrete (except for the fabric roof).

The diameter of the stadium is 688 ft (210 m) with a height of 225 ft (69 m). It covers an area of 8½ acres (3.44 ha). The facility has a seating capacity for 43,000 spectators plus 60 private suites (see Fig. 1).

The Florida Suncoast Dome, therefore, can truly be thought of as a large and most unusual precast concrete structure. Not only is it larger in diameter than the Super Dome in New Orleans, but it is also the first cable supported dome of its kind in the United States and the largest in the world.

Although the structure is designed primarily as a baseball stadium, sections of the stands are moveable on inflatable rubber tires so that it can become a mixed multisport arena. In addition to spectator sports such as basketball, tennis and soccer, the facility can also be used for concerts, exhibitions and other commercial events.

Structural System

The structural system can be considered somewhere between a hard dome (such as the King Dome in Seattle or the Astro Dome in Houston) and an inflated roof (such as in the Silver Dome in Detroit or the Hoosier Dome in Indianapolis). The roof system is actually an ultralight roof called a cable dome, and it employs a tension cable and compression post system originated by R. Buckminster Fuller termed a "tensegrity" system.

The cable roof system employed in the Suncoast Dome Stadium was designed by the late David Geiger. The roof is spanned using continuous tension cables, both radial and circular, and individual compression posts. Loads are carried from a center tension ring through 24 radial ridge cables to a perimeter compression ring. Therefore, the structural roof system is actually reversed to a typical dome structural system; the typical hard roof dome has a center compression ring whereas this system has a perimeter compression ring.

The cable structural roof system as envisioned by R. Buckminster Fuller was described as "continuous tension throughout with compression elements becoming small islands in a sea of tension." The Geiger revision uses continuous tension cables and discontinuous compression posts. The loads are carried from a central tension ring through a series of radial ridge cable tension hoops and intermediate diagonals until they are resolved in the perimeter compression ring.

The compression is created in the exterior outer ring by virtue of the radial post-tensioning forces, which create tremendous inward pressures on the ring (Fig. 1). The circumferential compression design force was 13,000 kips (57824 kN). A dome similar to this was completed in Seoul, Korea, for the 1988 Olympic Games, but is much smaller; it has only a 13,000 seating capacity and a 393 ft (120 m) diameter.

The fabric roof is made of teflon-coated fiberglass. One of the main advantages of the cable supported system is that it is far less expensive to operate than typical air supported systems which require constant air pressure to maintain their integrity.

The original design of the structure called for a cast-in-place framing system. POMCO, the precast concrete supplier, and Huber, Hunt & Nichols, the general contractor, offered an all precast concrete alternate for the framing system and were low in the bidding process. The Consulting Engineers Group, Inc., was then hired to completely redesign the cast-in-place structural frame to an all precast concrete system using the roof system loads furnished by Geiger/KKBNA.

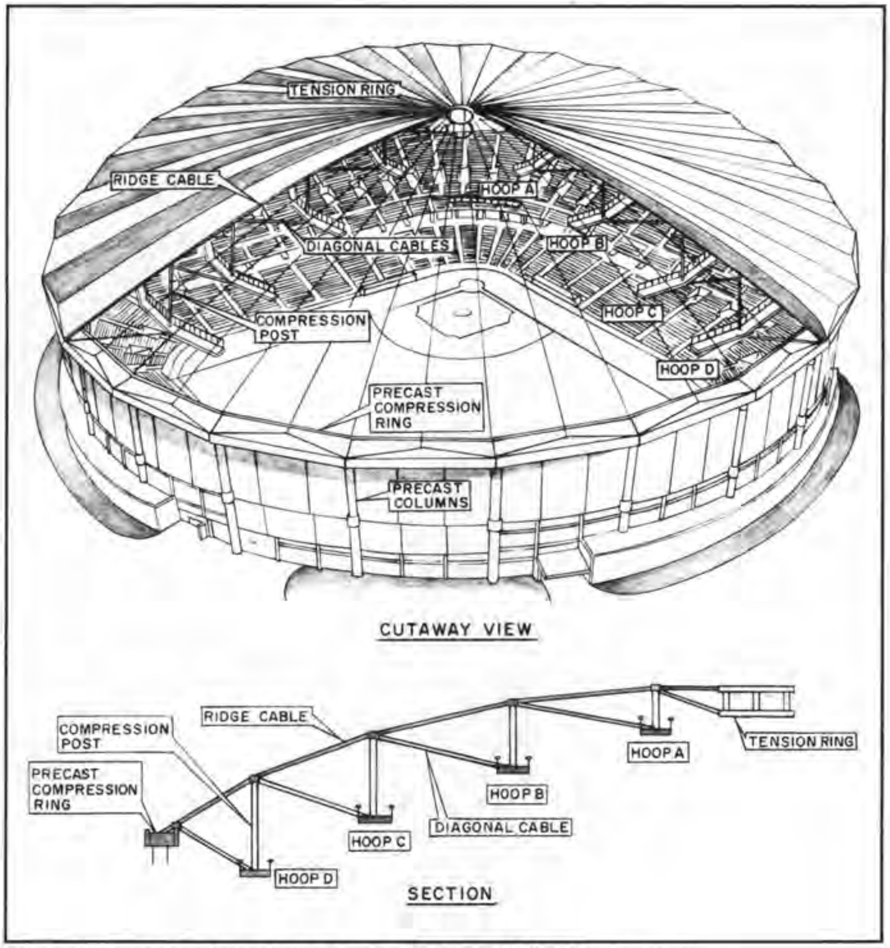

Fig. 1. Schematic of stadium showing basic roof framing system.

The construction stages were modified to accommodate the precast concrete system. The precast concrete system construction stages were as follows:

1. Ring beam framing erected and completed.
2. Steel lateral load framing system installed as individual bays of the precast system were completed.
3. Partial seating and concourse levels erected where they would not conflict with raising the cable roof.
4. Cable roof is raised, seating and exterior closures and circular ramps completed.
5. Remaining infield seating and meeting space and playing field completed.

The schedule of work as actually accomplished was as follows:

- Cast-in-place sitework started on April 4, 1987.
- The first pour of a precast concrete column commenced on June 25, 1987, which was the very first ring beam column.
- The first erection of a precast concrete element occurred on July 21, 1987, which was also a ring beam column.
- Roof cables were strung from June 1988 to April 1989.
- Fabric was placed on the roof from April 1989 until August 1989.
- The expected stadium completion date is April 1990.

Fig. 2 shows how the diagonals of successive rings are tensioned forcing the vertical compression posts upward and achieving the curved roof configuration. In this process the roof cables are strung to a temporary tension ring; then the hoop elements are assembled on the ground and raised in position. The posts are erected in their proper locations and then stressing begins.

A model of the ring beam framing showing the various precast concrete components was constructed by Paul Kraemer of

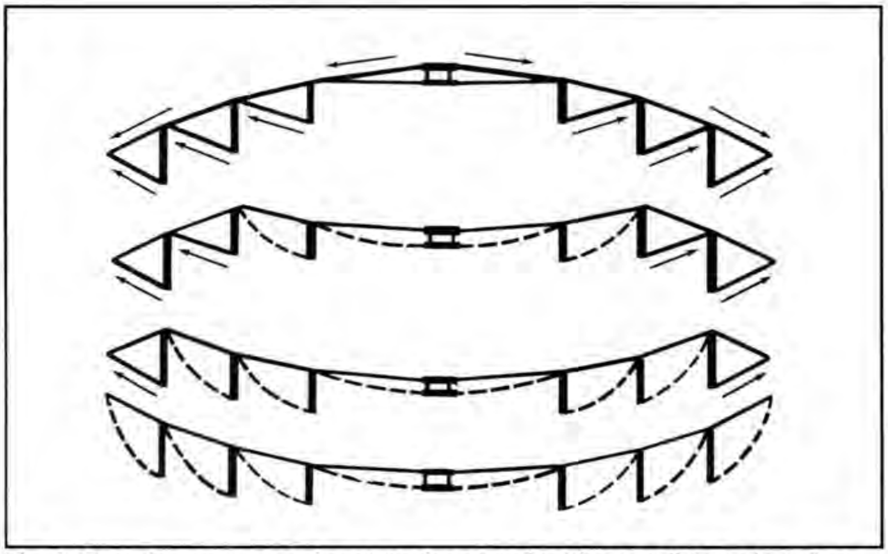

Fig. 2. Stressing sequence of post-tensioned roof cables to pull dome into place.

POMCO and was very useful in visualizing the framing system. The main ring beam framing consists of 6 ft (1.83 m) diameter precast concrete round columns supporting an 18 ft wide x 8 ft 6 in. deep (5.49 x 2.59 m) precast concrete ring beam which resembles a giant wide flange laid on its side.

Precast concrete raker beams, double tee concourses, ramp members, seating, hollow-core slabs, columns and beams comprise the major products used in the remainder of the stadium.

Many aspects of the design were dictated by handling and erection capabilities. The smaller, more typical components, such as seat sections, columns, beams and double tees, were produced at POMCO's main manufacturing plant in Port Manatee. POMCO elected to manufacture at a jobsite plant those members that were too large to haul over the local highways. This precasting plant was set up in an area which was later used as a parking lot for the stadium complex.

Table 1. Basic ring beam structural components.

MAIN COLUMN	6'-0"; 10"	LENGTH - 20' - 56' MAX. WEIGHT - 135,000 # MAX. REINFORCING - 26 #18 SQ BARS COLUMN LOADS TO 1,500,000 #
RING BEAMS	16"; VARIES 8'-6" TO 7'-6"; 8"; 8"	LENGTH VARIES: 90' MAX. WEIGHT 155,000 # MAIN PRESTRESSING 3-12 STRAND POST-TENSION TENDONS MAX. BENDING MOMENT 3,000,000 # LATERALLY UNSTABLE
FLAT SLABS	15'-3"; 6"	TYPICAL LENGTH 19'-9" TYPICAL WEIGHT 23,000 #
KNUCKLE ASSEMBLIES		36WF AND 24WF BEAMS WITH A 6" C.I.P. SLAB SUPPORTED BY A 2 1/2" METAL DECK
STEEL FRAMING		BASIC STEEL SHAPES - 10" WF PLUS VARIOUS ANGLES AND CHANNELS CONNECTED TO CREATE TRUSSES

Basic Stadium Components for Ring Beam Framing

The ring beam, which is the most unique part of the stadium, is made up of very large precast concrete elements. These elements had to be accurately cast and erected in the field to extremely tight tolerances in very inaccessible elevations. To avoid excessive scaffolding or hanging stages, the connections were carefully conceived to be either self-aligning or make use of separate alignment devices. This is best exemplified by the large 6 ft diameter x 56 ft long (1.83 x 17 m) hollow precast concrete columns.

It was decided that no column joints or splices would depend upon dry packing for their erection stability. This is because the members were extremely large plus the inaccessibility of dry packing joints 60 to 120 ft (18 to 37 m) in the air. The columns stand structurally unsupported for up to 170 ft (52 m) until the diagonal steel bracing is placed.

Therefore, carefully formed butt joints were used, and NMB splice sleeves and separate alignment and stabilizing jigs were employed. The alignment jigs also served as working platforms to accomplish the NMB sleeve grouting at a time later on and to stabilize the column pieces until the splice sleeve connection was complete. This same thinking regarding erection needs carried through to the knuckle assembly attachments, the connections of the precast concrete flange beams to the knuckles, and to other structural assemblies.

The required field accuracy can be explained by understanding that the roof cable post-tensioned anchor plate and the exiting of the roof tendons from the concrete surface of the knuckle had to be accurate within 1 degree. It must be appreciated that this task was accomplished 170 ft (52 m) in midair while pouring 60 cu yd (46 m^3) of concrete on top of, in and around the post-tensioned anchorage assembly which was an element of an ellipitically shaped roof pitching 15 degrees.

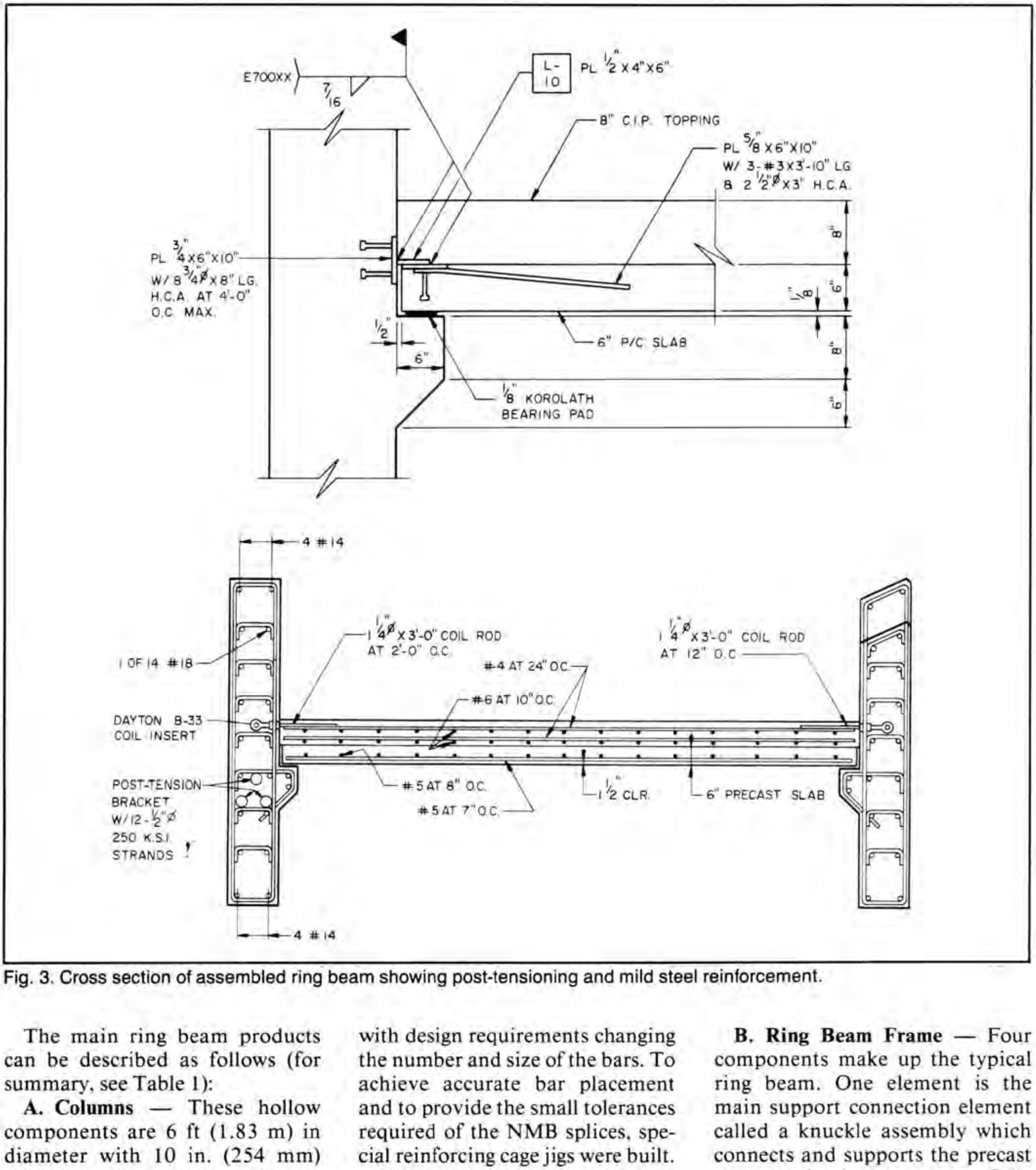

Fig. 3. Cross section of assembled ring beam showing post-tensioning and mild steel reinforcement.

The main ring beam products can be described as follows (for summary, see Table 1):

A. Columns — These hollow components are 6 ft (1.83 m) in diameter with 10 in. (254 mm) thick walls. They were cast in special forms with an interior collapsible mandrel. The loading in the columns changes substantially because of varying length and lateral loads. The largest reinforcement was 26 # 18 S bars, or 5.3 percent reinforcing steel area as related to concrete area.

For ease of manufacture and field connection, all reinforcing steel occupied the same location with design requirements changing the number and size of the bars. To achieve accurate bar placement and to provide the small tolerances required of the NMB splices, special reinforcing cage jigs were built.

The maximum column vertical design load was 1500 kips (6672 kN). The typical manufactured length was 56 ft (17 m), with the top column length varying based upon the 15 degree slope of the roof. The columns were typically made in four lengths with a maximum total length of 170 ft (52 m). Full moment splices were achieved at the column joints with the use of NMB splice sleeves.

B. Ring Beam Frame — Four components make up the typical ring beam. One element is the main support connection element called a knuckle assembly which connects and supports the precast concrete ring beams on top of the precast columns. The remaining elements are all precast concrete and are arranged to create the giant 18 ft (5.49 m) deep beam (Fig. 3).

The primary elements are the flange beams. These consist of two members, one outside and one inside beam. The outside beam is 8 ft 6 in. (2.59 m) high, 16 in. (406 mm) wide and approximately 90 ft (27 m) long. The inside beam

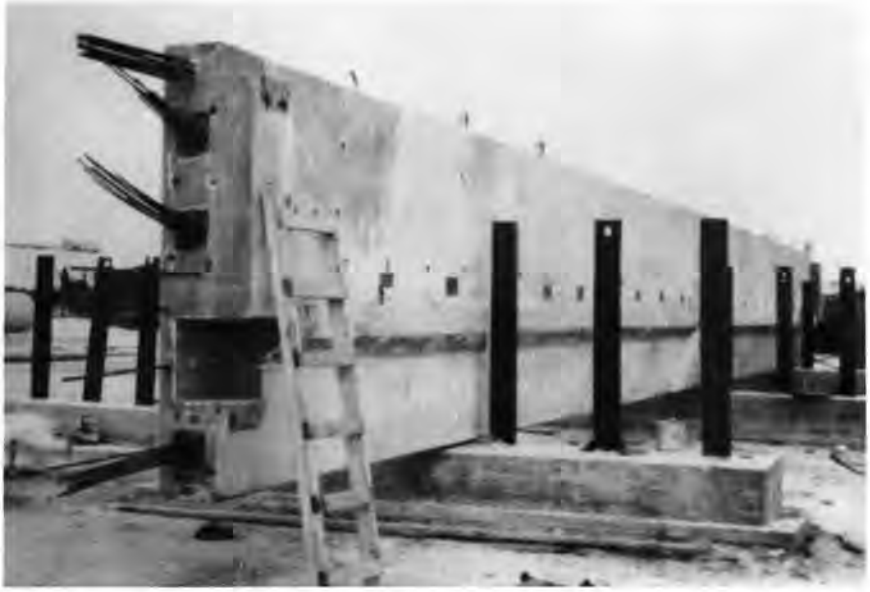

Fig. 4. Typical precast post-tensioned flange beam for ring beam assembly.

tapers from 8 ft (2.44 m) on its end to 7 ft 6 in. (2.29 m) on the center and is also approximately 90 ft (27 m) long.

The irregular interior beam depth creates the scalloped roof configuration. The interior beams are slightly shorter than the exterior beams and both are 16 in. (406 mm) thick. The long beams had to be this thin to minimize weight — and even at that, they weighed 160,000 lb (712 kN).

This heavy weight and their location 170 ft (52 m) in the air required the beams to be erected with two cranes. Because of their thin cross section, they were unstable under normal wind loads and had to be laterally braced until the web slabs could be erected and connected. Both beams have haunches on their sides to support flat slabs and the 10 in. (2.54 mm) composite concrete which acts as the web of the giant I-beam.

The web flat slabs are basically 6 in. deep x 15 ft 3 in. wide x 19 ft 10 in. long (152 mm x 4.80 x 6.05 m) and have 10 in. (25.4 mm) of composite concrete placed on top of them in the field to tie the entire system together.

Fig. 3 shows a cross section of the main ring beam and identifies the deep flange beams, precast concrete slabs and composite concrete tying the 18 ft (5.49 m) wide flange ring beam together. The ring beam was designed for dead and live flexural loads with a 3000 ft-kip (4070 kN•m) bending moment plus a 13,000 kip (57800 kN) axial load. The main ring beam reinforcements were three 12 strand post-tensioned tendons plus mild steel shear and torsion reinforcing bars. Fig. 4 shows a completed beam in storage.

C. Knuckle Assembly — The knuckle assembly is made of a 36 in. (914 mm) wide flange steel beam and perpendicular framed 24 in. (610 mm) wide flange steel shapes with a metal deck between them. A sufficiently strong metal deck could not be found to carry the 8 ft (2.44 m) of cast-in-place concrete, which is required to complete the knuckle assembly, so a 6 in. (152 mm) composite slab was poured on the ground on top of the metal deck. This created sufficient

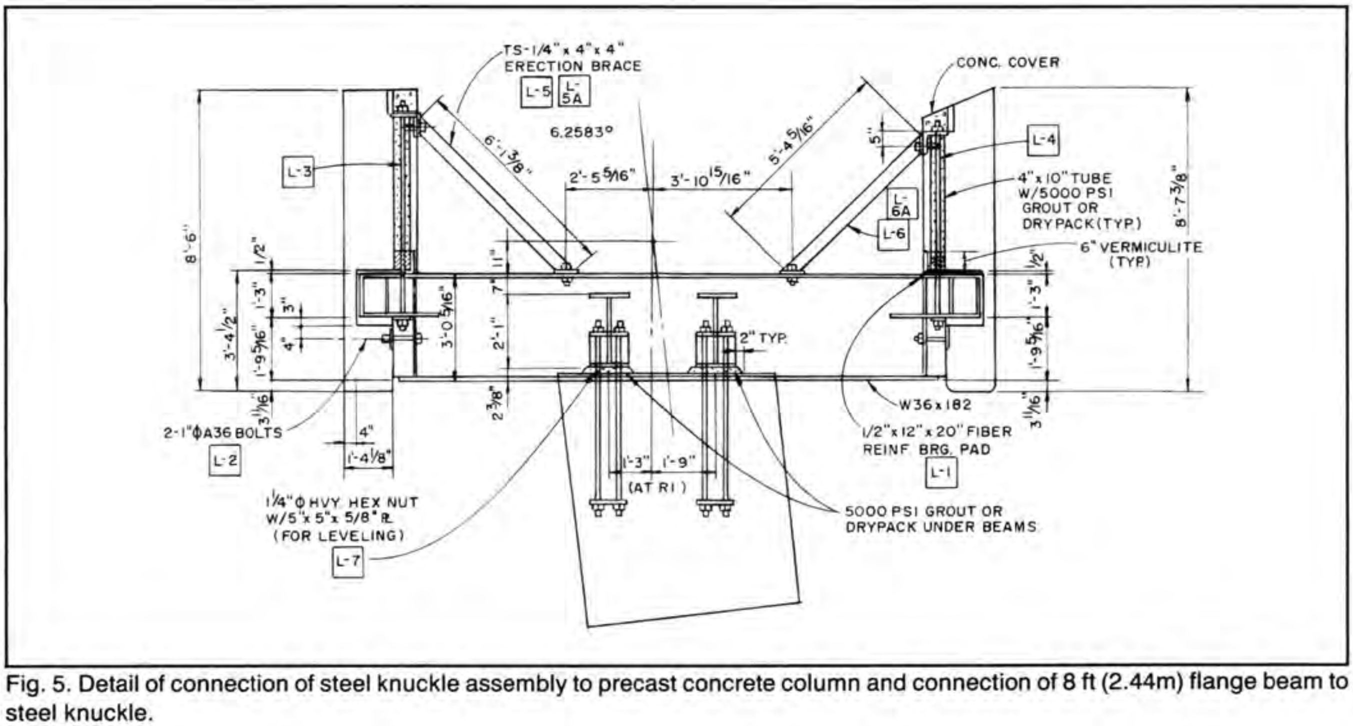

Fig. 5. Detail of connection of steel knuckle assembly to precast concrete column and connection of 8 ft (2.44m) flange beam to steel knuckle.

strength to carry the 8 ft (2.44 m) of field placed concrete. Details of the knuckle assembly are shown in Fig. 5.

The cast-in-place concrete which completes the knuckle assembly creates the roof cable post-tensioned anchorages. These forces ranged from 2000 to 3000 kips (8896 to 13344 kN), which meant that in essence a pretensioned bed anchorage had to be built 170 ft (52 m) in the air in 48 locations.

D. Lateral Load Steel Framing — The lateral wind loads are taken by diagonal steel ties made up of wide flange beams plus horizontal hoop trusses. The lateral frame also supports the metal siding which encloses the structure. The roof is positioned at a 15 degree slope to reduce structural volume. This created substantial geometry problems, because while the roof is circular in its 15 degree attitude, when projected to the ground, it is actually an ellipse.

Consequently, many geometry problems had to be considered regarding the connection and attitude of all the knuckles located on top of the structure. Since the work point was located above the top of the column and the knuckle had to be located in a perfectly radial position and the top surface of the column was also sloping, there was a need for special geometry control. CEG was able to provide accurate and correct dimensions, attitudes and member lengths using their computer aided drafting system and various company developed software programs. As a result, the field fit was essentially perfect.

Erection Stages

Column Erection — The columns were the first components of precast concrete erected. There are a total of 24 columns, cast in three or four pieces — the longest piece being 56 ft (17 m). The total maximum column length was 170 ft (52 m). The columns were connected at the base with anchor bolts plus a special cast-in-place plug to provide a degree of fixity at the column base so that they could with-

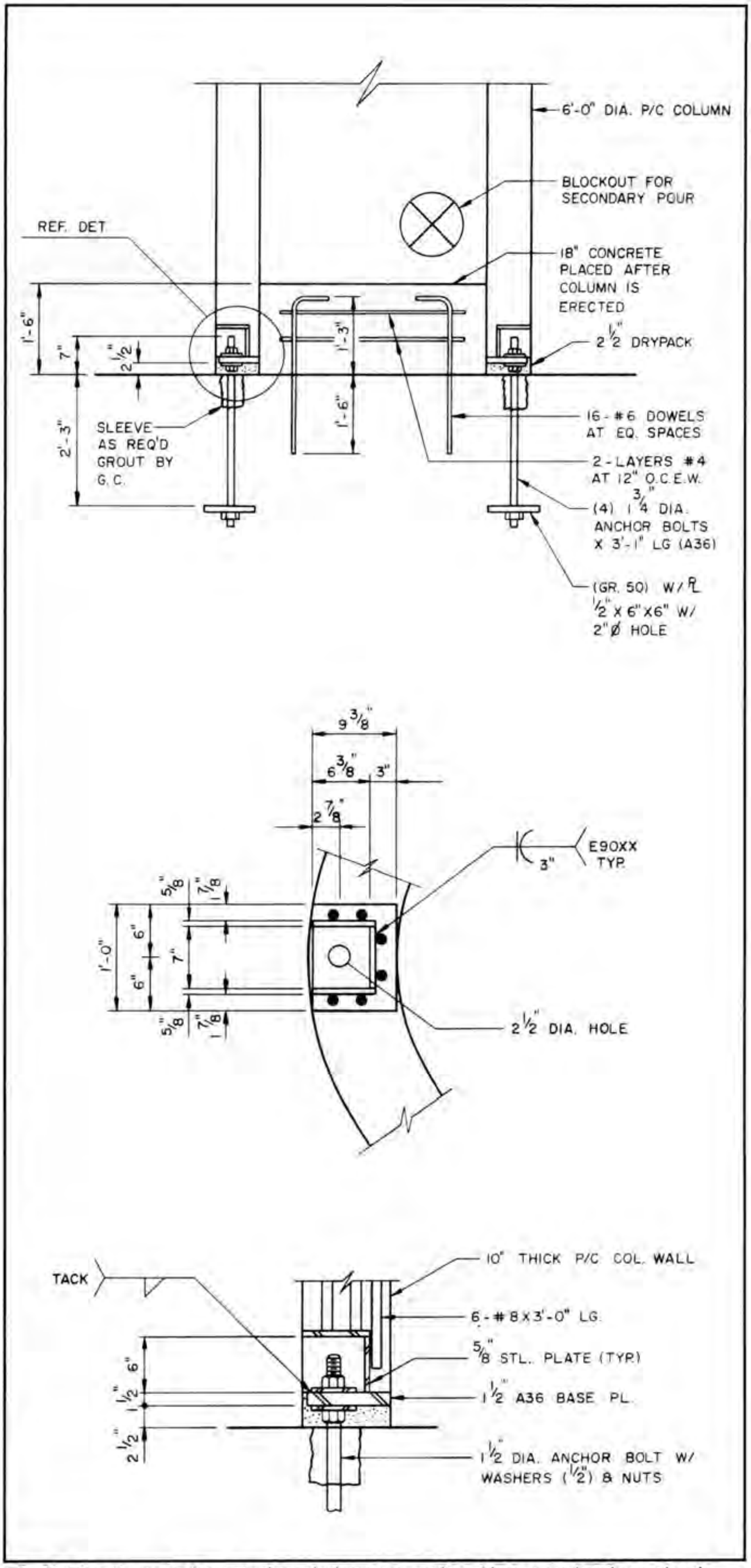

Fig. 6. Ring beam column to foundation connection. Note cast-in-place plug in circular void.

Fig. 7. Completed column with splice bars extending. A blockout is also shown to accommodate steel framing.

stand a certain amount of wind load, based upon a cantilevered flagpole design, until the framing system was completed.

The first two lengths of columns could be erected without bracing by using the anchor bolts in the cast-in-place plug. The bracing was then installed, and the remaining columns and the ring beam erected. Although the columns were not match cast, the forming system provided relatively perfect aligned faces of the columns so that they were merely sealed, similar to segmental beam construction. The projecting splice bars were then aligned with the splice sleeve assemblies in the lower portion of the column above it. Once the columns were fully erected, the NMB splice sleeves were grouted to complete the connection.

Fig. 8. Precast concrete deadmen connected with post-tensioned bars. This is used to support roof until roof cables are installed and steel bracing is in place.

Fig. 6 shows a detail of the column base connection with the four anchor bolts and the cast-in-place plug employed for added fixity.

Fig. 7 shows a typical column in storage where a slot has been provided in the top of the column. This slot allows the steel framing K brace to fit into the column as the column is erected. Prior to installing the steel lateral load framing, the K brace is carefully aligned at the proper angle and attitude and then anchored in place with cast-in-place concrete. A computer program was used to determine the required angle and diagonal brace length required for the sloping elliptical roof configuration.

The top of the column is solid and the connection between the knuckle assembly and the column was made by using 12-# 18 S bars and four 1⅛ in. (26 mm) anchor bolts. The anchor bolts provided a temporary connection while the 18 S bars were used for final load transfer. This connection supports the knuckle, which in turn supports the main flange beams of the ring beam assembly.

Because the work point was located at the center of gravity of the ring beam above the column and the top of the column was sloped, the knuckle assembly had to be perfectly radially oriented. Therefore, a special forming device was used at the column top bulkhead to properly orient the connection assemblies on top of the columns so that the knuckle could be set in its proper radial position.

Ring Beam

As mentioned previously, the thin beams were unstable under wind loads; therefore, during erection special stiffening trusses were used to provide the necessary stability to the members. In addition, because of the sloped surface, the flange beams were unstable after erection.

Fig. 5 is a section through the column showing sloping knuckle assembly flange beams and special braces that were used to prevent the 8 ft 6 in. (2.59 m) high flange beams from being blown over. A vertical bolted connection was used to connect the precast concrete flange beams to the steel knuckle assembly.

Steel Knuckle Assembly

The knuckle assembly was made up of 36 in. (914 mm) wide flange steel beams which cantilever out past the columns to support the end of the 160 kip (712 kN) precast concrete ring beams.

Fig. 5 also shows the support of the precast concrete beams on the knuckle assembly and the cast-in-place composite slab needed to support the remainder of the knuckle concrete.

Special Erection Considerations

As the columns were erected, the steel framing was fit into the slots left during the casting process. As mentioned previously, once two column lengths were erected, the anchor bolts and the cast-in-place plug could no longer take wind loads and lateral bracing was required. A bracing system was developed employing ½ in. (13 mm) thick, 30 in. (762 mm) diameter welded steel caisson pipe with articulated ends to support the columns and precast concrete ring beams until the roof system could be completed.

The steel pipe braces were supported from the ground using unique precast concrete deadmen. The deadmen measured 17 x 17 ft (5.18 x 5.18 m) and were made up of nine precast concrete elements which were bolted together using Dywidag post-tensioned rods (Fig. 8).

Precast concrete was used in the deadmen for several reasons; primarily, because the cast-in-place deadmen could not be left in place and would have to be removed after they were no longer required to provide stability to the structure. Therefore, it was determined that precast deadmen could be more easily removed from the site. Also, the deadmen assembly could be unbolted and taken apart and moved to another position around the perimeter of the stadium and reused.

Because the structure is basically circular, some integral stiffness was achieved as the circle was completed. Therefore, only nine pairs of braces were required to support the 24 column positions. A special bracing plan was developed by CEG to indicate where the bracing should be located for the different stages of erection as the circular ring beam was completed.

Fig. 9. 30 in. (762 mm) diameter pipe bracing in place to stabilize ring beam framing.

Fig. 9 shows the braces as they were installed to brace the columns. The ends of the 30 in. (762 mm) pipe braces were articulated to account for the differing elevations of the soil surrounding the stadium and therefore the different angles between the precast con-

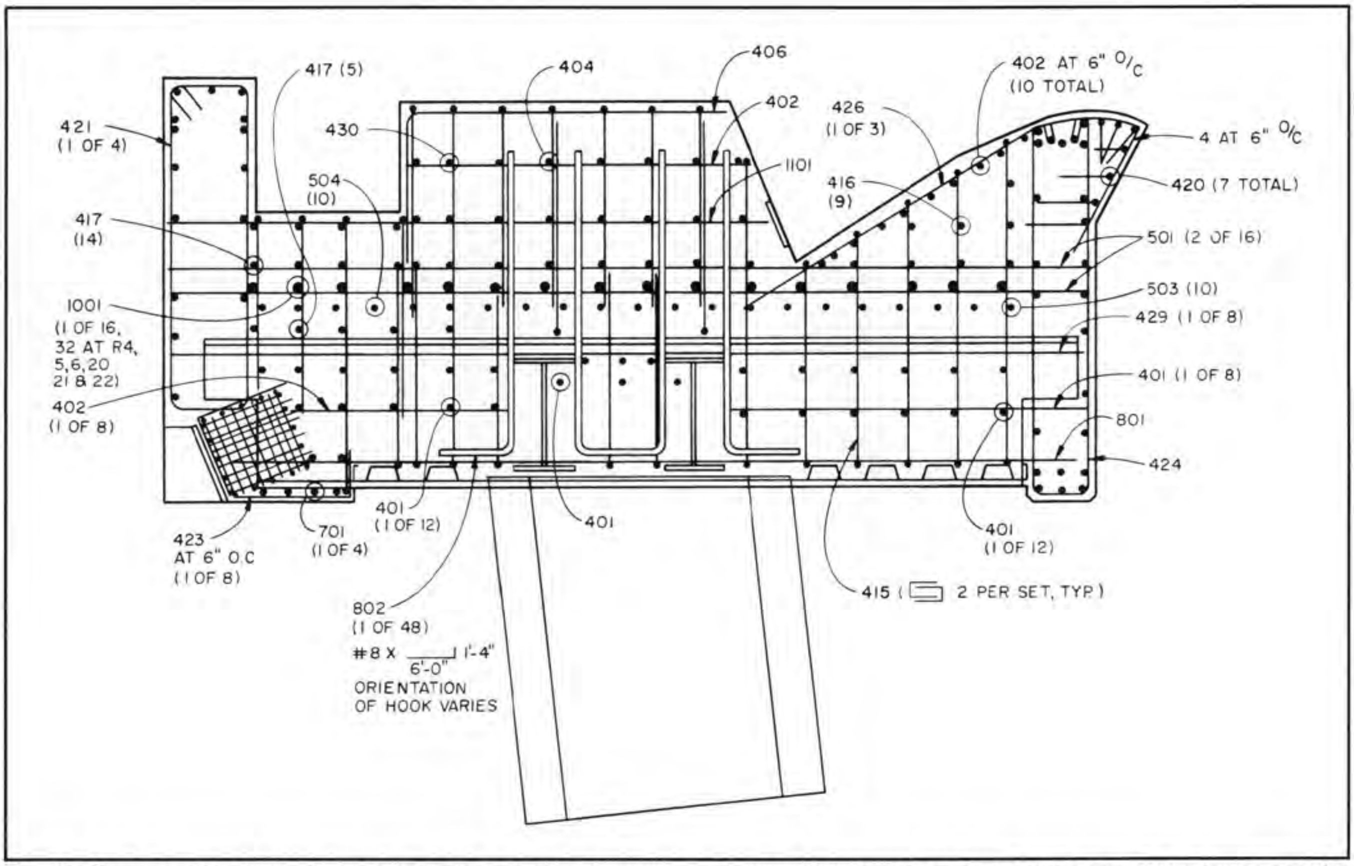

Fig. 10. Cross section through cast-in-place portion of knuckle assembly. Note bursting steel reinforcement in lower part for main cable anchorage (see Fig. 11).

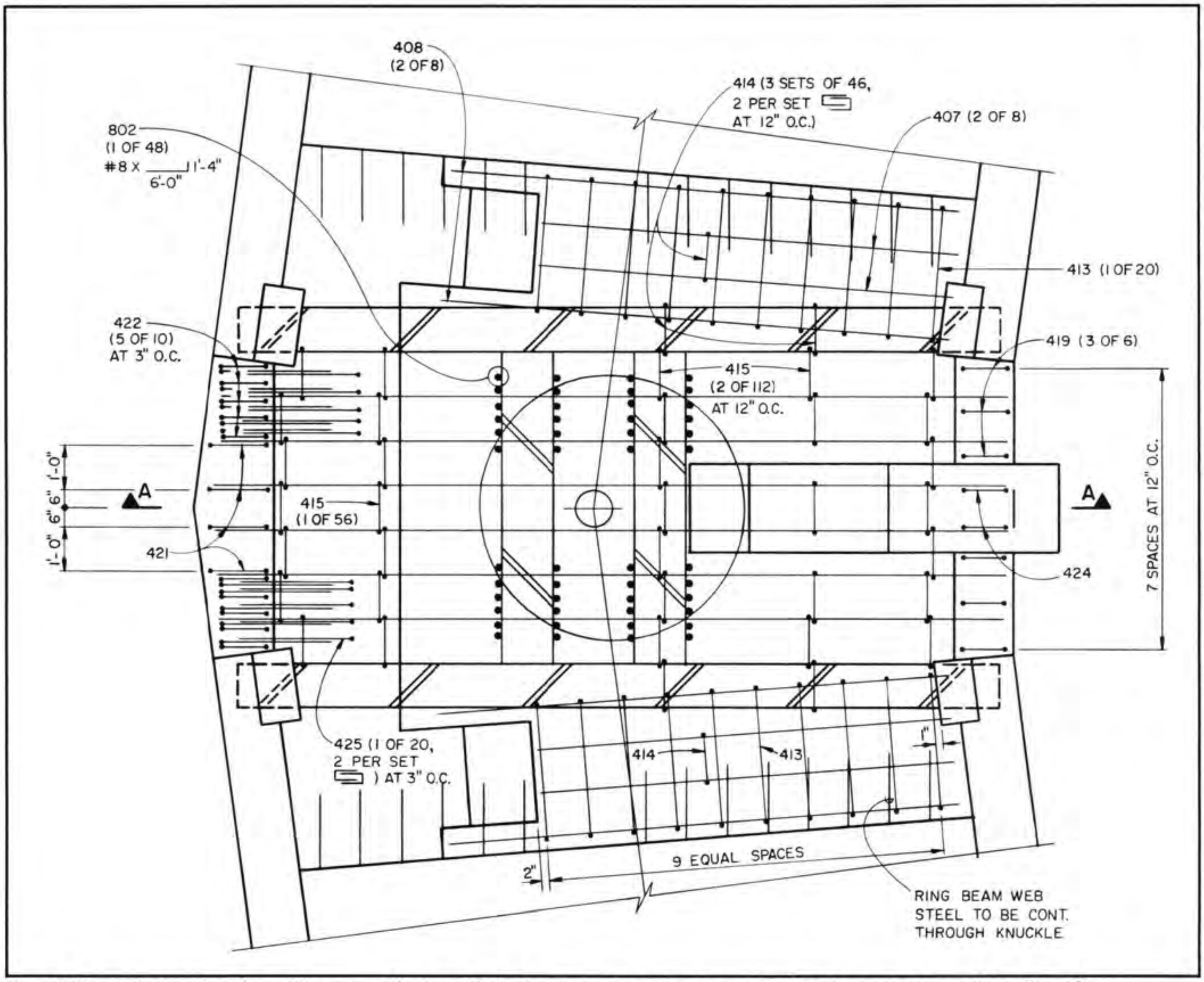

Fig. 11. Plan of complete knuckle assembly. Location of supporting precast concrete column is shown (see Fig. 10).

crete columns and the steel braces.

Another situation requiring special engineering related to the erection of the first pair of precast concrete ring beams on the first column. The column could not tolerate the extreme eccentric load with two 90 ft (27 m), 160 kip (712 kN) precast concrete beams on one side without serious cracking; hence, a cable supported deadman was used to balance the unbalanced load until the stadium could be completed. Thereafter, four beams were erected between three columns in a symmetrical manner. Once four flange beams were erected and the slabs were placed, then forming commenced on the cast-in-place knuckle assembly.

Fig. 10 shows a section through a knuckle assembly showing the required reinforcing steel. Fig. 11 is a

Fig. 12. Completed ring beam and knuckle assembly; at this height panoramic view of city and coast can be appreciated; note 15 degree beam slope.

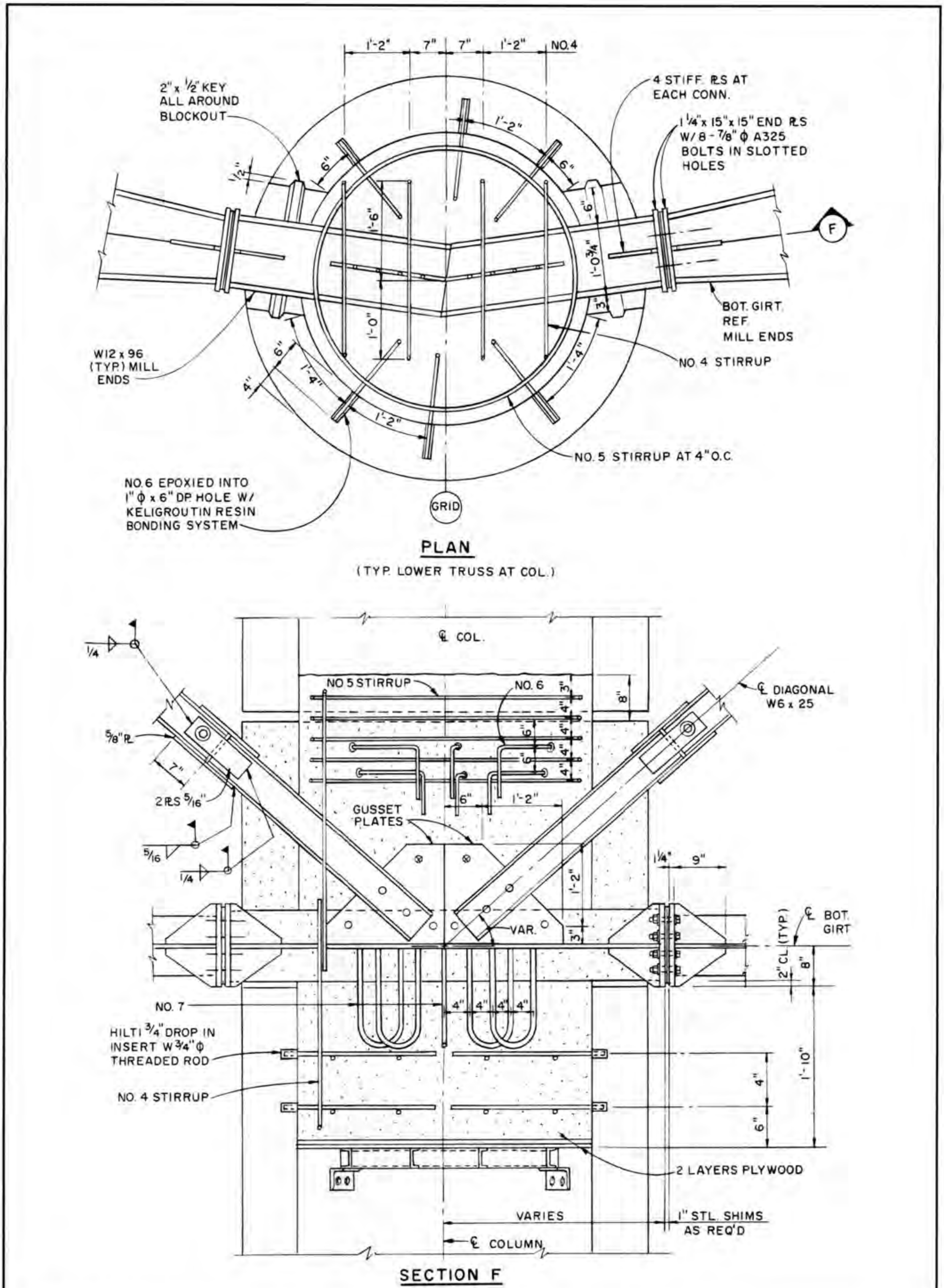

Fig. 13. Detail of steel lateral bracing connection to precast concrete column.

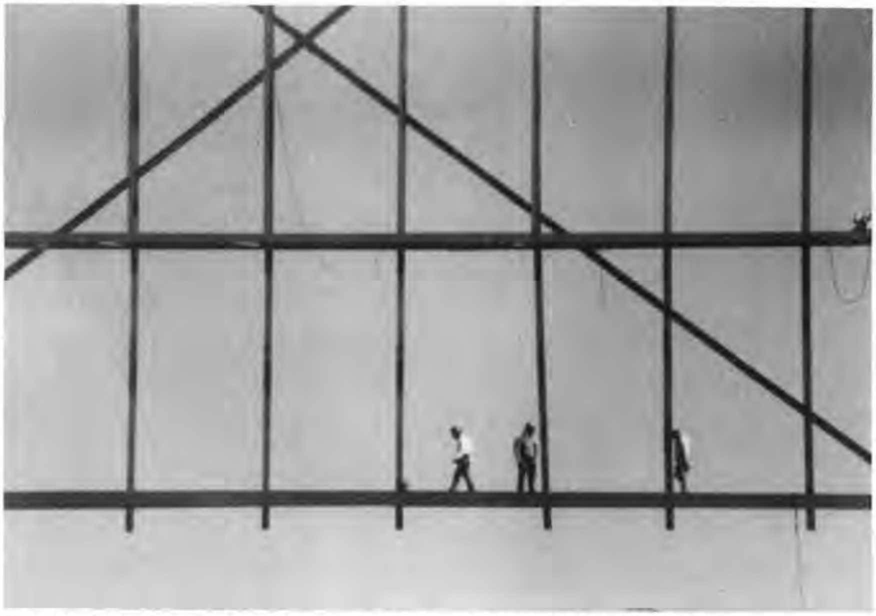

Fig. 14. Steel framing under erection showing ironworkers for scale.

plan view of the knuckle assembly reinforcement showing the bursting steel at the roof cable anchorage locations. A cast-in-place concrete saddle retains the roof cables and roof cable profile and anchorages. Note that the leading edge is curved so as to avoid stress concentrations on the concrete edge as the cable roof system is raised from the floor of the stadium into its final elevated position.

The total concrete volume per knuckle equaled 60 cu yds (46 m^3). An intermediate diaphragm was also used between a pair of columns which anchored the roof cables, creating the scalloped roof effect. The cast-in-place intermediate diaphragm had only one post-tensioned anchorage.

Fig. 12 shows the top of the ring beam with completed cast-in-place intermediate and knuckle assemblies. Note that a drainage hole was provided through the assemblies to allow for water to drain from the high side of the roof to the low side. The 15 degree slope is very apparent in Fig. 12.

Fig. 15. Completed steel bracing and metal siding supports.

Fig. 16. Cable roof in place; hoops are stressed around tension ring.

After the precast concrete columns and ring beam elements had been completed and knuckle assemblies cast, the lateral wind load steel framing system was erected. The primary elements here were diagonal braces, steel tension trusses and vertical hung beams and truss assemblies to support the steel siding.

To insure that the diagonals would line up, the K bracing was carefully located within the slots in the precast columns and then anchored with cast-in-place concrete. Fig. 13 shows the K brace assembly and its connection to the diagonals and horizontal trusses. The primary steel framing systems were 10 in. (254 mm) wide flange members plus angles and channels to create the trusses. Fig. 14 shows the steel framing during erection with ironworkers depicting the scale of the steel framing. Fig. 15 shows two bays of completed steel framing.

As the ring beam framing was completed, the last beam was fit into the opening and the circle was completed. The last pair of beams fit in the circle perfectly, again emphasizing the careful engineering, production and erection geometry, and quality control.

The next phase was to assemble the roof cables on the stadium floor and then raise the main roof erection cables. The circular hoops were assembled on the stadium floor and then lifted into position.

A steel center tension ring was installed and then the hoops were stressed, thus forcing the compression posts upward to provide the domed roof configuration (Fig. 16).

Fig. 17 shows a vertical roof post with the roof cables, which have been stressed to drive the post up, reaching from the precast concrete ring beam; the lateral bracing is also shown.

Once the hoops had all been completed and the roof cables installed, then the fabric roof was placed. Actually, the operation amounted to two layers of fabric placed from a large roller. The fabric was anchored at the outer ring

Fig. 17. Vertical roof post showing lower post-tensioned cables which are stressed to drive the post upward.

Fig. 18. Fabric roof being placed like a giant roll of tape.

beam and then rolled out to the center of the stadium, not unlike a giant roll of tape or toilet paper (Fig. 18). The roof seams were clamped at the ridge cables and the joints were heat sealed with a special seal and film to 680°F (360°C). The cables were then tightened and adjusted to create the scalloped roof appearance.

Fig. 19 shows the completed roof framing and cable roof.

The next focus will be on the interior stadium framing which covered the seating, concourse and various mechanical levels. The precast spiral ramps will also be discussed. This portion of the stadium was erected in two stages. The first stage followed the ring beam erection and included all material to the playing field perimeter. This area was kept clear for assembly of the cable roof components. Once the cable roof was completed, erection started again. Then, the remainder of the precast concrete framing and seating under the roof was erected, in addition to a small portion of the seating which was cast-in-place on grade.

First, the design features of the concourse are presented. Then, the various considerations involved in the design of the lateral load system are discussed.

Concourse and Seating Area Framing

From behind home plate to the left and right field corners, there are three levels of seating — upper seating, club concourse seating in the middle and lower seating as shown in Fig. 20. In the outfield areas there is one level of seating. The four concourse levels house the concession areas and the washrooms in addition to providing access to the seating from the outside ramps. The half-bay level above the upper concourse level is a mechanical area which has hollow-core slabs.

The concourse levels are designed for a live load of 100 psf (488 kg/m²) and a superimposed dead load of 50 psf (244 kg/m²). The mechanical level has a live load of 150 psf (732 kg/m²) and a superimposed dead load of 50 psf (244 kg/m²). Bays for the concourse levels are 42 x 42 ft (13 x 13 m). Four 10 ft (3.05 m) wide, 26 in.

Fig. 19. Completed roof framing and cable roof.

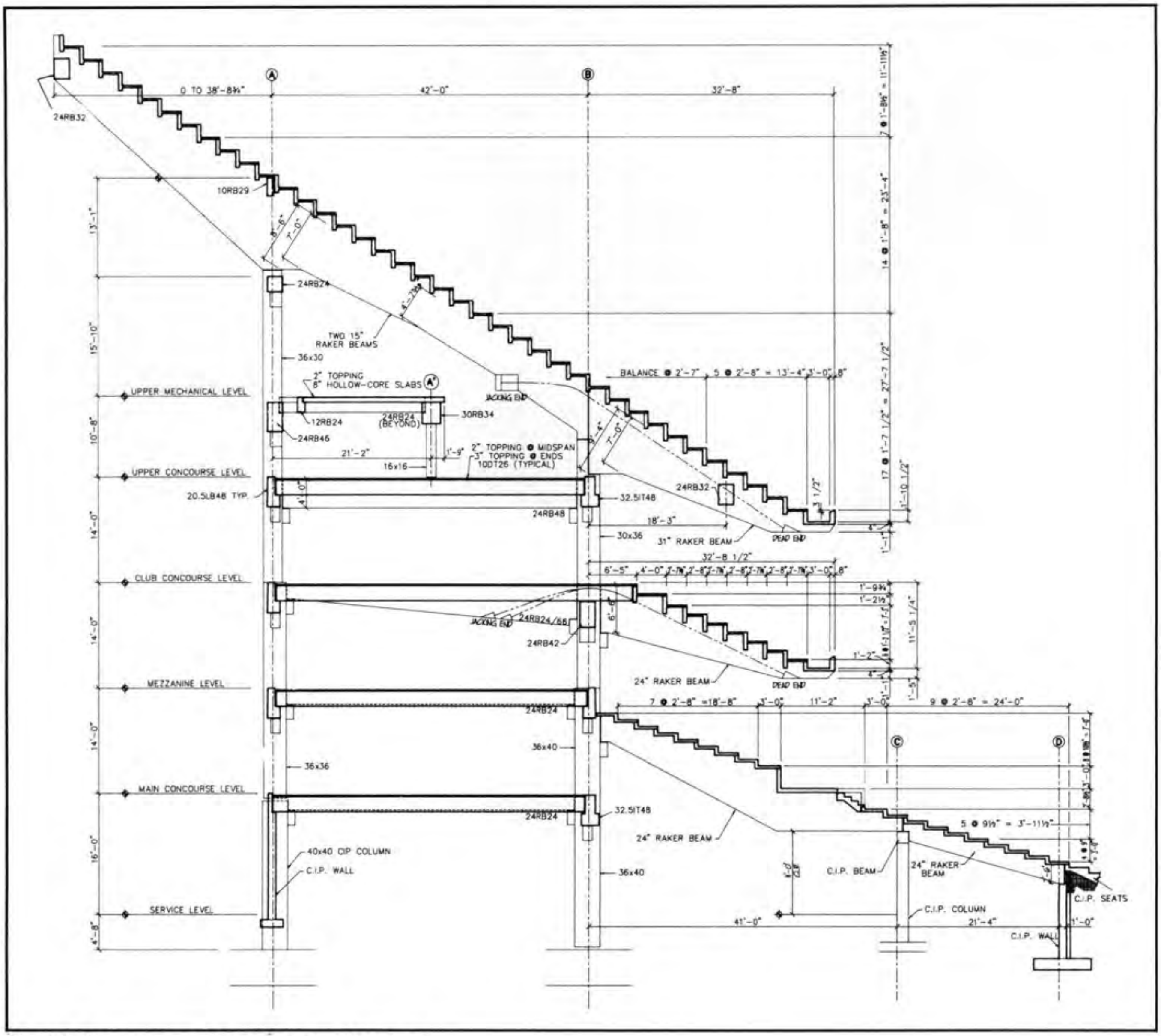

Fig. 20. Concourse and seating area framing.


Fig. 21. Concourse framing for 42 x 42 ft (13 x 13 m) bays showing double tees, ledger beams, rectangular beams, columns and shear walls.

(660 mm) deep double tees plus a 2 ft (0.61 m) filler beam are supported on 4 ft (1.22 m) deep ledger or rectangular beams as shown in Fig. 21. All concourse beams were prestressed. An angle connection on the beam top and a strap plate connection on the bottom were used to resist torsion.

The concourse columns were 36 x 40, 36 x 36 and 36 x 30 in. (914 x 1016, 914 x 914 and 914 x 762 mm) in cross section. They were spliced near midheight using a male-female structural steel tube connection. The columns were match cast and are mild steel reinforced.

Fig. 22 shows that the press area

behind home plate has special framing, namely, beams, columns, bathtub sections and hollow-core slabs. All the bays between first and third base are pie shaped. Every precast concrete member has a different length and skewed ends. The total number of precast pieces for the concourse and seating area is 3200, excluding the steps. With 1500 marks, the total piece to mark ratio for the concourse and seating is 2.1 to 1.

Lateral Load Systems

Preliminary designs for the Florida Suncoast Dome by the Engineer of Record, Geiger/KKBNA,

Fig. 22. Press area special framing.

Fig. 23a. Raker beams.

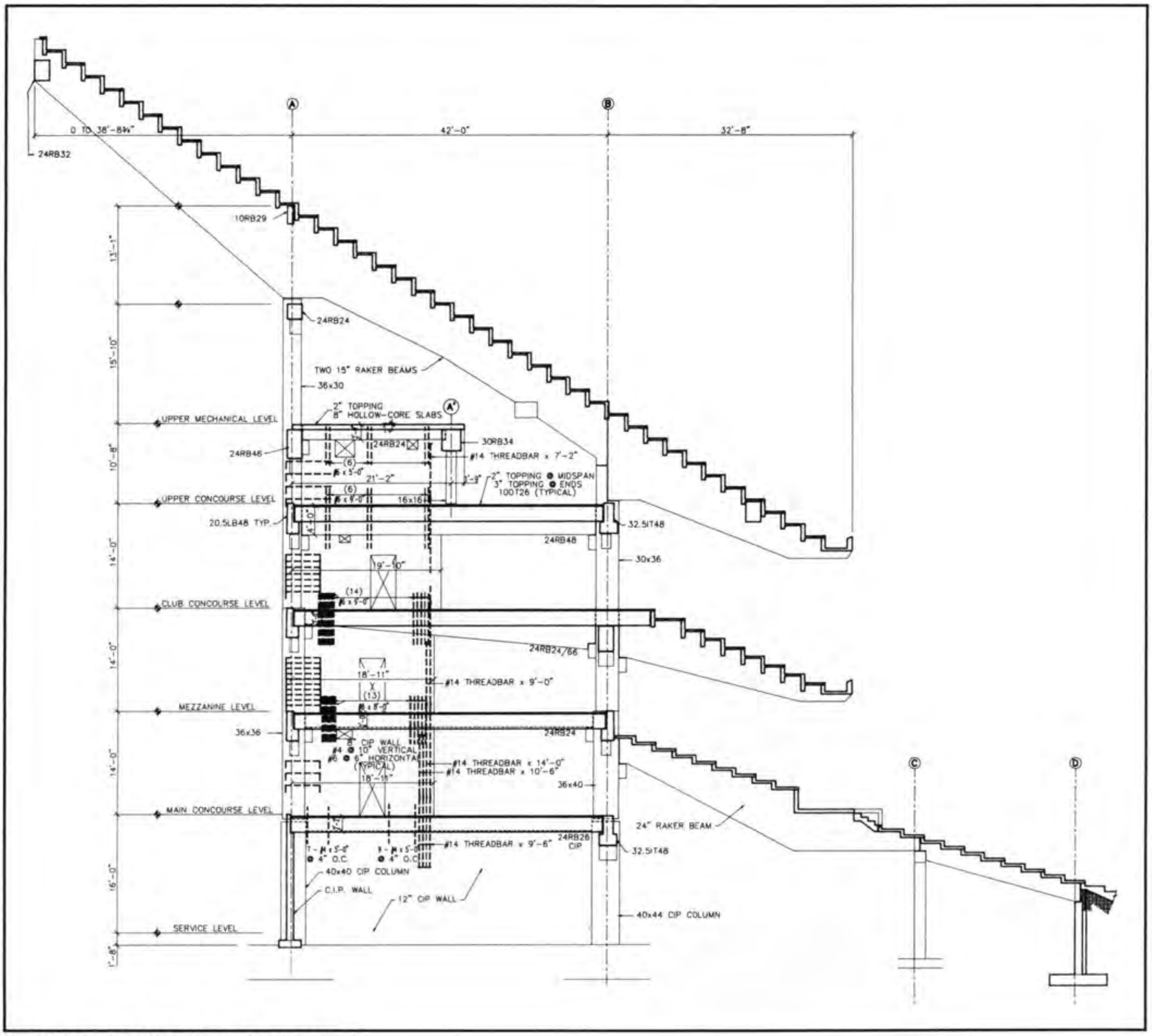

Fig. 23. Typical framing with shear wall.

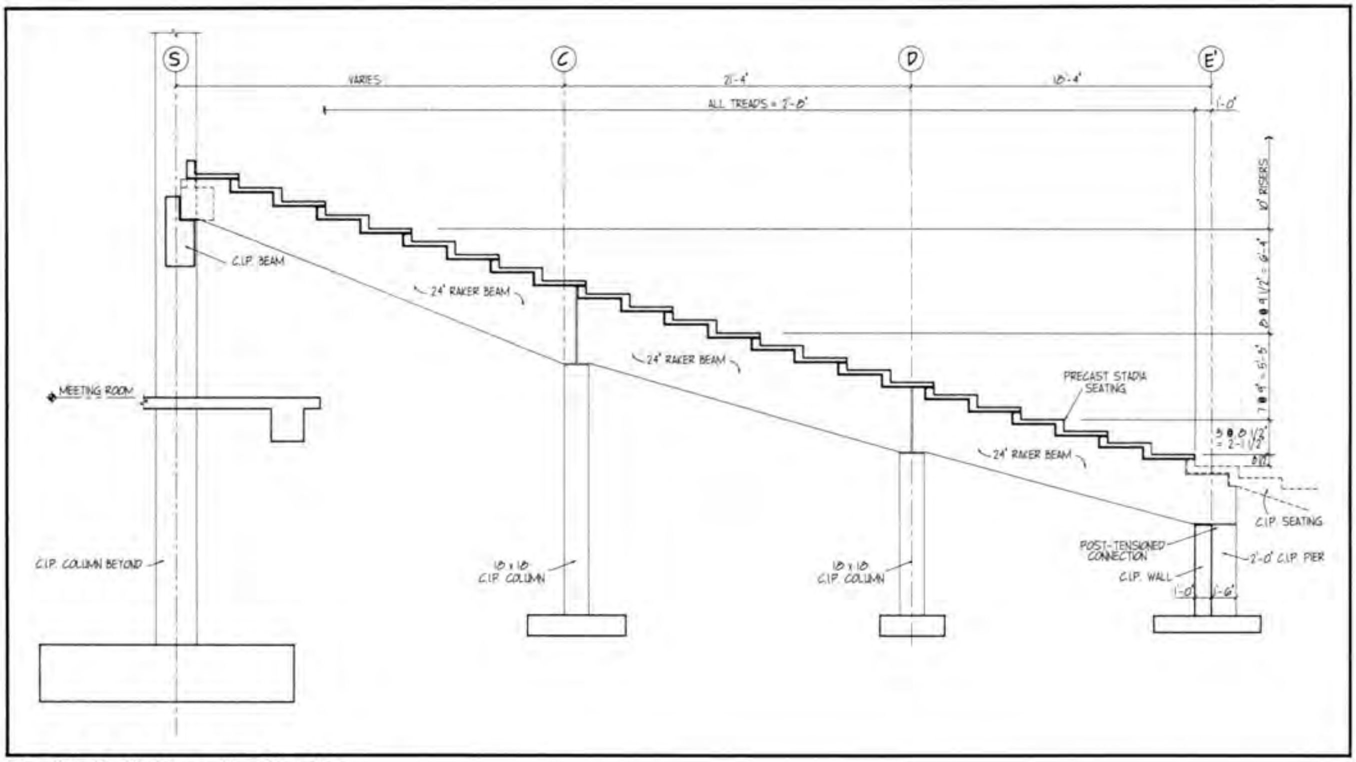

Fig. 24. Outfield seating framing.

were for a cast-in-place concrete frame with precast double tees and stadia seats. The lateral load resistance was to be a beam-column frame. When it was decided to use all precast concrete, a frame was not desirable because of the connection and volume change restraint problems.

A shear wall system was substituted for the beam-column frame to resist lateral loads. Every second or third bay has a shear wall which is about 20 ft (6.10 m) long (see Fig. 23). At the bottom story, the shear wall is 42 ft (13 m) long and connected at each end to the main columns. There is no uplift at the bottom from either direct or suction wind because of the column gravity loads.

The shear walls were cast-in-place concrete. They were formed, reinforced and poured by the precaster. The shear walls were cast from above through slotted holes in the precast filler beams. Typically, the walls are between concession areas or washroom areas and do not interfere with the architectural layout. Lateral loads perpendicular to the shear walls are resisted by concrete block infill walls.

Fig. 25. Precast stadia seating showing upper seating, club concourse seating and lower seating.

In the outfield, the architect, HOK Sports Facilities Group, could not fit in shear walls so Geiger designed the precast concrete beam and cast-in-place column system to be a moment resistant frame. POMCO and CEG revised the framing so that the only moment connection is between the low end of the beam and short cast-in-place pier. It is post-tensioned but the rest of the beams are simple span with pinned end connections (see Fig. 24).

Stadia Seating

There are three basic shapes for the stadia seats or risers. Upper seating units have horizontal tread dimensions of 2 ft 7 in. or 2 ft 8 in. (0.79 or 0.82 m) and riser dimen-

Fig. 26. Typical upper seating unit is on left. Note 5 in. (127 mm) projection below tread. Typical lower seating unit is on right.

Fig. 27. Lower seating units and raker beams. Note the slender ends of club concourse raker beams at upper left.

sions of 1 ft 7½ in., 1 ft 8 in. or 1 ft 8½ in. (0.50, 0.51 or 0.52 m). Club concourse seating units have a tread dimension of 2 ft 8 in. (0.82 m) and a riser dimension of 1 ft 2½ in. (0.37 m). Lower seating and outfield seats have 2 ft 8 in. (0.82 m) treads and riser dimensions of 9, 9½, 10 and 10½ in. (229, 241, 254 and 267 mm).

All of the various stadia seating tread and riser dimensions were determined by HOK to achieve the best sightlines. Each bay of the upper seating has a different number of seating rows, as shown in Fig. 25. Areas behind home plate and at the left and right field corners have 28 rows. They increase gradually to 43 rows behind first and third base. The top row is about 125 ft (38 m) above the playing field and 250 ft (76 m) horizontally from the closest foul line.

The club concourse seating has nine rows. The lower seating has 37 rows of stadia — the lower 17 are cast-in-place concrete, slab-on-grade; the upper 20 rows, including a special 11 ft 2 in. (3.40 m) wide unit for an aisle and handicap seating, are precast concrete.

Most of the seats were cast two rows together as shown in Fig. 26. Upper seating and club concourse stadia span 42 ft (13 m). Each vertical leg has three ½ in. (13 mm) diameter strand. Bent welded wire fabric reinforce the legs and treads. Geiger was concerned about vibrations, human reaction and dynamic loading. Their design for vibration is based on a supplement to the National Building Code of Canada. A leg height of 25 to 30 in. (635 to 762 mm) is usually required to satisfy the dynamic loading for a 42 ft (13 m) span. There is a 5 in. (127 mm) projection below the tread in order to get the required depth.

Lower seating and outfield seating span 21 ft (6.40 m) or less. The center leg has three ½ in. (13 mm) diameter strands and the other leg has two strands. The treads and legs have mesh reinforcement.

These seating sections are not symmetrical like a rectangular beam or a double tee. Standard design methods or computer programs usually do not give conservative results for unsymmetrical sections. There is, however, a good reference in the PCI JOURNAL* for the design of stadium seating.

The seating units bear directly on top of the raker beams and are connected by grouted pins as shown in Fig. 27. The holes for the pins are preformed in the seating units and field drilled into the raker beams. There are 1460 precast seating units with 525 different marks. The ratio of pieces to marks for seating units is 2.8 to 1. There are 2400 precast concrete step units on top of the seating units with 70 marks for a piece to mark ratio of 34 to 1.

Raker Beams

Raker beams are the stepped or sawtooth beams which support the seating units (see Fig. 20). The upper seating raker beams have a main span of 42 ft (13 m), a front cantilever of 33 ft (10 m), and a back cantilever which varies from 0 to 38 ft (12 m) depending on which bay is being considered. The club concourse rakers have a cantilever of 33 ft (10 m) and a back span concourse beam which anchors the cantilever. Lower seating raker beams span 41 and 21 ft (12.5 and 6.40 m). In the outfield, the raker beams have many different span lengths up to 41 ft (12.5 m).

The upper seating raker beam main span and back cantilever were cast as one piece. Sawtoothed side forms and a tilt table were used to cast these raker beams

*Kelly, John B., and Pike, Kenneth J., "Design and Production of Prestressed L-Shaped Bleacher Seat Units," PCI JOURNAL, V. 18, No. 5, September-October 1973, pp. 73-84.

Fig. 28. Reinforcing bars and post-tensioning ducts being installed for an upper seating raker beam at POMCO's jobsite plant.

at the site plant as shown in Fig. 28. Originally designed as a 30 in. (762 mm) wide section, the raker beams are two 15 in. (381 mm) wide sections side by side in order that they could be erected. The heaviest weighs 60 tons (54 t) and is 100 ft (30.5 m) long. The front cantilever had to be erected later in order that the dome roof contractor could assemble the rings on the ground and lift them into position. The upper seating rakers are post-tensioned using the VSL multi-strand system.

Geiger and HOK had specified a minimum 7 ft (2.13 m) depth for the main span of these rakers. Well into the design and shop drawing phase, they discovered a headroom problem and reduced the depth from 7 ft to 4 ft 6 in. (2.13 to 1.37 m). The front cantilever has two 31 strand tendons which are anchored in the main span as shown in Fig. 29. Also, the main span has either two 24 or two 31 strand tendons. These four tendons cross each other near the end of the main span. The high cantilever moment is also at this location. Ultimate strength capacity could be obtained with the reduced depth, but compression stresses under service load far exceeded the code maximum of 0.45 times the concrete strength.

The end of the rakers were loaded with # 11 reinforcing bars and # 5 stirrups. ACI 318, Section 18.4.3, states that permissible stresses in concrete may be exceeded if shown by test or analysis that performance will not be impaired. It was decided to do a full scale test. 100 percent of the service loads was applied to the cantilever by means of jacking prestressing strand against concrete deadman blocks on the ground.

The upper raker beam performed well under the test. There was no bulging, bursting or compression failure, and deflections were less than anticipated. All of the cantilever rakers have been erected and post-tensioned, and there have been no problems due to the high compression stresses.

Club concourse cantilever raker beams are post-tensioned through the column and anchored in a backup beam with a tapered depth. Some of the post-tensioning moment is distributed to the column above and below — the secondary moment effect. The upper seating raker has pinned connections to the columns and does not distribute moment to the columns.

There are structural steel embedments in the raker cantilever ends because of the shallow depth and the reduced section at the post-tensioned anchorage. Lower seating raker beams are mild steel reinforced and designed as simple spans.

There are 300 raker beam pieces with 130 different marks for a piece to mark ratio of 2.3 to 1.

Pedestrian Ramps

There are four pedestrian ramps for spectators to access the various

Fig. 29. Post-tensioning the precast cantilever raker beam (right) to the main span raker beam (left). Resistance to high compression stresses at this location were verified by a full scale test.

levels. They are believed to be the first all precast concrete spiral ramps constructed in the United States (see Fig. 30). Their outside diameter is 94 ft (29 m) and the inside diameter, 43 ft (13 m). The clear ramp width between spandrels is 24 ft (7.31 m).

The ramp columns are 22 x 24 in. (559 x 610 mm) half rounds. Each column consists of two pieces spliced near midheight. Their splice is done with tubes similar to the concourse columns.

Ramp spandrels are curved and sloped. They are 5 ft (1.52 m) deep and 8 in. (203 mm) thick with a 6 in. wide x 8 in. deep (152 x 203 mm) ledge to support the double tees. The spandrels are mild steel reinforced.

Fig. 30. All precast concrete spiral pedestrian ramp.

Ramp double tees are 14 in. (356 mm) deep with a 2 in. (51 mm) flange and span 24 ft (7.32 m) (see Fig. 31). Stems are 5 ft (1.52 m) apart and are dapped at each end. The tees are pie shaped with a width of 12 ft (3.66 m) at the outer circle and 5 ft 10 in. (1.78 m) at the inner circle. The double tees were cast with a 3 in. (76 mm) warp to fit the geometry of the inside and outside spandrels. They are prestressed with three ½ in. (13 mm) strand in each stem.

One of the unique features of these ramps is that their stability and resistance to wind loads is not from shear walls or typical beam-column frame action. The ramp circular and sloping geometry, together with torsion resistant connections between the spandrels and columns, form the lateral load resistance system. A three-dimensional space frame was modeled on the computer program STADD to analyze the ramps.

The spandrel to column connection resists the double tee eccentricity torsion as well as helping to provide the frame resistance for lateral loads. Columns are pocketed and the spandrels have sleeves for coil rod connections to the columns similar to typical parking garage connections as shown in Fig. 32.

There is a total of 1050 precast concrete pieces in the four ramps. There are 180 different marks which results in a ratio of pieces to marks of 5.8 to 1 for the ramps.

Fig. 33 shows the Florida Suncoast Dome near completion.

Altogether, more than 7000 precast components weighing over 50,000 tons (45350 t), were used in the structure.

The total cost of the entire project was about $100 million.

The facility is scheduled for completion in April 1990.

Fig. 31. Ramp framing includes warped double tees, spandrel ledger beams and half round columns.

Fig. 32. Typical ramp connection has pocketed columns and coil rods.

Fig. 33. Florida Suncoast Dome in September 1989. Administration building is between two left ramps and mechanical/truck dock area is between two right ramps.

Credits

Owner: City of St. Petersburg, Pinellas County, Pinellas Sports Authority.

Architect: HOK Sports Facilities Group, Kansas City, Missouri.

Structural Engineer: Geiger/KKBNA PC, New York, New York.

General Contractor: Huber, Hunt and Nichols, Inc., Indianapolis, Indiana.

Precast Prestressed Concrete Manufacturer: POMCO Associates, Inc., Palmetto, Florida.

Precast Specialty Engineer: The Consulting Engineers Group, Inc., Mount Prospect, Illinois, and San Antonio, Texas.