Design-Construction of Olympic Stadium in Honduras

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An all-precast/prestressed concrete system was used to build the new Olympic Stadium Project in San Pedro Sula, Honduras. Designed for the 1997 Central American Olympic Games, the stadium (mainly intended for soccer but can accommodate other sporting events) has a seating capacity of 40,000 together with 250 skyboxes. More than 7000 precast components were used which included single columns, A-frames, H-frames, primary beams, cantilever beams, single risers, double tees and skyboxes. This article presents the conceptual design, geometric layout, design features, load testing and erection highlights of the project. CONHSA-PAYHSA Group was responsible for the entire design and construction of the project.

The new Olympic Stadium Project is an all-precast prestressed concrete sports arena. Built specifically for the 1997 Central American Olympic Games, the stadium is situated in the south suburbs of San Pedro Sula, Honduras. The stadium is mainly intended for soccer but can accommodate other sporting events.

The stadium has 40,000 seats and 250 skyboxes. The arena can accommodate 35,000 spectators in the grandstands which are located from ground level to 12 m (40 ft) high. Another 5000 spectators can be seated comfortably in the skyboxes, which are situated in the second and third stories on opposite sides of the stadium (see Figs. 1 and 2). The stadium seating is designed so that all spectators have a clear view of the sporting activities.

More than 7000 precast/prestressed concrete components were used to build the stadium. A variety of shapes and sizes were fabricated including single columns, A-frames, H-frames, single risers, primary beams, cantilever beams, double tees and skyboxes.

Construction of the project began in January 1996. The stadium was completed in November 1997, in time for the Central American Olympic Games.

This article presents the conceptual design, geometric layout, structural design features, load testing and erection highlights of the project. Particular emphasis is placed on the precast concrete aspects of the job.

Fig. 1. Aerial view of mid-stage erection of The Olympic Stadium Project in San Pedro Sula, Honduras.

DESIGN OPTIONS

The stadium project was opened to the public in a design-build competition. Initially, a simple general plan and main section of the stadium were offered by the Olympic Organizing Committee (see Fig. 3).

The proposed scheme comprised an oval-shaped plan composed of four segments of a circle with different diameters. Access to the main part of the stadium was through six axes and six levels.

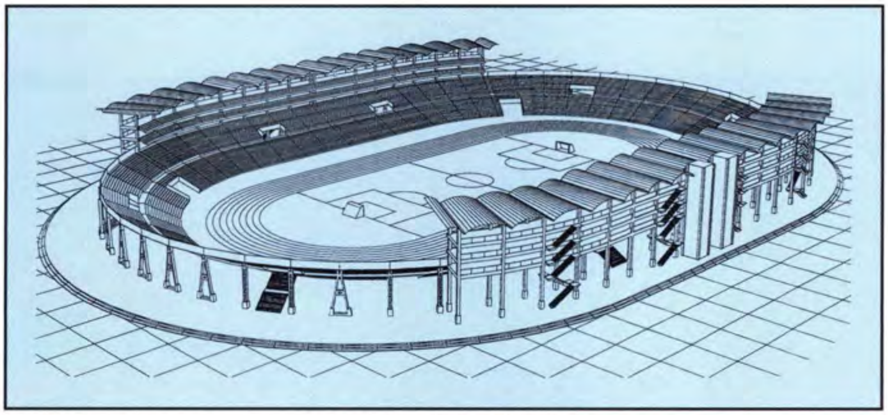

Fig. 2. Perspective drawing of The Olympic Stadium Project.

During the competition phase of the project, four design options were offered including cast-in-place concrete, precast concrete, and structural steel. However, another three structural grid patterns were completely controlled by the original plan and section of the predetermined size and seating layout. In that case, the area and dimensions of the stadium were rather irregular in plan, with very little symmetry or opportunity for repetition of elements. This lack of regularity did not lend itself easily to re-use of cast-in-place or precast concrete formwork.

After studying several options, the authors concluded that the most efficient and economical scheme would be a completely precast/prestressed concrete structural system. They then devised several rough sketches and preliminary designs to arrive at a

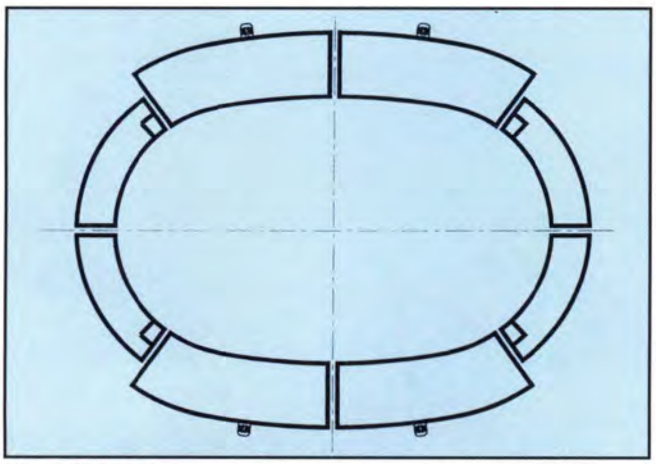

Fig. 3. Original general plan of predetermined design for design-build competition.

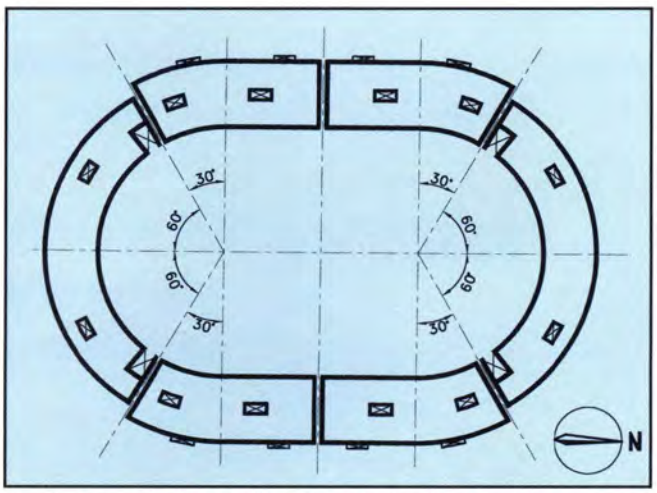

Fig. 4. General plan of The Olympic Stadium Project.

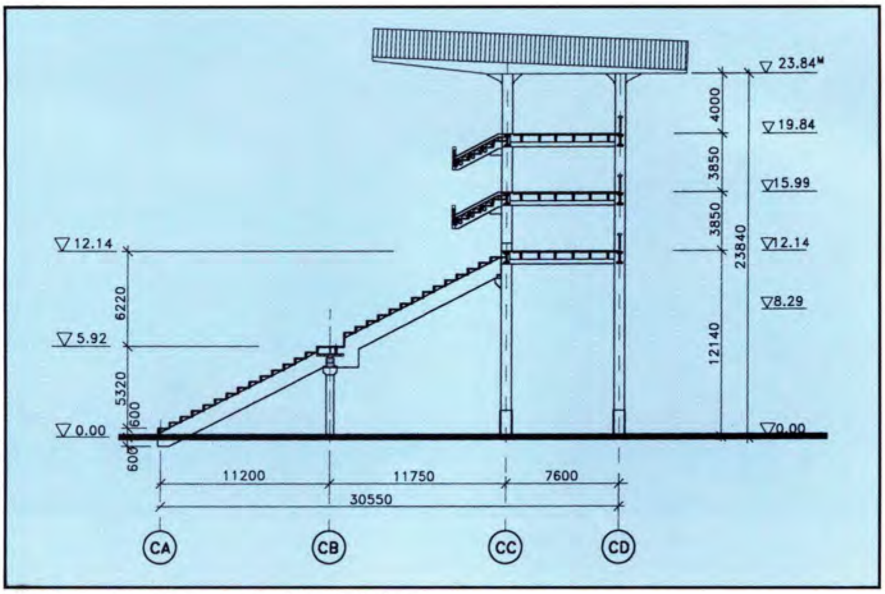

Fig. 5. Structural frame section of skybox.

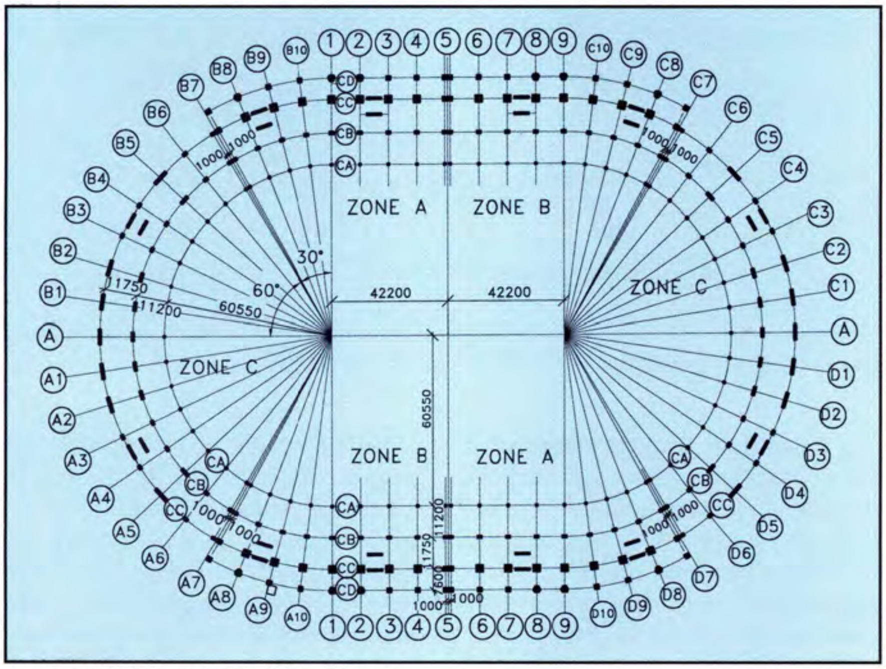

Fig. 6. Distribution plan of foundation.

modified final scheme (see Figs. 4, 5, and 6).

The geometric plan was composed of two segments of a circle with one diameter and two straight lines. In actuality, only a very slight difference existed between the two segments. When a cost analysis of the various systems was made, it was found that the precast solution was about 30 percent less expensive than the cost of the other three schemes. Also, it was shown that the precast system had a shorter construction period.

In the end, the owner selected the authors' modified preliminary design because the precast system offered the best advantages in terms of quality, cost and scheduling.

GENERAL LAYOUT

The stadium structure is partitioned into six blocks, separated by six openings which are basically expansion joints 2 m (6 ft 7 in.) wide. Fig. 6 shows a typical expansion joint for the given structure. The longitudinal length of both sides is 87.6 m (287 ft) and the maximum length of the curved parts is 172.4 m (566 ft). The maximum space between the two structure frames is governed by the length of the span of a single riser element.

The grandstand is supported by two primary beams having a slope of about 1 to 2.

The objective of the design approach was to create the largest possible span while considering component weight and hauling limitations. This approach produced the most efficient and economical design.

Fig. 7. Erection of primary beams.

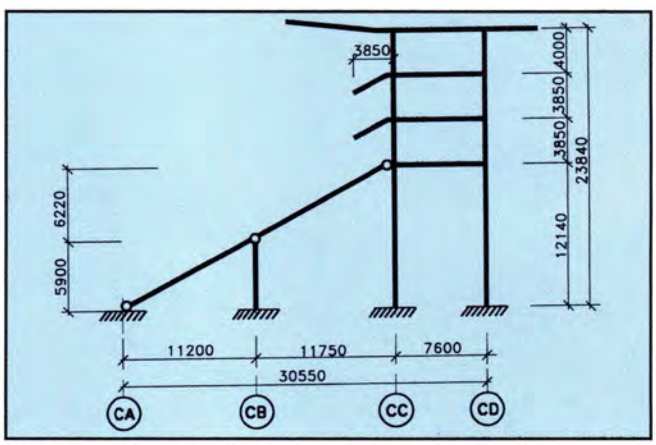

Fig. 8. Analytical model for structural frame of skybox.

Fig. 9. The steel cross bracing and the staircases.

STRUCTURAL FRAME DESIGN

The skybox frames take up four levels. Each frame comprises two columns with rigid connections at the foundations and in the next three levels. The primary beams, which carry the main grandstand in a sloping axis, are hinge-connected to the skybox frames and footings at both ends and between each other.

There is a large variety of special beams (see Figs. 7 and 8). The partial horizontal load acting on the skybox frame is transferred by the sloped beam. In the other direction, horizontal stability in the longitudinal structural frame is provided by the steel cross bracing and the staircases (see Fig. 9).

The skybox structural frame is composed of two rectangular columns with one story height in the first and top story. The column of one story height for the top story has a hollow core. Thus, the weight of a column segment is reduced from 4.8 to 3.3 t (5.3 to 3.6 tons)(see Fig. 10).

The columns are located on Axes CB and CC of the curved part, which is composed of two precast components. The columns are integrally cast with the two parts in the precasting plant or construction yard. Most column shapes here are of an H form. The partial shape of the columns on Axis CC of the curved part is of an A form. The A-shaped column is comparable to a monolithically cast column and, therefore, can transmit horizontal in-place forces to the foundations (see Figs. 11, 12, and 13).

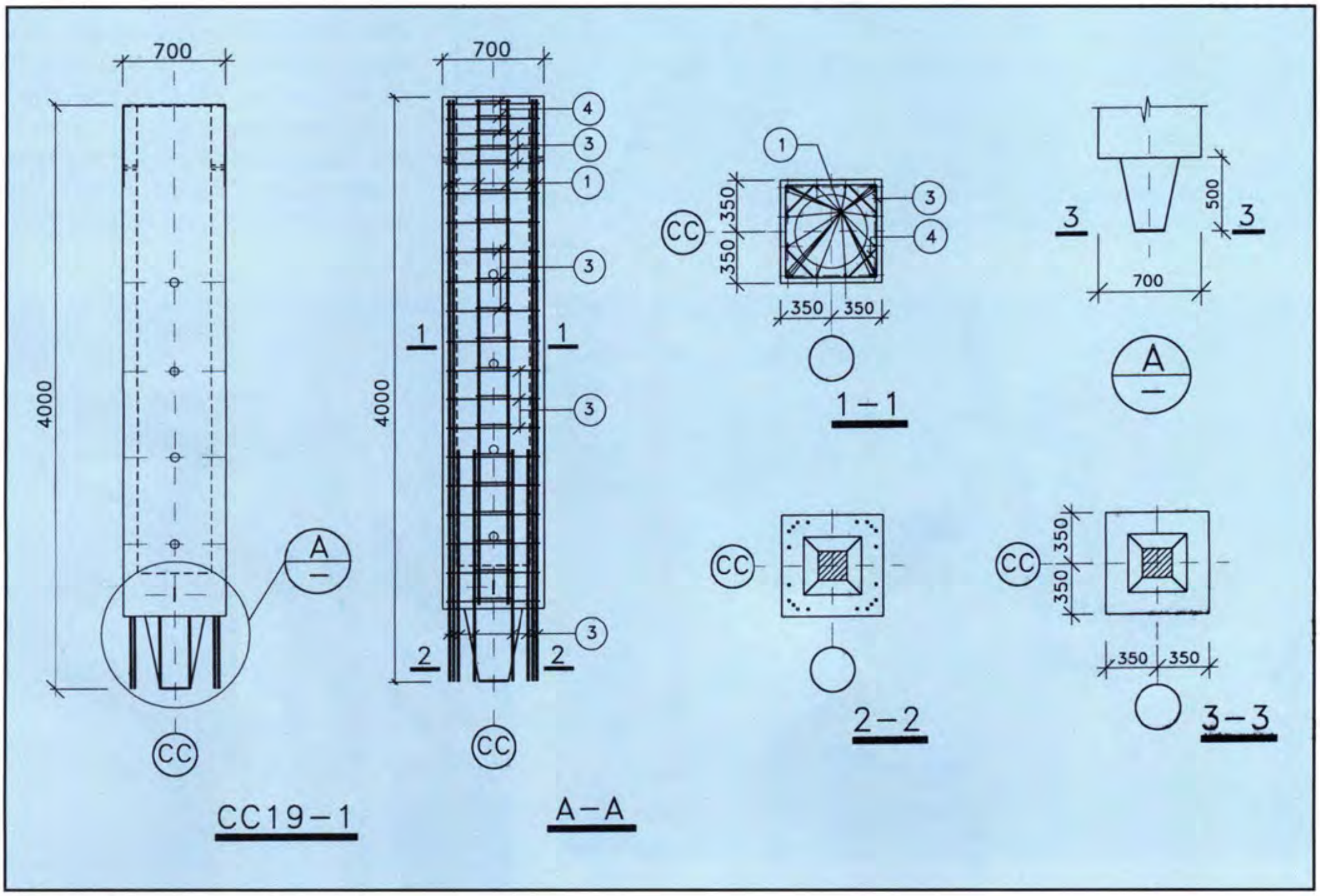

Fig. 10. Column segment element for top stories.

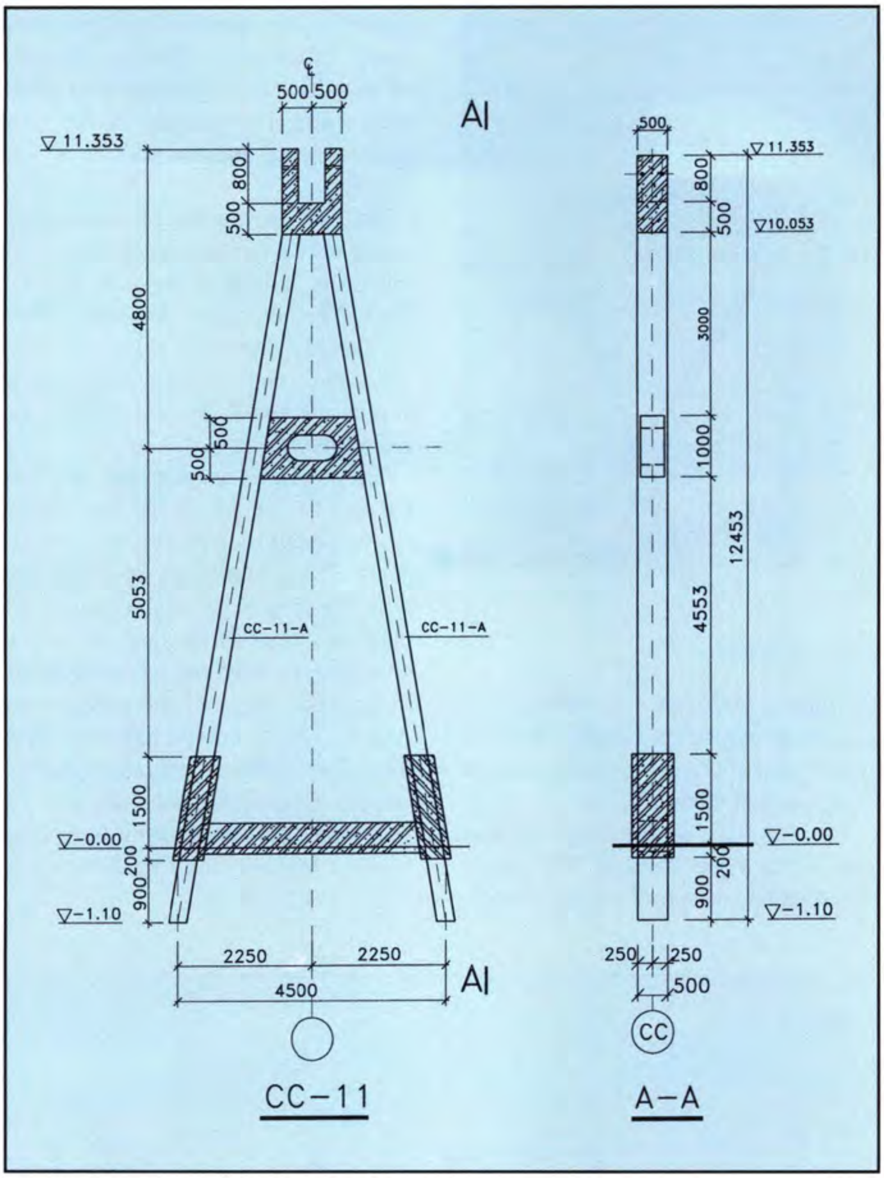

Fig. 11. Elevation and section of A-shape column.

The skybox floor structures consist of double tees that are mostly 460 mm (18 in.) deep. The floors are finished with a structural topping 50 mm (2 in.) deep. They are supported with a partially prestressed main beam having an inverted T shape. These beams are single span and rigidly connected to the columns by specially developed hidden connection details. To strengthen the structural frame in the other direction, I-shaped beams are used with a span ranging from 10.3 to 11.6 m (33 ft 10 in. to 38 ft 1 in.).

CANTILEVER BEAM DESIGN

A total of 72 precast cantilever beams are used on the second and third stories for the riser seats of the skyboxes. These beams are partially post-tensioned to prevent cracking and to increase their durability. The standard dimensions of the section are shown in Figs. 14 and 15.

A composite section using an inverted T-shaped beam with cast-in-place topping was adopted. The strength of the topping was 55 MPa (8000 psi). Cantilever beams are connected to the bottom of the end with the corbel of the column by four bolts.

Debonded post-tensioning tendons were assembled at the job site from coated strand which was protected by grease and four layers of plastic sheet.

Fig. 12. A-shape column in position as primary beam is being lowered to rest on top of column.

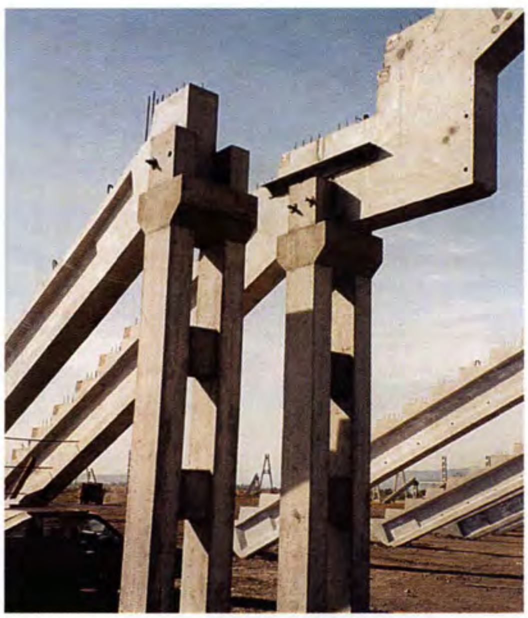

Fig. 13. H-shape column in final position.

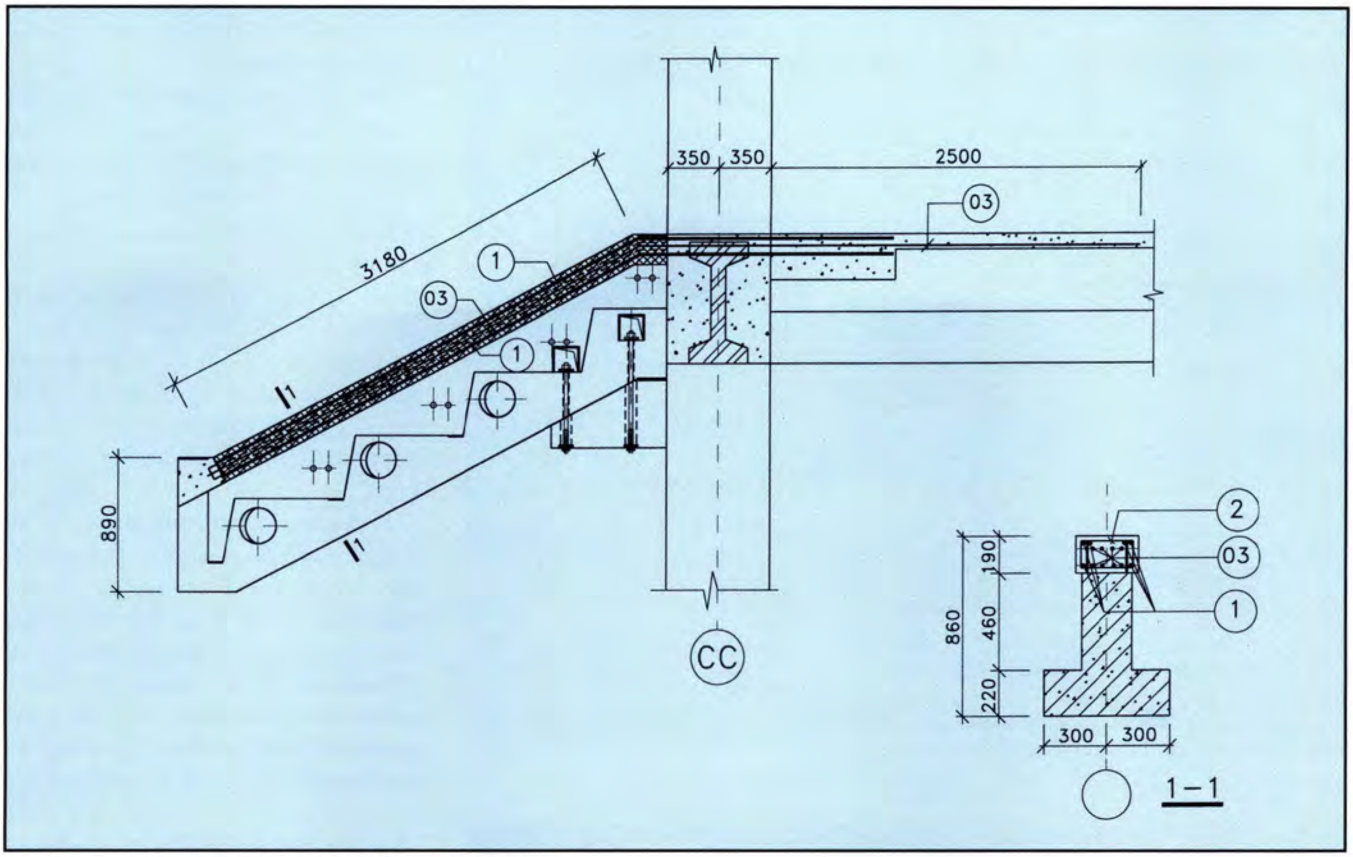

Fig. 14. Elevation and section through cantilever beam.

Dead-end anchorages were installed in the composite topping concrete of the main beam of the skybox frame. After the tendons were tensioned and the strand ends burned off, a Dywidag wedge anchor system was used to secure the tendons.

SINGLE RISER DESIGN

Precast, prestressed single seating risers were used. Their size and shape depended on the steel forms from which the double tees were fabricated. After modification, a 150 m (492 ft) long double tee steel form was adapted to fabricate the seat risers. This scheme produced economies in the precasting process and production schedule.

A total of 1632 seating riser components, totaling 17000 m (55,774 ft) in length, were used in the grandstand. The riser components [7.28 to 11.88 m (24 to 39 ft) long, 615 to 935 mm (24 to 37 in.) wide, and 430 mm (17 in.) deep] are supported by primary beams.

Some 256 risers, with about a 2600 m (8530 ft) total length, were placed in the second and third levels to seat spectators in the private skyboxes. The riser components [9.898 to 10.204 m (32 to 33 ft) long, 792 to 935 mm (31 to 37 in.) wide, and 470 mm (18.5 in.) deep] are supported by inverted T-shaped cantilever beams. To distribute the load, a flange bearing plate, 50 mm (2 in.) thick, is used. In addition, a

Fig. 15. Erection of cantilever beam.

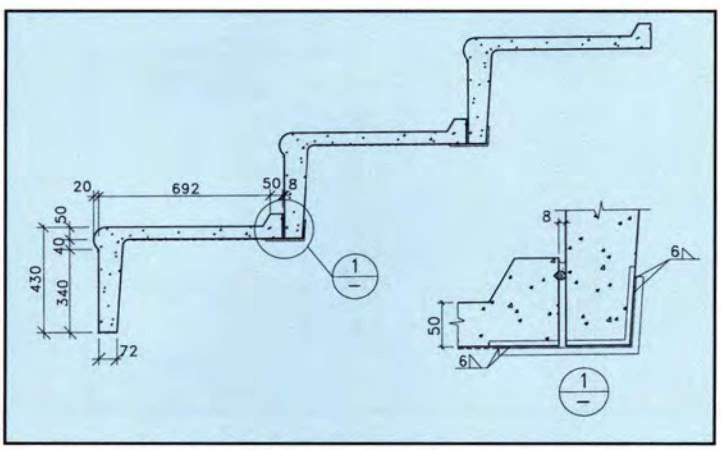

Fig. 16. Cross section of typical risers and connection between each component.

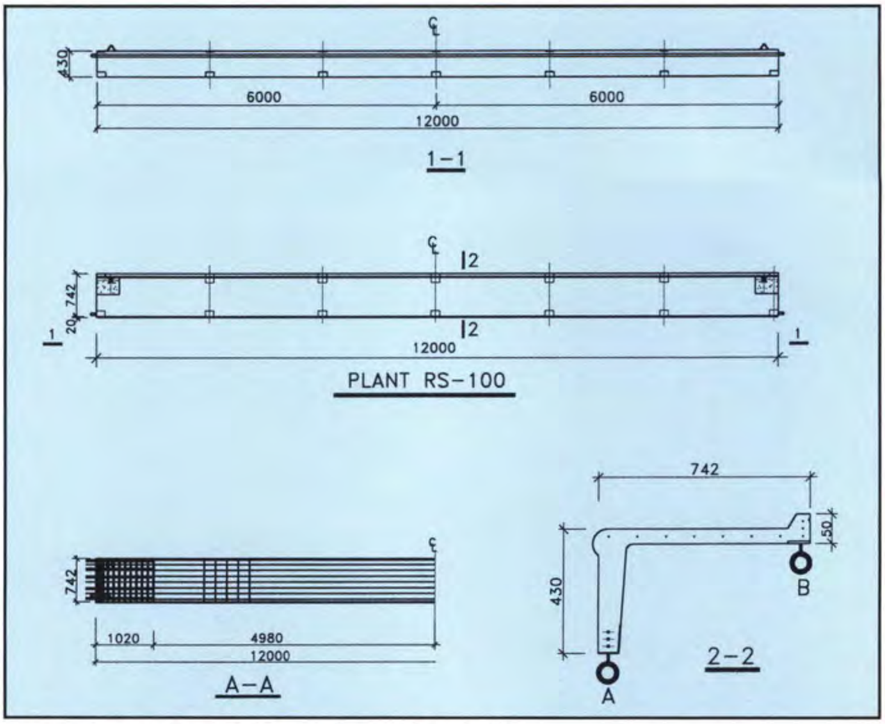

Fig. 17. Longitudinal views and cross section of risers.


Fig. 18. Arrival of single riser at project site.

17.5 mm ($^2/_3$ in.) thick protective coating is employed.

The maximum length of the single riser is dependent on the minimum span-to-depth ratios. The minimum ratio of 1/30 is based on practical considerations taking into account the architecture and structure of the stadium. Indeed, as far as the design is concerned, the maximum 12.30 m (40 ft) length is 1/28 of the span-to-depth ratio at the curved parts and the maximum 11.6 m (38 ft) is 1/27 of the span-to-depth ratio at the skybox parts.

In precast construction, particular attention must be given to tolerances. In this structure, an 8 mm (0.31 in.) space was allowed between the two single riser components. This gap not only facilitates erection of the precast components but also permits fill-in with waterproof material at a later stage of construction (see Figs. 16 and 17).

LOAD TESTING OF RISERS

Because the risers are unsymmetrical, the connections at the longitudinal joints are designed to transfer large shear forces. Welded joints, with a 2.0 m (6 ft 6 in.) spacing, are visible from below. Load tests of risers were necessary to validate the strengths of full-scale single risers and triple risers.

Single Risers

During storage, transportation and erection, the riser is normally supported at both ends (see Fig. 18). How-

TABLE. 1

	TEST LOADING Q=KG/M²	MIDSPAN DEFORMATION MM A	B	DESCRIPTION
1	100	3	3	
2	200	11	13	
3	300	17	21	
4	400	23	28	
5	500	31	38	A FIRST CRACK IN THE MIDSPAN
6	600	41	52	CRACK WIDTH= 0.1mm IN THE MIDSPAN
7	700	64	89	A FIRST CRACK IN FLANGE OF BOTH ENDS
8	800			CRACK DEVELOPING
9	900			COLLAPSE

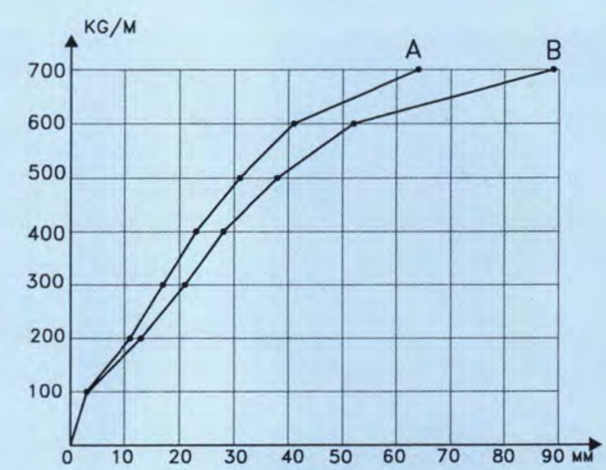

Fig. 19. Load-deflection relations of single riser test.

Fig. 20. Load testing of single riser specimen.

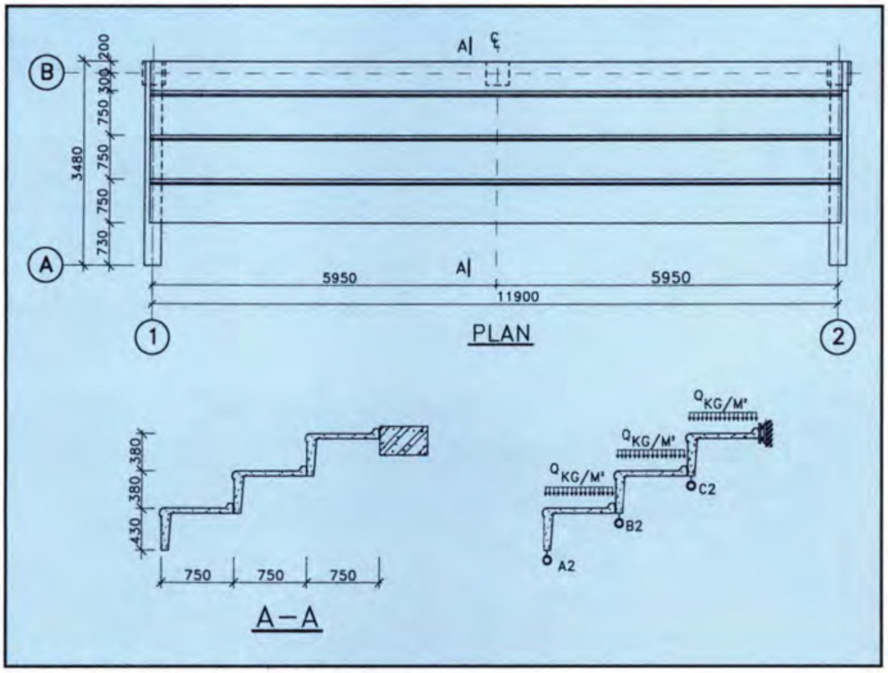

Fig. 21. Plan and sections of triple risers.

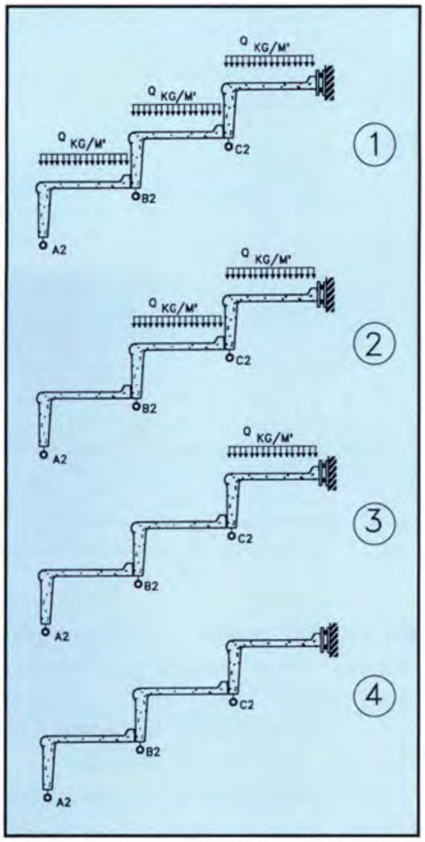

Fig. 22. Unloading step of triple riser testing.

ever, during erection and even after the precast component is permanently placed, it may be subjected to heavier loads. Therefore, it was felt prudent to analyze and evaluate the behavior of the riser element during full load testing.

Precast concrete paving stones were used to provide the load testing. The test specimen was placed on two small columns.

Fig. 19 provides a schematic of the test setup. The test procedure and loading stages are shown in Table 1. Visible cracks initiated at midspan above a load of 500 kg/m^2 (102 lbs per sq ft). When the load was increased beyond 700 kg/m^2 (143 lbs per sq ft), visible cracks initiated in the flange at both ends and cracks developed quickly along a 45-degree angle from each end.

Ultimate collapse occurred at a total load of 1150 kg/m^2 (236 lbs per sq ft), which includes a test load and dead load of 900 and 250 kg/m^2 (184 and 51 lbs per sq ft), respectively. A typical value of measured deflection for 500 kg/m^2 (102 lbs per sq ft) was 31 mm (1.2 in.) at midspan for a deflection-to-span ratio of 1/350 (see Figs. 19 and 20).

Triple Risers

In this test program, the riser seat loading on the grandstand was simulated by the triple riser loading test. For the full-scale, 12 m (3 ft 8 in.) span, the simulated loads were validated twice. In the first test, a 600 kg/m^2 (123 lbs per sq ft) load was used whereas in the second test, 100 persons pressed together standing on the triple riser were the load. The triple risers were connected to each other when the test specimens were placed on the inclined beams, which had the same sloped axis as the grandstand.

Dead load testing — The test procedure and loading stages are shown in Table 2 and Fig. 23. A total of 2100 paving stone units, weighing 21.6 t (24 tons), were placed on top of the triple risers. The maximum test load was 600 kg/m^2 (123 lbs per sq ft). During the 500 kg/m^2 (102 lbs per sq ft) loading stage, a first visible crack was initiated in the rib of the riser at midspan.

During the 600 kg/m^2 (123 lbs per sq ft) loading stage, a first visible crack was initiated in the flange of the riser at the end, which was visible from below. The crack widths were about 0.08 mm (0.003 in.) or less. The measured deflection for the 600 kg/m^2 (123 lbs per sq ft) test loading was 25 mm (1 in.) with a deflection-to-span ratio of 1/480. The specimen did not show any failure at any of the simulated test loads.

Fig. 22 shows the behavior of the specimen during the unloading stage. Note that the primary objective of the

TABLE. 2

	TEST LOADING Q=KG/M²	DEFORMATION MM A2	B2	C2	DESCRIPTION
1	0	0	0	0	
2	100	2.25	3.00	2.50	
3	200	5.75	6.50	5.00	
4	300	9.00	10.00	8.50	
5	400	13.50	15.00	12.00	
6	500	19.00	20.00	15.50	A FIRST CRACK IN THE MIDSPAN
7	600	23.00	25.00	19.50	A FIRST CRACK IN FLANGE OF BOTH ENDS

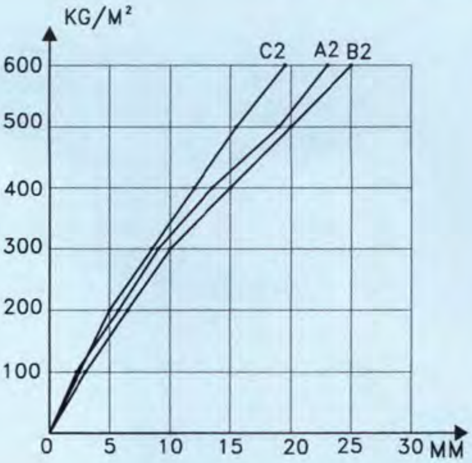

Fig. 23. Load-deflection plots of triple riser test.

test program was to evaluate the performance of the welded joint connection along the longitudinal joint under shear loading during the unloading stage (see Figs. 21, 22 and 23).

Live load testing — A total of 100 persons standing together on a 27 m² (2.9 sq ft) area of riser simulated the load of the spectators. The total weight of the people amounted to 6660 kg (14,680 lbs). Line loads of 2.8 persons were checked to be equivalent to 187 kg (412 lbs) on the single riser. A maximum simulated load of 245 kg/m² (50 lbs per sq ft) was computed.

The live load persons test was designed to be as realistic as possible in order to obtain monolithic action under service loads. It was concluded that both the dead load and live load persons tests confirmed that the design assumptions were satisfactory (see Fig. 24).

Fig. 24. Testing of triple riser with about 100 persons standing together.

CONNECTIONS CONSIDERATIONS

The design of the connections between each of the precast components was one of the most important design considerations in the stadium project. The main objective was to ensure that the designated connection would perform as intended. This was done by examining the structural behavior of various types of joint configuration.

Riser Element Connections

Connections between risers are shown in Fig. 16. The connection details at the support area depend on being both welded to each other in the bars of the flange, then pouring concrete and welding between the sloped beam with the rib of the riser element.

The support length of the riser on the sloped beam is designed to be 100 mm (4 in.) in both the rib and the flange. In some cases, the shorter support length was reduced to a minimum

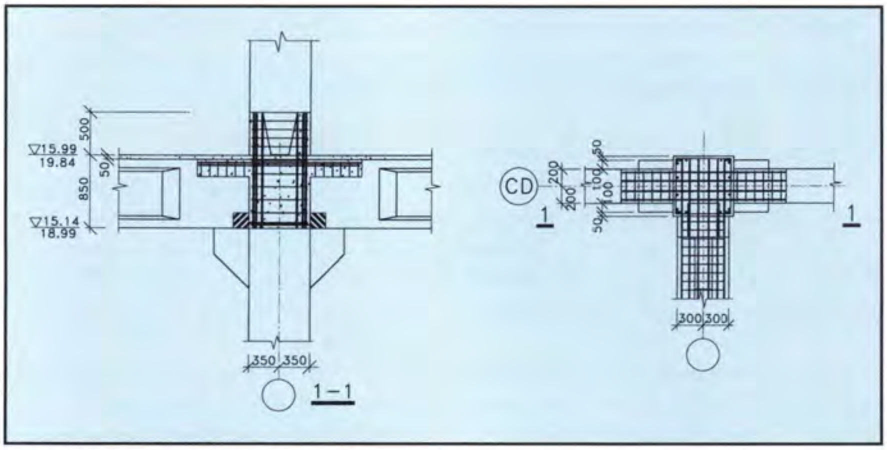

Fig. 25. Beam-to-column connection.

of 80 mm (3 in.) in order to accommodate the required tolerance. Those tolerances had to be maintained on both the length of the riser and the position of the sloped beam.

Double tee element connection — The double tee is supported on two ribs and connected by welding to the inverted T-shaped beam. The reinforcing steel of each element is welded to each other and cast-in-place concrete is poured around the joint to improve the stability of the connection.

Skybox structure frame connection — The structure frame joints are designed to ensure that an adequate transfer of horizontal and vertical loads and moments occurs between the adjacent components. The weld is formed between fully anchored mild steel reinforcement. After welding, concrete is poured around the reinforcement (see Figs. 25, 26, and 27). The compressive strength of the concrete at the joints was 55 MPa (8000 psi).

Fig. 26. Connection at frame joints between columns using vertical reinforcement.

Fig. 27. Column, riser and cantilever beam on floor.

CONCLUSIONS

Based on the experiences gained on this project, the following conclusions can be made:

1. The precast/prestressed concrete system selected was the ideal choice in successfully completing this project on schedule and within budget.

2. In selecting the appropriate structural system, it is important to realize that there are fundamental differences between a cast-in-place and a precast system that can influence the general layout, architecture, design criteria and construction of the project.

3. In a precast system, connections play an extremely important role in the performance of the structure and therefore must be designed very carefully.

4. The test results of the full-scale riser loading tests confirmed that the design details were adequate for this system.

5. The tolerances for the 1888 riser components used in this project were within acceptable limits.

6. The design and construction methods used in this project proved successful and can be easily applied to other stadium projects.

7. The ultimate goal of the authors is to prepare specific guidelines for the design and construction of stadiums in which the inherent advantages of precast and prestressed concrete are fully exploited.

8. Lastly, the people of San Pedro Sula have themselves a first class Olympic stadium that is very functional, strong, durable and aesthetically pleasing.

ACKNOWLEDGMENT

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