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Renovation of the Santiago Bernabéu Stadium. Live with the past: evaluation and reinforcement of the original structure

José Luis Martínez Martínez*, José Antonio Martín-Caro Álamo

Ines Ingenieros, Núñez de Balboa 120, Madrid, Spain
*Corresponding Author:
Email address: jlmm@inesingenieros.com

Abstract: This article presents the process of evaluation and reinforcement of the original structure of the Santiago Bernabéu stadium, adapting the building to current requirements. The evolution of the construction phases is detailed, from the original construction in 1947 to the recent interventions, placing emphasis on the structural reinforcements implemented. The pillars, ribs and slabs were intervened using techniques such as perimeter screed and reinforcement with microconcrete, as well as reinforcement with carbon fibers (FRP) and metallic solutions. These actions made it possible to improve the safety, rigidity and durability of the stadium, preparing it for the demands of its modern use.

Key words: structural strengthening; Santiago Bernabéu stadium; structural rehabilitation; micro-concrete; carbon fibers(FRP); columns; railway tunnel; foundation underpinning

1 Brief overview of the stadium's construction phases

Visitors to the stadium can hardly tell when they are walking on an 80-year-old structure and when on a new one. In contrast, the designer clearly recognizes this difference when applying the requirements of a modern, top-tier stadium and current standards to a structure like the Bernabéu, the result of successive construction phases and expansions.

1.1 1947–1953

The original construction dates back to 1947, and the first expansion took place as early as 1953 [1]. The first Bernabéu was designed for 70,000 spectators, occupies an area of 34,000 m², and already exhibits a characteristic that has remained unchanged over the years: the distinctiveness of the east stand, which was completed with a different section and design six years after the initial construction.

The playing field and the lower stands, made of mass concrete, are buried in the ground, set about 8 m below street level. The cross-section (see Figure 1) features four rows of columns founded on mass concrete piles ("pilarotes") with widened bases. The columns have a constant square cross-section throughout their height (sides ranging from 0.40 to 0.55 m). The spans vary between 4.50 and 5.50 m. The structure consists of two tiers, the first amphitheater cantilevered over the lower tier and the second tier above it and set back toward the exterior. These reinforced concrete porticos that form the cross-section are spaced 7.00 m apart and consist of beams measuring 0.30 x 0.55 m (width x depth). In the longitudinal direction, the beams measure 0.30 x 0.65 m and feature half-lap joints at one-quarter of the span every 3, 4, or 5 bays, simulating continuous flexural behavior while maintaining very moderate expansion lengths. The four floor levels are constructed with concrete slabs supported on the 5 x 7 m perimeter formed by the aforementioned portal and longitudinal beams. This box-shaped support allows for bending in two directions and a depth of only 120 mm for the slabs, which replicate the longitudinal joints using dowels. The bleachers are constructed of reinforced concrete, utilizing the rear riser with a depth of 0.57 metres [2].

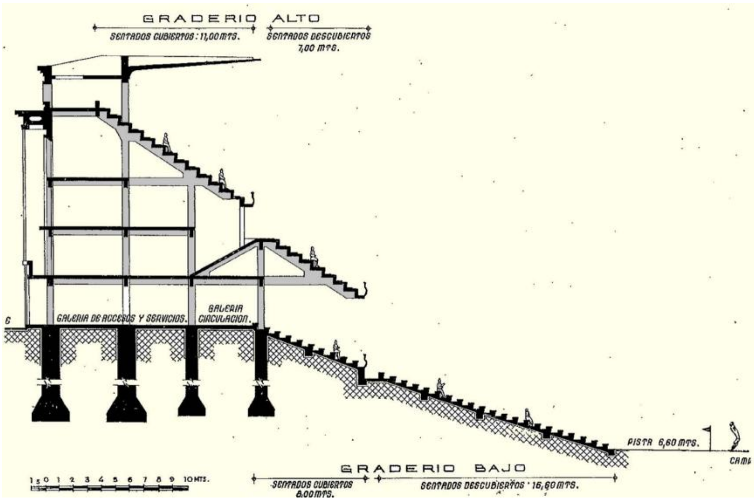

Figure 1. Typical cross-section on the west side of the original stadium (1944–1947). The upper canopy, with a 10-meter overhang, was never built

A unique feature is the presence of the ‘Railway Link Tunnel,' built before the stadium, although it did not enter service until much later. It crosses the field from south to north on its eastern side and, due to the lowered level of the playing field, its keystone is less than one meter from the surface. In several areas of the backstands, bridge structures were built over the tunnel [3].

The design loads were 4.00 kN/m² for the floor slabs and 6.00 kN/m² for the bleachers. The latter value is, fortunately, higher than the current public access load and is due to the fact that almost the entire audience was standing. The concrete code [4] and the text by Carlos Fernández Casado, the structural engineer [5], provide guidance on the materials used: concrete comparable to HA-15 and steel with a yield strength of 240 MPa (Table 1).

Table 1. Concretes and reinforcing steels commonly used in 1944 and their characteristics

Concretes				
Type	Cement [kg/m³]	Water [l/m³]	w/c Ratio [-]	f28 [MPa]
A	400	210	0.53	15
B	350	205	0.59	13
C	250	200	0.80	8.5
D	200	193	0.97	6.0

Reinforcing Steels			
Type	fy [MPa]	fu [MPa]	Elongation [%]
Regular	24	36	23
Special	36	50	18

The calculation of allowable stresses used a safety factor of 2 for steel and 3 for concrete in bending and compression. Shear resistance was still a poorly understood phenomenon and was addressed by increasing the number of positive reinforcement bars as they approached the support zone. The bars were smooth, and anchoring was achieved using hooks. The usual diameters [mm] were 5, 6, 7, 8, 10, 12, 14, 16, 18, 20, 25, 30, and 35 [4]. This wide range of diameters complicates the identification of bar sizes during test pits.

The project was completed leaving the east side unfinished and pending an expansion that took place in 1953 [1], which created a structure similar in typology but with more levels [7], a cantilevered facade, and a double-width bay on each floor. To maintain the dimensions of the slab frames, this required a longitudinal beam that did not rest on pillars, but rather connects it to transverse beams. This unique feature—as will be seen—led to a different type of reinforcement on the east side.

Figure 2. Appearance of the stadium after the 1982 (i) and 1994 (d) renovations

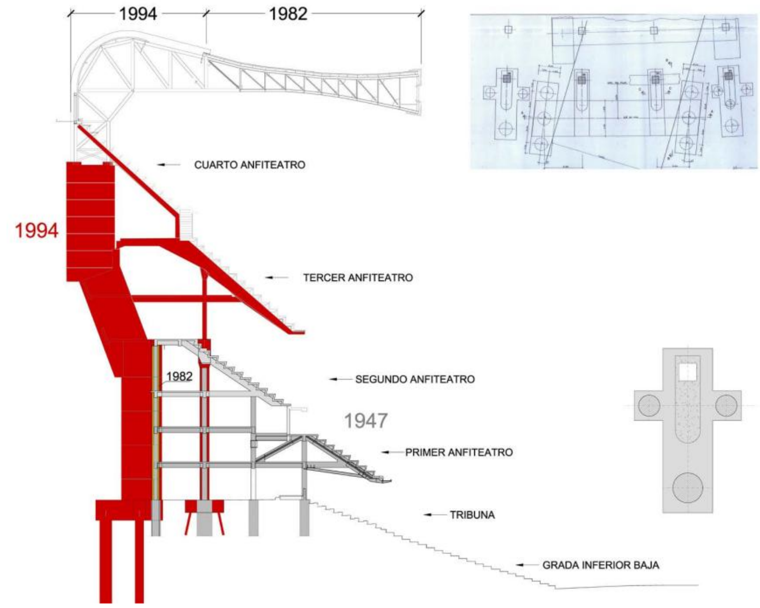

Figure 3. West cross-section and end areas after the 1994 expansion, foundation plan of a typical rib (bottom right) and reinforcement of the bridge structures over the tunnel on the south side (top right)

1.2 1982

The hosting of the World Cup in Spain led to the following renovation. The seating was converted to fixed seats, and the stadium (except for the east side) was covered with a metal lattice that was the subject of a pioneering wind tunnel study [7]. The 20-meter-long lattices rest on the pillars of the 3rd and 4th rows, compressing the former and applying tensile force to the latter, the outermost one, which was enlarged to a cross- section of 0.70 x 0.70 [m] and reinforced to withstand the tensile force, which is greatest at the top but is neutralized before reaching the foundation.

1.3 1994

The 1994 intervention [8] was a large-scale transformation with profound implications for the structure. The stadium incorporates two new amphitheaters extending vertically and outward, expands the roof, creates four access towers, and drastically alters the outer alignment of the columns (4th row) and their foundations.

Figure 3 shows the cross-section resulting from the 1994 expansion. The two new amphitheaters rest on the pillars of the first and second alignments, while the roof is raised vertically and complemented by a new curved section that allows it to be embedded in the rib of the 4th alignment. The second-row pier receives a vertical reaction from the new structure, so its cross-section is reinforced with a perimeter extension and its foundation is micro-piled.

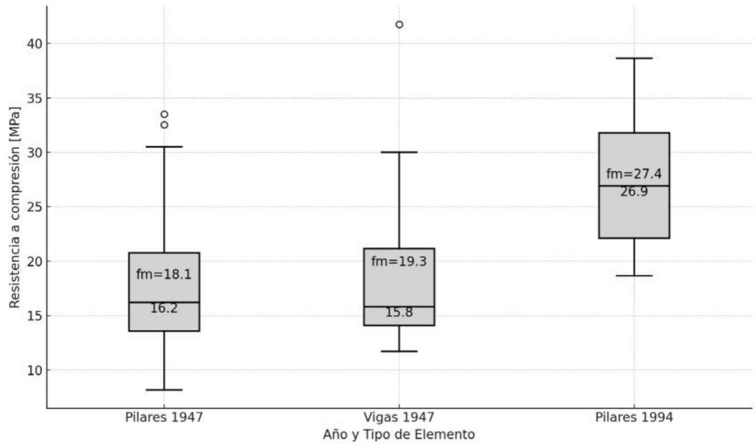

Figure 4. Distribution of compressive strength results in 100 mm diameter core samples with h/ø=2. The mean value fm and the median value are indicated

The new column in the fourth alignment, the rib, has a complex layout: vertical at its base where it incorporates the pre-existing column (originally from 1947 but reinforced in 1982), then angled to make room for the upper amphitheater, and finally vertical where it meets the roof. We believe the designer's intention was to separate this massive pier from the original structure, as a panel of expanded polystyrene is inserted between the existing column and the new concrete. The break at the height of the third level is a source of problems when load increases occur, as will be seen later. The most uncertain aspect of this design is the foundation of the ribs. Initially, a foundation with four piles was designed; however, these were positioned asymmetrically on the plan relative to the pillar, offset toward the exterior. This positioning was necessary because it was not possible to install a pile cap beneath the 1947 stadium. The foundation ultimately constructed has three piles: one centered pile with a diameter of 1.20 m and two, one on each side, with a diameter of 0.85 m. This cross-shaped pile foundation, with its offset toward the exterior of the stadium and its unique construction make it very difficult to adapt and reinforce in response to new loads. Added to this is the fact that the retaining wall is interrupted by the base of the 1947 pier (Figure 3, right). On the bridges over the existing tunnels in the affected sections, the 1994 project expands and reinforces these structures. For their part, the new grandstands are metal structures with hollow-core slab decks. In this project, the box seats are incorporated all around the stadium, making use of a space that was only partially occupied on Level 2 in the original construction. Other aspects of interest are described in Cauce 1994 [9].

1.4 Other interventions: 2004 and 2011

In the 1994 renovation, the east side was expanded in a manner analogous to the rest of the stadium, adding two new tiers, but without ribs or a roof. This work was undertaken in 2004 along with the reorganization of the east towers and the new facade on this side. In 2011, a row was added to the cantilever of the first tier, and the first-row column and the inclined beams resting on it were reinforced.

2 Audit of the existing structure

As a first step, a campaign was undertaken involving document collection, structural analysis, and fieldwork. These, together with laboratory tests, made it possible to update the assessment of the structure's safety, functionality, and durability. The objective was to identify the elements most susceptible to the new requirements of the renovation and to define the necessary repair actions or measures to increase the structure's durability within the project.

All graphical information on the structure was compiled, as well as data from four previous survey and testing campaigns carried out by Ayesa, Cones, Intemac, and Retineo. The compressive strength results (Figure 4) show coefficients of variation of around 40% for the older concretes and 25% for those from 1994. The results are consistent with an HA-15 for 1947 and HA-20 (design value) for 1994. The compactness of the concrete was investigated (UNE 83-312- 90), with void ratio results between 9% and 10% for both groups of concrete, indicating "concrete of good quality and compactness."

In a total of 60 core samples, the carbonation depth and cover were measured in beams and columns (Table 2). The cover in the beams was significantly less than in the columns (18 mm versus 38 mm), and in nearly two out of every three core samples, carbonation had reached or exceeded the reinforcement. The cover data for slabs were highly variable, with minimum values of practically zero and maximum values not exceeding 20 mm.

Table 2. Cover and depth (mean ± standard deviation) and proportion of cases where carbonation has exceeded the cover

	Columns 1947	Beams 1947	Columns 1994
Average concrete cover [mm]	38 ± 11	18 ± 11	60 ± 24
Average carbonation depth [mm]	20 ± 8	27 ± 8	27 ± 9
Cases where carbonation depth ≥ concrete cover [%]	20%	61%	0%

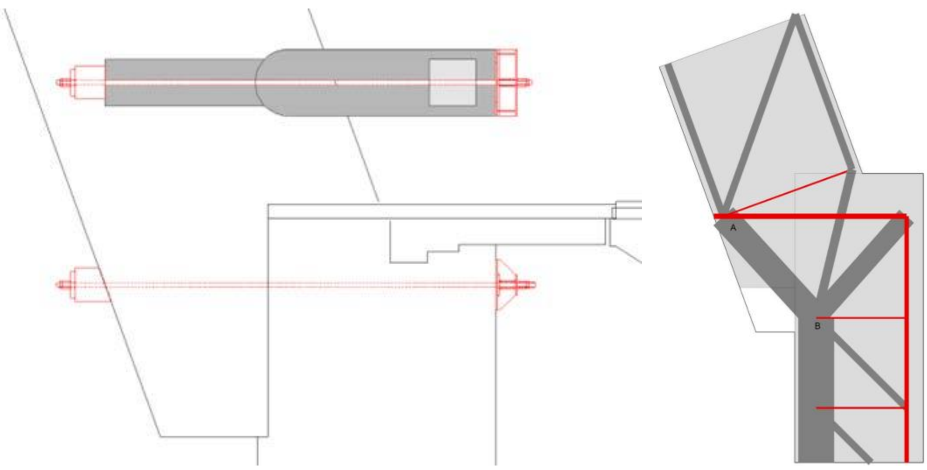

Figure 5. Rib reinforcement. Prestressed bar (i) and diagram of tie rods and struts. The heel extension allows node B to be lowered and also improves the dimension of tie rod AB

However, despite the above results, no signs of reinforcement corrosion were found, even in beams with large carbonation depths, with the exception of areas under poorly ventilated stands with constant moisture. The chloride content in the cement was negligible, and the sulfate content was always below 0.25 mg/m³.

In a total of 10 samples of plain reinforcing steel, an elastic limit of 317±48 MPa (minimum value 244 MPa), a tensile strength of 462±48 MPa, and an average elongation at break of 26% were obtained.

With regard to structural safety conditions, the main conclusion was that certain elements required reinforcement work even before the load increase due to the new use. In particular, the first-row columns and the 120 mm-thick slabs from the 1947 construction, as well as the balcony slabs, which had previously known deflection issues. Significant cracks were detected at the junction of the first (vertical) and second (sloped) sections of the ribs.

3 Structural reinforcements

The increases in surface load are due to new uses, flooring, finishes, and installations, as well as fire protection, the reinforcement measures themselves, and the requirements of current codes.

3.1 Reinforcement of columns and ribs

The columns in the first row along the entire perimeter of the stadium, along with some columns on the lower levels of the east side, were reinforced along the perimeter with continuous reinforcement between floors. A 40 MPa self-compacting micro-concrete with a thickness of 100 mm was used.

The 1994 ribs experienced increased stress on the upper levels. The junction between the vertical base and the inclined section of the ribs was subjected to stress similar to that of a (large) short cantilever. The problem was studied in detail to provide reinforcement consisting of a horizontal tie and a concrete build-up. The tie consists of a ø 65 Y-1050 steel bar within a 90 mm diameter, 5.85 m long borehole. The purpose of the concrete build- up was to increase the flange width of the diagonal section at the junction between segments (Figure 5). Although the tie was located at the center of the rib section, the tension force at the internal anchor could not be applied in the central zone because the 1947–1982 column functions as a 700 mm wide central opening near the edge. The anchor rests on a metal structure that bridges the reaction to two strips only 150 mm wide at each edge. To limit second-order effects out of plane, bracing was installed using metal trusses with a 2.50 m flange width at the top, where they were compatible with the existing installations.

3.2 Reinforcement of slabs and joints

The 120 mm-thick slabs required improved strength and stiffness, as well as fire protection on their upper surface. This was achieved through a nominal 45 mm-thick overlay of self- leveling micro-concrete, which increased the slab depth by 50%. The reality of the slabs' deformations and imperfections necessitated a precision point-cloud survey and the definition of an architectural elevation at each level so that the overlay would always have more 45 mm at its thinnest point. This adjustment affects both architectural elements (stairs, etc.) and effective dead loads. The slab extension is effective under normal roughness conditions, without tie bars between concrete sections and without a connecting bridge. At the joint, dowels are left in the new concrete, and bars are drilled to anchor U-bolts that prevent the dowels from being pushed out. The reinforcement of the overlay consisted of a #ø 6 mesh at 0.15 m and ø 10 bars at 0.15 m on the beams supporting the slab frames.

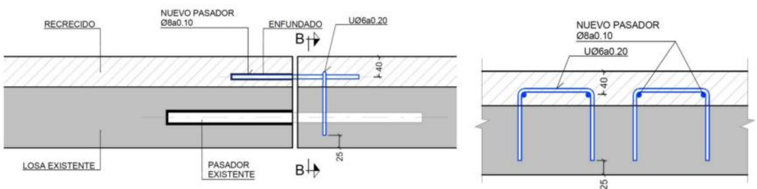

Figure 6. Detail of dowels in slab joints from 1947

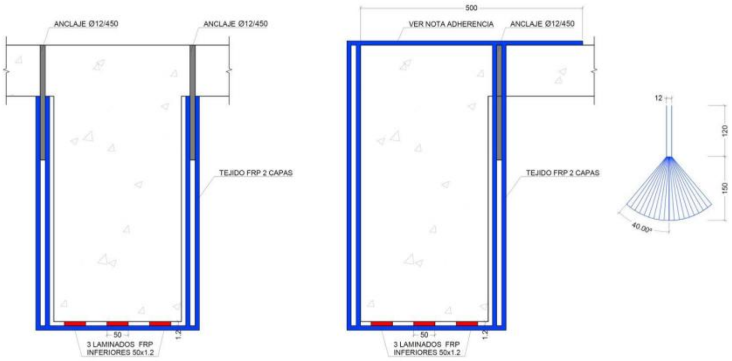

Figure 7. Detail of the anchoring of FRP shear reinforcement fabrics in interior and edge beams

The longitudinal and transverse beams do not significantly improve their safety due to the described overlay. Two types of reinforcement were designed: flexural and shear reinforcement using carbon fiber laminates and fabrics, as well as metal reinforcement. FRP reinforcement was applied whenever possible. Shear reinforcement on T-beams suffers from the problem of reduced effectiveness of the fabric applied to three faces, which also increases the stress on the positive reinforcement. To resolve this, fan-shaped anchors were used. These elements are not covered in the FRP reinforcement standards [10,11], so specialized literature was consulted [12]. Seventeen standard reinforcements were designed, each with between 3 and 9 laminates measuring 50 x 1.2 [mm] and two or three layers of 0.167 mm thick fabric.

FRP reinforcements are applicable up to a certain level of safety improvement required. Furthermore, in some cases it was necessary to demolish sections of beams. This disrupts the continuity of the original design and leaves the sections adjacent to the removed section with a problem in their longitudinal reinforcement, since the hooks anchoring it were located in the removed span. In these cases, an adaptation of a classic wooden beam reinforcement was used by attaching UPN profiles laterally. Where clearance allowed, metal reinforcements were used beneath the existing beams. In total, 20 standard metal reinforcement types were defined, including those found in sections with half-lap joints. All lower metal reinforcements are passive in the sense described in [13].

In the eastern floor slabs, the unique double-width span with a longitudinal beam bolted to the cross beams made standard metal solutions inapplicable. To reinforce the cross beams, which bear point loads, a perimeter micro-concrete overlay was designed, in which the stresses are resisted by the new longitudinal and transverse reinforcement. The reinforcement is conceptually similar to that of the column build-up, but its execution is considerably more complicated. The horizontal interfaces between the existing concrete and the reinforcement concrete required a high-roughness treatment to ensure the transfer of shear forces. The holes drilled in the slab to insert the vertical U-shaped reinforcement also served for pouring the micro-concrete. A similar shear reinforcement using sprayed concrete is presented in [14]. The anchorage zone with the longitudinal beam requires local reinforcement and detailed reinforcement (Figure 9).

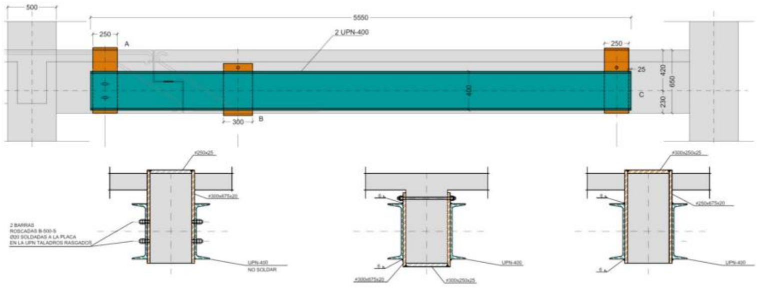

Figure 8. Typical steel reinforcement of a longitudinal beam at a joint section

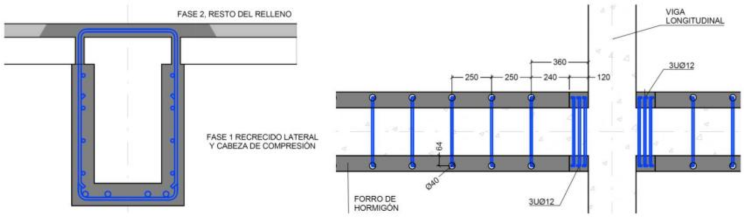

Figure 9. Section of the overlaid transverse beam and plan detail of the connection with the longitudinal beam

Once all standard reinforcements have been defined and verified, and their conditions of applicability (spans, loads, clearance, and continuity) established, the project is organized from the general plan, assigning the minimum reinforcement in each span according to predetermined rules, as if it were a board game. This is necessary to undertake a large-scale, complex project in which construction proceeds in parallel with, if not ahead of, the design and with real-time data.

Meanwhile, the balcony slabs were replaced with cast-in- place reinforced concrete slabs. These slabs rest on the metal profiles that supported the previous slabs—though their elevation had to be lowered—and on the original longitudinal beams via dowels. The cantilevers of these new balcony slabs remain the original slabs.

4 Special interventions

4.1 Structural underpinning next to the vehicle tunnel

One objective of the renovation was to provide truck access from the outside to the playing field. The layout of this access (the vehicle tunnel) defined a grade line below the foundation pits for the pillars of the east building. To construct the tunnel, it was necessary to shoring and apply load to two alignments, simultaneously supporting five pillars with a total force of 11 MN. The application of active load was considered important to prevent the shoring from suddenly bearing the load while drilling the existing pits or excavating the tunnel.

To this end, a line of formwork was constructed on micropiles and separated from the ground by expanded polystyrene. A wall beam was formed on this line, encircling the existing pillars. A gap was left between the wall beam and the foundation to accommodate 17 jacks. Once the loads were applied—which differed for each jack depending on the reactions of the corresponding pillars—shims were inserted and the gap was filled with concrete. The total movements of the structure under maximum load were less than one millimeter.

4.2 Structures over the tunnel

The bridges over the railway tunnel from the original project and their expansion and reinforcement in the 1994 project have been described. At both the southern and northern crossings, the deck is supported by auxiliary supports (the crutches) resting on these structures. The most complex case is found on the north side (Figure 11), where the support is located next to a rib that is already supported at one end by the bridge of the original project (fourth-alignment pier) and at the other end by a reinforced concrete bridge with a 3-meter depth.

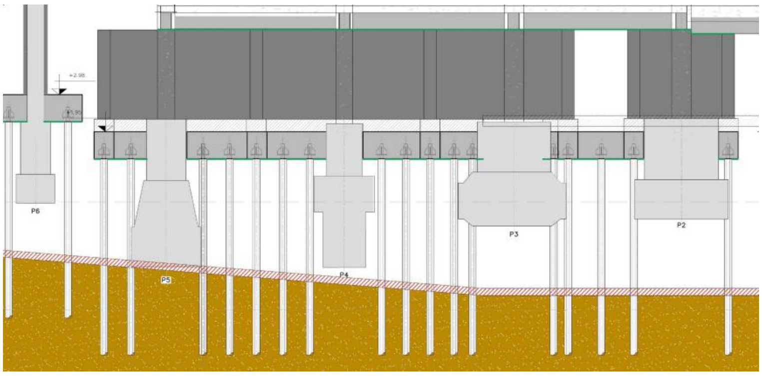

Figure 10. Partial elevation of the structure's underpinning and the lower level of the vehicle tunnel

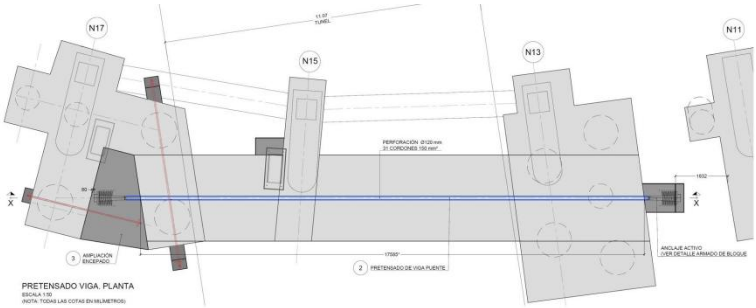

Figure 11. Plan view of one of the structures over the tunnel showing the successive interventions of 1947, 1994, and the current reinforcement. The new roof supports are located next to ribs N15 and N17

The significant increases in stress on this element (biaxial bending and torsion) were addressed with a set of reinforcements that included the stitching of several foundations and the longitudinal prestressing of the bridge girder, which required a 17.5 m borehole drilled with a misalignment of less than 60 mm. The prestressing provided additional resistance to the stresses and sufficient stiffness to prevent the reactions from straining the older parts of the structure.

4.3 Isolation of the east stand and reinforcement of the tunnel

The Atocha-Chamartín tunnel runs through the field from south to north, lying very close to the surface for much of its length beneath the east stand. The Club and Adif agreed to correct this situation through two measures: first, reinforcing the tunnel vault in the section beneath the field, and second, decoupling the grandstand from the tunnel. Thus, we had to transform this grandstand, which rested on the ground, into a self-supporting structure. The resulting structure cantilevers over the tunnel or is double-supported, depending on the area, and is coordinated with the foundation of the new movable playing field slab, described in another article in this issue.

4.4 Reinforcement of DAC rib foundations

Finally, we briefly describe the reinforcement required in eight ribs on the north end and an equal number on the south end due to the force limited by DAC devices at a rate of 500 kN per rib. Similar reinforcements were installed in the shafts of these supports—albeit on a larger scale—to those of the standard ribs (Figure 5), supplemented with FRP fabrics in the inclined section to reinforce internal ties comparable to the element's shear force. In these 16 cases, the analysis of forces at the joints revealed the need for reinforcement. It is worth noting that no new reaction elements (piles or micropiles) were required, but rather an effective internal mechanism to transfer the forces to the existing piles. Figure 13 shows the plan view of the static diagram. The eccentric axis of the rib is represented by compression in its outer zone (semicircular cap) and tension whose center of gravity lies in the opening due to the pre-existing column. The struts or tie rods shown as dashed lines are in a horizontal plane; the rest are 3D struts.

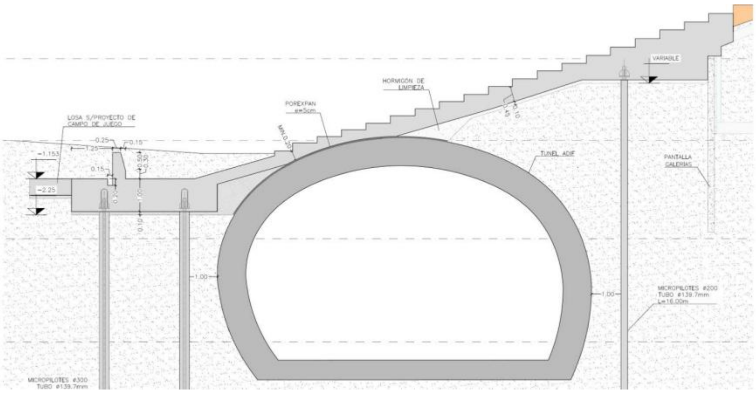

Figure 12. Cross-section of the structural grandstand and tunnel on the east side (the reinforcement of the tunnel itself is not shown)

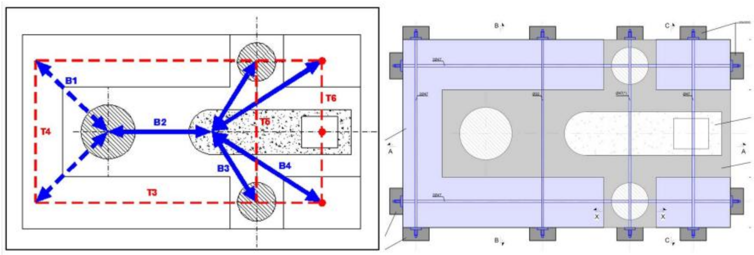

Figure 13. Plan view of the reinforced abutment equilibrium scheme (i) and extension and connection with prestressed bars of the abutment

5 Conclusion

The comprehensive renovation of a living infrastructure such as the Santiago Bernabéu Stadium requires a precise understanding of the different vital stages it has undergone throughout its history, its past interventions, and its daily operations.

The work carried out on the existing structure demonstrates how it is possible to preserve and adapt structures over 75 years old to new operational requirements using modern reinforcement and rehabilitation techniques. This variety of techniques applied to columns, ribs, and floor slabs has ensured the stadium’s stability and safety in the face of new usage demands; however, above all, they had to coexist with a stadium that has remained operational at all times, which has required analyzing its suitability throughout different construction phases. Finally, the process of separating the east stands from the railway tunnel is an example of innovation applied in contexts of structural complexity.

Conflicts of interest

The author declares no conflicts of interest regarding the publication of this paper.

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