Chapin Stadium Extension, Jerez, Spain

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Introduction

Scope

The horseracing world championship was held in Jerez, Spain, in September 2002. For this occasion, the town owned track and field stadium has been extended and refurbished. The paper describes the basic ideas for the conceptual design of the extended structure, bearing in mind the complex boundary conditions. Since the extension of the stadium leads to an important increase of the internal forces, challenging problems emerged in relation to the existing structure. The adopted solutions are emphasised in the paper.

Existing Stadium

The existing stadium, which is a purely functional construction of no particular architectural or structural interest, was completed in 1988 (*Fig. 1a*). Only the main grandstand is covered with a roof (*Fig. 1b*). The grandstands are constituted by in situ concrete frames at 6 m centres and by precast concrete elements as platforms for the public. The maximum height of these frames above the ground level is 10,8 m, approximately. A typical frame is constituted by an inclined beam (standard cross section of 0,85 m × 0,5 m with variable height near the columns) and two columns, separated 9,25 m approximately, transmitting the loads to the foundations (*Fig. 1c*). The effective cross sections of the outer and inner columns are 1,1 m × 0,5 m and 0,8 m × 0,5 m, respectively. The separation of expansion joints, constituted by double frames, is 24 m.

Planned Intervention

The extension of the stadium required the addition of a new roof as well as a new gallery to the existing grandstand. The accesses for the public were also reorganised. Furthermore, a sports centre and a hotel were to be integrated into the stadium. A section through one of the standard frames after the extension of the stadium shows the new, cantilevered roof with an integrated, 1,16 m wide catwalk for maintenance purposes, as well as the new gallery with a slab, 2,25 m wide, for the movement of spectators (*Fig. 2a*). The section also shows the new concrete wall surrounding the whole stadium with integrated doors giving access to the new stairways that, on their hand, lead to a new slab and walkway giving access to the grandstand. In the case of the main grandstand the solution is basically the same, the main differences being related to the existence of a basement (*Fig. 2b*). Furthermore, the new gallery includes a slab extended to its whole width and the access to the grandstand is constituted by a full slab rather than by a walkway.

Boundary Conditions

In the case of most buildings, the main difficulty for an engineer arises from the need to translate the architectural and functional requirements into a geometrically consistent structure. In the present case, apart from these usual requirements, a certain number of particular conditions were to be considered:

- The extension of the existing structure implies an important change of the static system, suggesting therefore the need of an extensive intervention, including important demolition work. In this way, freedom can be obtained for the adoption of a functional and reliable solution.
- The owner made it a requirement that the stadium should be in use during construction. This condition suggests the need for minimum intervention, avoiding as far as possible the demolition of existing structural members.
- The immovable service date also suggests a minimum intervention and that the time of execution should be minimised through the adopted methods of construction.

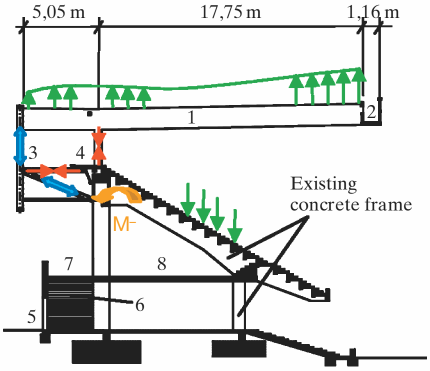
The combination of these contradictory conditions have serious consequences with relation to the selection of the materials to be used in the construction, the global structural concept and detailing, and the very methods to be used in its manufacture and assembly. Particularly, the requirement to keep the stadium in use during construction was one of the dominant conditions for its structural design.

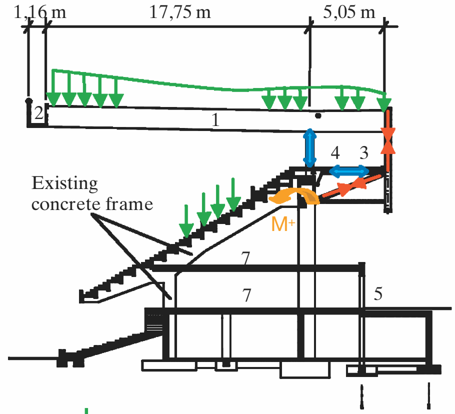
Conceptual Design

General Remarks

The reliability, the functionality and the economy of a structure directly depend on its conceptual design. Al-

Fig. 1: Existing stadium: a) Outside view *b) Covered main grandstand* *c) Typical frame*





New elements
1 Steel box girder of the new roof
2 Catwalk for maintenance
3 Gallery
4 Composite slab for movement of spectators
5 Concrete wall surrounding the stadium
6 Stairways for the access of spectators
7 Concrete slab connected to existing structure
8 Walkway giving access to the grandstand

Load
Tensile force
Compression force
M Bending moment in the joint of the existing frame

a) Typical frame

b) Frame of main grandstand

Fig. 2: Planned intervention. Cross sections

Fig. 3: Extended stadium with new roof: a) inside view

b) outside view

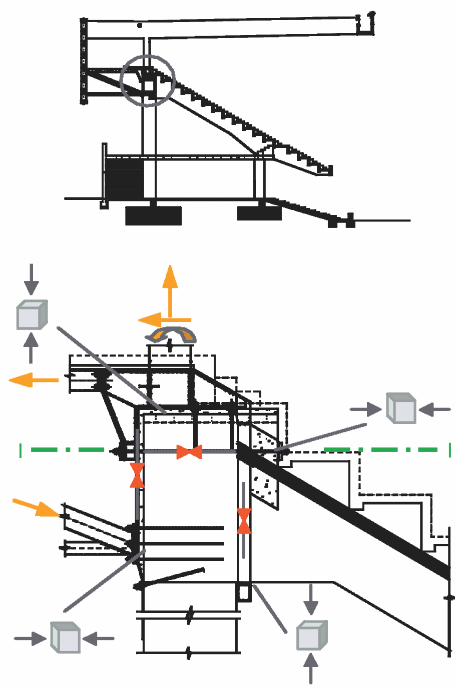
though structural analysis is important for the verification or optimisation of any adopted solution, the conceptual design is possibly the most important step in the whole structural design process. Going out from a structural idea, the conceptual design consists of developing the solution in terms of schematic drawings including structural detailing. The viability is demonstrated by means of simplified calculations. The outcome of this step also includes the definition of the main dimensions of the structural members for the adopted solution.

Global System and Force Transfer

Steel frames are added to the existing concrete frames, constituting the new roof. Each new frame consists of a steel box girder with a total length of 22,8 m, including both the cantilevered part (17,75 m) with a constant cross section (1400 mm × 300 mm) and the part covering the new gallery (5,05 m) with a cross section with a slightly increasing height towards the end of the beam according to the planned draining system. Each main girder is supported by a steel column with a welded box section (500 mm × 300 mm) that introduces the reactions directly into the existing reinforced concrete frame, and a tension member (tie) connected to a truss-like structure of the new gallery that is also connected to the concrete structure. All of these elements are constituted by profiles with hollow sections. The roof panels span the 6 m between the main beams thus avoiding the disposal of a secondary structure. Therefore, the only transversal structural elements are, respectively, the catwalk, two chords at the top and the bottom of the so-called tie and the composite slab of the new gallery, constituted by a profiled steel sheeting and cast-in-place concrete. By avoiding the need for the secondary structure – originally planned by the architects –, a very neat solution is obtained for the roof structure (*Fig. 3*). This example shows that sometimes the architectural design may be improved through structural considerations.

Gravity loads lead to the flow of forces in the steel structure as shown in *Fig. 2b* and to important action effects, particularly bending moments, in the existing concrete frame. As wind tunnel tests showed, an inversion of the action effects can take place due to negative wind pressure. In this case, tensile forces appear in the steel column, compression in the so-called tie and inverted bending moments in the existing concrete structure (*Fig. 2a*).

When drilling holes in existing reinforced concrete structures, it is reasonable to assume that some of the reinforcement bars will be damaged. In the present case it appears to be particularly important not to weaken the existing structure since the addition of the new roof leads to an important increase of the action effects. The load transfer from the new to the existing structure should therefore be achieved by contact pressure on the concrete surface rather than by post-installed anchorage bars. The conceptual design of the connecting device is such that this aim is reached in the case of the inversion of the action effects due to



Internal forces and moments, new structure
Tensile elements integrated into the connecting device
Contact pressure

Fig. 4a: Force transfer by contact pressure in case of load inversion due to negative wind pressure

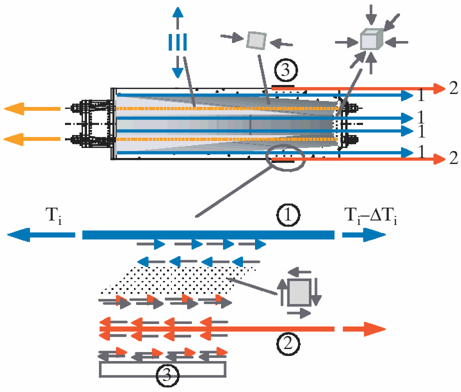


Fig. 4b: Stress field in joint of post-strengthened frame (Section I–I)

negative wind pressure (*Figs. 4a* and *4b*). However, since alternative solutions would require demolition of part of the existing structure, which was to be avoided to keep the stadium in use during construction work, the most reasonable solution for the transfer of the tensile forces from the diagonal into the concrete frame includes the use of post-installed anchorage bars.

Stability

Considering the stability of a standard frame, it must be distinguished between situations with dominant gravity loads and situations with dominant negative wind pressure. In both cases out of plane and in plane stability must be achieved. The out of plane stability in case of negative wind pressure appears to be critical. Indeed, in that case both, the so-called tie and the diagonal of the truss-like structure of the new gallery, are under compression (*Fig. 2a*). The out of plane stability of the frame is achieved through its interaction with the catwalk, the transversal chords at the top and the bottom of the tie and the composite slab of the new gallery. Each of these elements constitutes a lateral support to the steel frame. In this way, although under compression, the tie provides an elastic lateral support to the diagonal under compression. Furthermore, considering the system with an out of plane deformation, it appears that the horizontal tension member of the truss tends to restore the original geometry, introducing an additional stabilising effect.

Wind Induced Dynamic Effects

Positive and negative wind pressure on the roof as well as the dynamic effects induced by wind lead to variable stress cycles in the different structural elements. These stress cycles can induce fatigue problems, particularly in the connecting device between the new and the existing structure and in the concrete of the locally high stressed zones due to load introduction. In order to mitigate the risk of fatigue, the vertical bracing and the horizontal post-installed anchorage bars are prestressed. Conservatively, the prestressing forces are chosen such that the detachment of the steel elements from the concrete surface is avoided for the characteristic value of the wind load as well, in case of positive or negative wind pressure.

Existing Structure

Assessment

In the present case, the structural safety of the existing structure was to be assessed for three main reasons:

– the increase of the action effects in the existing structure due to the addition of the new roof
– the possible damage to the reinforcement bars when drilling the holes for the post-installed anchorage bars
– the detected differences between the original drawings and the existing structure, particularly in relation with the reinforcement of the joint of some of the existing concrete frames, where apparently less reinforcement bars have been placed than planned according to the original project.

The assessment of the structural safety was carried out according to a staged procedure, which included the application of reliability methods. Taking advantage of the fact that many characteristics may be measured from the structure under consideration which, at the time of its design, were just anticipated quantities, the level of accuracy for the load and resistance models needed for the assessment was increased from stage to stage. However, in the present case it was not possible to verify structural safety, not even by carrying out a probabilistic analysis using site data. Therefore, the structure was to be strengthened, for the design situations with dominant gravity loads, leading to (by definition) positive bending moments in the joints of the existing concrete frames. This also applied to the design situations with negative wind pressure leading to negative bending moments.

Post-Strengthening of Existing Structure

For the case of positive bending moments due to dominant gravity loads, the existing concrete frames have been post-strengthened by bonding steel plates to the outer surface of the concrete columns. For the case of negative bending moments due to dominant negative wind pressure, different solutions have also been adopted for typical frames and double frames at the location of the expansion joints. For the latter, the device for the load transfer

Fig. 5: Post-strengthening of existing concrete frames with externally bonded CFRP laminates: a) Before painting b) After painting c) End anchorage

from the new to the existing structure has been adapted in a way that both frames are integrated into the resistance mechanism. In this way, accurate structural safety can be justified for the new action effects even without further post-strengthening of these frames. In the case of the typical frames, the upper reinforcement of the inclined beam is to be supplemented to achieve the required reliability for the new action effects. For this purpose, Carbon Fibre Reinforced Plastic (CFRP) laminates were externally bonded to the inclined beams of the frames (*Fig. 5a*). This solution was preferred to other possible strengthening methods due to the ease of handling the material on the construction site and the corresponding reduction of execution time and labour costs. Furthermore, from the aesthetic point of view, this solution is also very satisfactory since after painting the CFRP strips are almost not perceptible (*Fig. 5b*). Some problems did arise from the fact that the available anchorage length at the upper end of the CFRP strips was not sufficient to balance the tensile forces. Therefore, it was necessary to use a plate anchorage device. The use of bolts was excluded for architectonic reasons, the connection between the steel plates and the concrete has been achieved by means of an epoxy adhesive (*Fig. 5c*). Since models are not available from literature for the calculation of the resistance of this kind of end anchorage, a specific design model has been developed. In a second step, representative field tests have been carried out to reduce the uncertainties associated with this model.

Testing

For the design of the required post-strengthening and its anchorage a consistent model based on the lower bound theorem of the theory of plasticity was developed using the stress field method. To this end, the lower bound theorem was reformulated as follows: A stress field is chosen such that the equilibrium conditions and the statical boundary conditions are fulfilled. Given the dimensions and the existing reinforcement of the structure, the post-strengthening has to be proportioned such that the resistances are everywhere greater than or equal to the corresponding internal forces. In order to ensure the desired failure mode of the strengthened structure, characterized by the failure of the CFRP composite during yielding of the steel reinforcing bars before a compressive failure of the concrete, a non uniform distribution of the stresses in the concrete was chosen with a limitation of the maximum concrete strains. These assumptions lead to a conservative estimation of the resistance of the post-strengthened structure. *Fig. 4b* shows a horizontal section through the stress field for the joint of the post-strengthened concrete frames including the anchorage zone of the CFRP composites with the bonded steel plates. The material properties, as well as the data for the force transfer between the different elements have been taken or deduced from the literature, the information provided by the suppliers and from tests. The set up of the field tests for the validation of the established model for the resistance of the anchorage device is represented in *Fig. 6*. The tests showed that the model uncertainties are rather small. Indeed, the mean value (m_M) and the coefficient of variation (cov_M) of the model variable (M) that describes the deviations between the analysis and the experimental results due to the simplified resistance models, indicate a good quality of the established model (m_M = 0,994; cov_M = 0,069).

Fig. 6: Field tests for the end anchorage device

Conclusions

The most important conclusions from the present case study can be summarised as follows:

- Challenging problems have to be solved when extending existing structures, particularly in relation with the assessment of their structural safety for the new conditions. The assessment of an existing structure based on incomplete or defective information may be completely wrong. Therefore, correct updating of data is probably the most important step in a structural evaluation.
- Models in the form of stress fields have the advantage to allow a designer to follow the forces through a structure. Therefore, these models not only help to conceive and design reinforced concrete structures, but they also offer a very promising way for the design of the post-strengthening of existing structures.
- Design assisted by testing is a very powerful, efficient and economic tool where codes or literature do not give sufficient information on structural properties. When determining such properties by testing it must be ensured that they are associated with the same level of reliability as when determined according to codified calculation models.

SEI Data Block

Owner:
Sociedad Jerez 2002, Spain

Architects:
Cruz y Ortiz Arquitectos, Seville, Spain

Structural engineers:
P. Tanner and J.L. Bellod, Cesma Ingenieros, Madrid, Spain

Main contractors:
Joint venture FCC – Inabensa, Spain

Subcontractor, steel structure:
Estructuras Metálicas ELTE S.A, Spain

Structural steel (t):	1105
Total cost of structure, stadium only (EUR millions):	3656
Capacity (seated spectators):	22 000
Service date:	September 2002