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# Hybrid masonry shell technology in the work of Idelfonso Sánchez del Río

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**ABSTRACT**

*Idelfonso Sánchez del Río is a less known pioneer of reinforced concrete shells in Spain who through his career designed and patented ribbed construction systems for large spanning slabs and vaults and in particular shell enclosures using a hybrid system of concrete and masonry infills. The module called "dovela-onda" or wave-voussoir was made of large ceramic blocks forming a short barrel with flanges at the edges. This paper aims to discuss the technical innovations of this system and assess its structural efficiency. The design and construction process will be studied through literature published by Sánchez del Rio and surveys of two case studies in Oviedo (Spain), the Sports Hall (1977) and a warehouse in Granda. In order to assess their structural efficiency, his own calculation process will be verified by thrust line analysis and Finite Element spatial elastic modelling. The FE model allows the failure mode and the distribution of the loads to be assessed, and gives further insight to the behaviour of the scheme and the design and construction process.*

**KEYWORDS**

Reinforced brickwork, shells, hybrid construction, Sánchez del Rio, elastic instability

## 1 INTRODUCTION

Idelfonso Sánchez del Río y Pisón (1898-1980) is a less known pioneer of reinforced concrete shells in Spain who through his career designed ribbed construction systems for large spanning slabs and vaults. After holding public posts as engineer in Oviedo and Madrid until 1946, when he carried out his early advanced designs, he set up his own consultancy specialising in shell enclosures using a hybrid system of concrete and masonry infills. The system used a module called *dovela-onda* or wave-voussoir, assembled from hollow ceramic blocks in the form of short barrels with flanges at the edge. The units were reinforced and would be then bonded to form series of arches of deeply undulated section connected along their edge, to form a corrugated uninterrupted enclosure.

This paper aims to discuss the technical innovations of this range of buildings and assess their structural efficiency. Most of these roofs were built above warehouses, while the largest of them formed the enclosure of the Municipal Sports Hall in Oviedo, his last major work, completed in 1977. The origins of the system and the form-finding process will be initially examined as a result from Sánchez del Rio's earlier work on concrete shells, like water reservoirs (Oviedo) and market halls (Pola de Siero). The study discusses the transition from the earlier use of ceramic units for lightweight floor decks to the off-site production of the undulated modules.

The design and construction process will be studied through literature published by Sánchez del Rio and surveys of case studies in Asturias, in particular the Oviedo Sports Hall and a warehouse in Granda, outside the city. In order to assess their structural efficiency, the calculation process he followed will be verified by thrust-line analysis and Finite Element spatial elastic modelling. The deformation and load distribution in the structures will also be assessed, aiming to improve knowledge of their structural behaviour and the design process behind them.

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## 2. SÁNCHEZ DEL RIO'S EARLIER REINFORCED CONCRETE SHELLS

Sánchez del Río's most creative period was during his post as a municipal engineer for the City Council of Oviedo in Asturias, Spain (1924- 1940). Due to the wide range of duties and the extent of jurisdiction of his post, he designed and constructed structures as diverse as water reservoirs (Oviedo, Villapendi, Trubia), market halls (Pola de Siero) and industrial buildings (gas works in Oviedo). Even more expressive have been the self-standing roofs such as the Grandstand of the old Tartiere stadium in Oviedo (now demolished) or the characteristic *paraguas* (umbrellas) in market squares (Oviedo, Cerdeño, Pola de Siero). The two most known and influential shell structures from that period are the 4th Water Reservoir (Cuarto Depósito de Aguas) in Oviedo (1926-28) and the City Market (Mercado de Abastos) in Pola de Siero (1930) (Fig. 1). The shell roofs cover quite different and non-conventional plans in each building (a central plan and a triangular site respectively) and this was made possible by using conical shells roofs that explored the possibilities of reinforced concrete strengthened by ribs.

**Figure 1.** The deep rib reinforcing the intersection of the main conoid vaults, City Market, Pola de Siero (1930)

## 3. HYBRID MASONRY SHELLS

Following this experience, the role of the rib in generating and strengthening the intersections became apparent. At the same time, Sánchez del Rio [1942] experimented with composite lightweight systems in order to form long-spanning floor decks (Campoamor and Filarmónica theatres, Oviedo). The wide-spread jack-arches or similar decking systems of the time used the ribbed ceramic unit only as a mould for the concrete, which would then perform the load-bearing function. He developed ceramic blocks that incorporated the ribs of the unit in the load-bearing function and optimise the use of concrete.

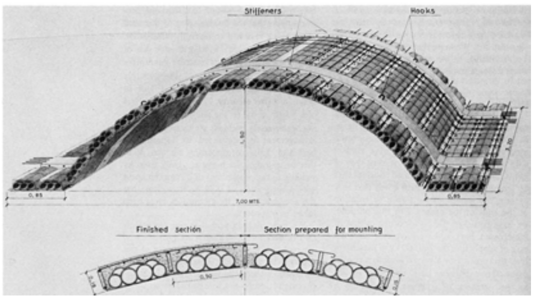


**Figure 2.** Diagram of the *dovela onda* (wave-voussoir) element [Sánchez del Rio 1962b]

In this study, the focus on his work will be on the systems he created for shell roofs. His experience with rib construction led him to the module of *dovela-onda* that can be translated as wave-voussoir. The unit shown in Fig. 2 functions as the voussoir of large spanning arches and was devised to combine increased depth in the section (to improve resistance in bending and axial compression) and standardisation in construction. The hollow blocks resulted not only in lower loads but also in higher thermal and acoustic insulation. When these arches are built next to each other they can form undulated roofs capable of roofing large spaces, while if their height is varied they can then let natural light through the gap formed between adjacent arches. Spans up to 100 m became possible, supported in two main arrangements: either upon the lateral walls of the enclosed space and braced with ties, as in the case of industrial sheds; or upon a series of slender concrete buttresses that could hold the weight and counteract their thrusts, as in the case of the Oviedo Sports Hall.

The voussoir (Fig. 2) is made of large and hollow ceramic units bonded with grout and with a layer of steel reinforcement between them, further strengthened with two stiffener ribs running through the entire unit. The arch is then assembled by hoisting the units from hooks on the stiffeners upon a dense temporary falsework and then bonding them together with a concrete mix. Concrete would also be poured on the extrados of the vault in order to bond the units and the reinforcement, completing therefore the co-action of the three materials (brick, concrete and steel bars). An entire arch could be built and set within 7 days [Sánchez del Rio 1962a].

In all these cases, the cost of the falsework is a major factor. To reduce such expenditures, it was important to simplify the layout of the works by standardising the units to be supported. Also, the form had to be such that temperature or workmanship quality variations would have a minimum effect on the temporary stability of the unset concrete. The solution was a catenary profile (which would minimise bending moments) in the form of a two-hinge arch (which would function as an isostatic structure that is not dependant on material properties).

The system has similarities to a process developed later for the construction of the CNIT centre in Paris as also the system of inverted-T units used at the Duxford Air Museum. In retrospect, Sánchez del Rio questioned even how the design of the Siero Market could have been more structurally and thermally efficient had he developed such systems earlier [1962a]. The success of such an approach will be assessed in the two case studies.

## 4. PALACIO DE DEPORTES (MUNICIPAL SPORTS HALL), OVIEDO, SPAIN (1977)

### 4.1 Design of the centre

Sánchez del Rio discussed comprehensively his design for the forthcoming Sports Hall in [1962b] and this publication will be used in this study as the basis for the verification of the structural performance of the building based on other theoretical methods like thrust line and Finite Element (FE) analysis. The Sports Hall was finally built in 1977 as a series of eight 96 m span deep arches over the track (span L = 96 m, rise F = 19.2 m), flanked by two pairs of lower, 88m-span arches at the ends (Fig. 3). The height difference between the two types and the self-standing capacity of each arch allows for clerestory light arrangement (between the arches) and major openings at the ends. The aspect ratio is L/F = 5 (for both) and the directrix was formed using the equation:

$$y = -\frac{1}{129{,}000}\left[0.05425x^4 + 950x^2\right] + 19.20 \qquad [1]$$

**Figure 3.** Exterior and interior of the Oviedo Sports Hall

**Figure 4.** The reinforced concrete buttresses supporting the roof, fitted with a pin connection at their lower end.

Focusing on the shells, each arch is supported by slender concrete buttresses that function as props to contain the thrust and anchor the vault (Fig. 4). The two hinges that are necessary by the structural design in order to provide an isostatic behaviour, are fitted towards the base of the buttresses using stainless steel bars which are exposed and encased in a glass box. At the end of each arch, a self-supported cantilevering reinforced brickwork canopy was added, with the intention to act as an open abutment to the springing, which would provide stiffness without fixity. The exposed hinge was intended as a key educational feature towards the understanding of the unique structural scheme and unfortunately the Hinges Gallery planned to enclose the buttresses and celebrate this aspect was never materialised. While the outer cantilevers had their masonry exposed, the intrados of the main arches at the interior was coated with a concrete screed, probably for acoustic and fire protection.

**4.2 Construction process: the use of the wave-voussoir**

Figure 5 was taken during the erection of the vaults and highlights how critical is to economise on the falsework for the assembly of the arch. The voussoir shown is held by hooks fixed on the stiffeners and further reinforced (cf. Fig. 2). As the voussoir had to carry its own weight during this delicate phase and as the shell action was slowly being built up while the concrete was setting, it was important to ensure the unit could function as an arch in its own merits. A catenary section was therefore formed also for the section. The geometry and construction of the cross-section follows the layout in Fig. 2 and the thickness of the shell t = 0.18 m.

**Figure 5.** Centering and assembly of the roof of the Oviedo Sports Hall [Colegio 2004]

**4.3 Stability of a typical arch**

In his paper [1962b], Sánchez del Rio discussed how he assessed the safety of the form, especially in elastic instability, the major type of failure expected to affect a catenary arch. Assuming elastic modulus for the masonry E= 20 kN/mm$^2$ and $\nu = 0.15$ (relating to the mechanical properties of the constituent materials retrieved from tests), he evaluated the critical load for the major arch spanning L=96 m using the Euler formula:

$$P_{cr} = \frac{k \cdot \pi^2 \cdot E \cdot I}{L^2} \qquad [2]$$

Because the structure is a two-hinged arch, k=1, therefore $P_{cr}$ = 5330 kN. The maximum axial load corresponding to self weight of the structure was evaluated as P =1740 kN, and the resulting factor of safety $\gamma_f$ = $P_{cr}$ / P = 5330 / 1740 = 3.06. It is expected that the overall safety factor of the entire roof will be higher due to the continuity of the eight arches, in which case this calculation is a conservative estimate. A similar assessment was carried out for the shorter (and lower) lateral arches spanning L = 84 m, resulting in a safety factor $\gamma_f$ = 4.

The behaviour will be examined with a more rigorous theoretical procedure that was codified and used by Eladio Dieste, a major shell designer of the 20th century, as part of the design of Gaussian, double-curvature brickwork vaults [Dieste 1985]. This method was successfully applied to the design of his JHO Warehouse in Montevideo and was later verified using the FE method [Theodossopoulos and Pedreschi 2004]. This procedure establishes the thrust line of an arch in buckling and also caters for any possible cross-section by allowing even variations in the geometry and moments of inertia of the section between the supports ($I_{support}$ ) and the crown ($I_{crown}$ ). Equation 3 describes the buckling centreline of a catenary arch and Fig. 6 illustrates the relevant quantities:

$$\frac{d^2 y}{du^2} = -\chi \cdot y \cdot \sqrt{\gamma^2 + (1-u)^2} - \frac{\gamma^2 \cdot y}{\left[\gamma^2 + (1-u)^2\right]^2} \qquad [3]$$

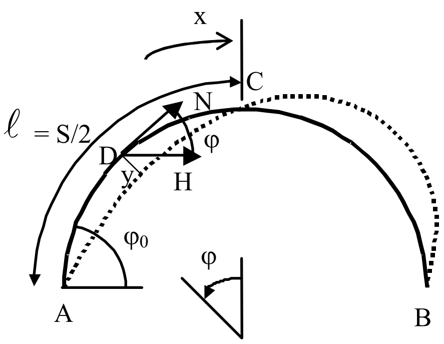


**Figure 6.** Buckling of a catenary arch under dead load – definitions after [Dieste 1985]

Where $v = \overrightarrow{AD}$ and $u = \frac{v}{\ell}$, $\ell$ is the half length of the arch S, $\gamma = \frac{1}{\tan \varphi_0}$ and $I$ is the moment of inertia of the cross section at the support. As $u \in [0,2]$ , the quantity $\chi$ will be obtained from the boundary conditions in eq. (3): $y$ = 0 at the locations $u$ = 0 and $u$ = 2 (bases) and at $u$ = 1 (apex). The differential equation is integrated by a numerical/ graphic method: a value for $\gamma$ is chosen and then for every $\chi$ a value for $y$ at the supports is calculated ($y_B$), which in general should not be 0. The values of $y_B$ are then plotted and the roots of the equation $y_B(\chi)$ are evaluated graphically. The solution is given by eq. 4 where $\chi$ can be evaluated in terms of the springing angle $\varphi_o$. (and $\gamma$) and this relationship was organised in graphs in terms of the variable $\upsilon$ [eq. 5] that is used to define the change in the cross section (Fig. 7). The problem of elastic instability can be summarised as this: for a given arch (defined by $\ell$ and $\gamma$) $\chi$ can be evaluated from eq. 4 and therefore the critical load $q_{cr}$ :

$$\chi = \frac{q \cdot \ell^3}{EI} \qquad [4]$$

$$\upsilon = \frac{I_{crown}}{I_{\sup port}} \qquad [5]$$

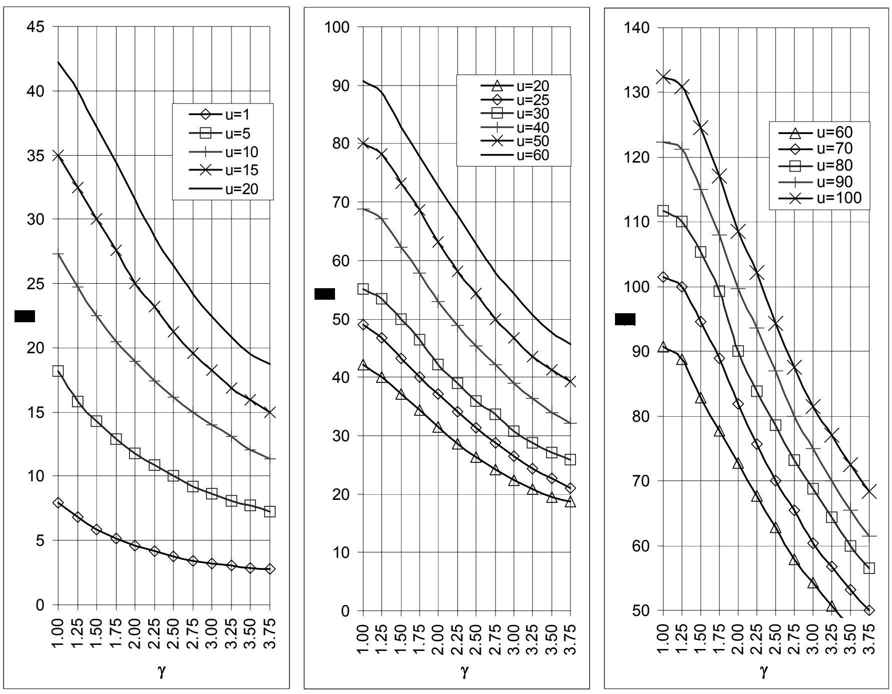


**Figure 7.** Values of $\chi$ in terms of the profile of the arch ($\gamma$) and its cross section ($\upsilon$) [Dieste 1985]

In the case of the Oviedo Sports Hall, the cross section is uniform section, so $\upsilon$ = 1. Using catenary arches tables calculated by E. Dieste [1983], some geometric quantities of the cross section (Fig. 2) and the directrix can be evaluated, as both curves fit the catenary profiles tabulated by Dieste. Regarding the cross section (aspect ratio $Lv/Fv = (7\text{-}2{\cdot}0.85)/(1.5+0.18/2)=3.3$), the second moment was calculated as $Ig = 0.3764{\cdot}t{\cdot}Fv = 0.272\ m^4$ . In the case of the directrix, $\ell = 47.67$ m and from the tables, as the ratio L/F = 5, the springing angle $\varphi_o = 40$, and $\gamma = 1.19$. From Fig. 7, $\chi = 7$ and rearranging eq. 4, $q_{cr} = 351$ kN/m or 42.75 $kN/m^2$ if divided with the overall width (length of the cross section) 8.21m. The equivalent linear uniformly distributed pressure due to the self weight of the structure results from multiplying the unit weight of masonry 16 $kN/m^3$ by the width (8.21m) and thickness (0.18m) of the shell and is equal to 23.64kN/m. As a result, the degree of safety $\gamma_f = q_{cr} /q = 14.9$, a much wider margin of strength than the earlier conservative approach.

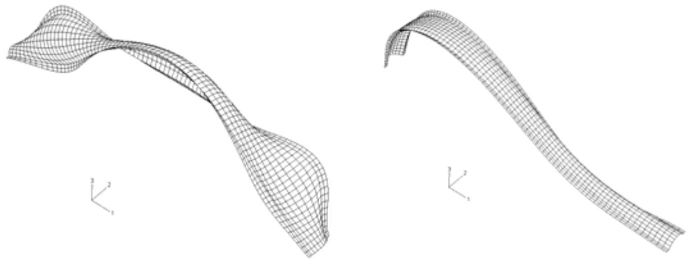

**Figure 8.** First (16.5) and second (18.7) buckling modes of the Finite Element model

The same problem was assessed by FE modelling using the program ABAQUS, considering the structure as a homogeneous shell. Curved, 4-node shell elements S4R5 were used, pin-support conditions were applied at the ends above the abutting points and linear elastic analysis was performed [Theodossopoulos and Pedreschi 2004]. The buttresses were not modelled in order to maintain a direct relevance to the theory. The first buckling modes were evaluated and the eigenvalues (safety factors) for the first three modes are 16.5, 18.7 and 22.4. The values are of similar magnitude to the one evaluated theoretically (14.9), so both methods demonstrate the relatively high reserves in strength of the form. Pinned supports were considered applied directly on the shell and in a next phase of this research the effect of the concrete piers needs to be considered. Among the first two modes (Fig. 8) the second one seems the most probable form [Dieste 1985], so the safety factor against buckling $\gamma_f$ is 18.7 .

### 4.4 Other aspects of the structural performance of the shell

The FE model brings further insight to the behaviour of the shell, like the pattern of the distribution of the loads and the deformation [Fig. 9]. As expected, the higher deformation occurs at the apex of each arch but the deep section results in a reduction of the higher values along the ridge. The values overall are relatively high, with a maximum at the apex being 26 mm, but in terms of serviceability limit is 1/3700 of the span, well below any relevant limit applied for equivalent girders (although it may affect the glazing and services). Minor wrapping of the section around the apex is also observed (2 mm inwards), so the flanges of the adjacent cross sections should be bonded and reinforced similar to a vault valley in order to maintain the integrity of the roof.

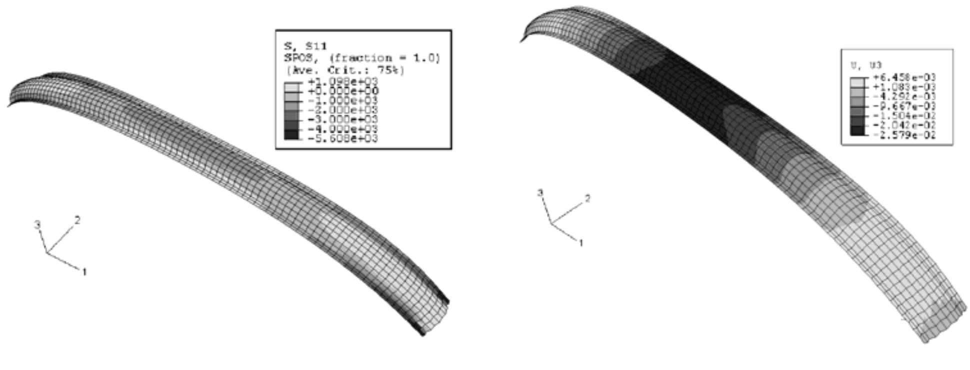


**Figure 9.** Hoop stresses at the extrados [$kN/m^2$] and deflections [m] under self weight

The upper portion of the shell is mainly in compression, with a max stress at the apex 1.8 $N/mm^2$, i.e. below the strength of masonry. This action makes the evaluation of the critical load relevant. The maximum axial force (equal to the total thrust) is 1385 kN and is less than 1740 kN estimated by Sánchez del Rio [1962b]. It is interesting that the arched portion of the cross section receives most of this axial compression, with the flanges in very low stresses. Bending also develops around the haunches of the shell, an area critical in every arch, with a max tensile stress of 1 $N/mm^2$ that is expected to be resisted by the reinforcement at the extrados of the units.

The FE model was also used to assess the behaviour of the isolated shell at wind load. An asymmetric pattern following recommendations by the code of practice was applied, i.e. a positive pressure of 0.5 kN/m$^2$ on half of the vault and a negative pressure of -0.6 kN/m$^2$ at the other half. Figure 10 shows how the arch deforms in combination with its self weight. The max deflection at the apex increased to 27 mm but overall the behaviour was not affected much. This behaviour indicates a high degree of safety but a more careful application of the wind pattern as well as study of the relevant critical load is required. Consequently, other actions need also to be studied, like the response to thermal and moisture changes, which Sánchez del Rio [1962b] assures they have a minor effect. Such issues need to be verified in a more detailed programme of analysis.

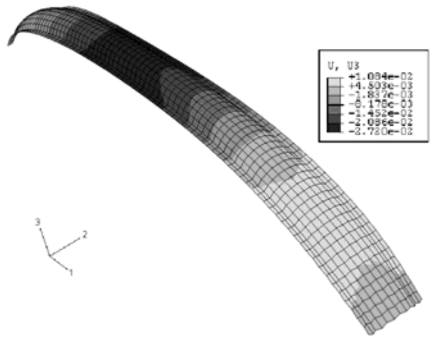


**Figure 10.** Deformation [mm] of a typical arch under wind loads

The discussion on the architectural effects of the solution can focus at the possibilities for openings and natural light, the internal quality of the enclosure as also the treatment of the areas where the shells come closer to and can be appreciated by the visitor (especially the supports). Regarding the openings, further investigation is required on the framing of the major opening at the end and whether it prevents the deflection of 27 mm from bringing the shell in contact with the glazing, as also the differential deformation of the two types of vaults and the extent it may affect the glazing in between them (Fig. 3). The ribs give certainly a rhythmic effect on the roof, especially at daylight, due to a succession of reflections and shadows. When viewed in relation to the buttresses, the effect is quite powerful and the interior of the Hall needs to be kept as free as possible in order to make this aspect an integral part of the visual experience. In addition, the idea of exhibiting the hinges at the buttresses is quite unique as it clearly conveys an appreciation of the huge forces within the fabric.

## 5. WAREHOUSE BUILDINGS

Most of the applications of the wave-voussoir system were in warehouses and factories, a striking resemblance to the work of Eladio Dieste who was part of the next generation of shell builders. Apart from the creation of an uninterrupted enclosed space, other considerations were the minimisation of thermal movement, acoustic and thermal insulation, natural daylight and of course economy in construction. Two of his most ambitious projects, the factory buildings for Río-Cerámica (span L=35 m) and FEFASA (L=30 m), highlight the average spans roofed. Thrusts in these buildings are either contained by a series of pier buttresses supporting the shells or braced by ties within each bay, the choice depending on the activity and the constraints outside the building.

Due to ease of access, a warehouse in Granda, outside Oviedo, is studied briefly in order to appreciate the context of Sánchez del Rio's systems. Information was not available and the geometry was established following a site survey, estimating dimensions that were out of reach. The shed is abandoned (Fig. 11) and the construction period or original use is unknown, although it probably dates after his 1962 papers and the remains inside suggest a kiln operated. The shell roofs a space 23.84 m x 40 m, made of 12 arches spanning L = 23.84 m and a rise F estimated as 4.7m. Each arch is made of 17 wave-voussoir units 4m wide (cf. Fig. 2) and about 0.7 m high.

Using the thrust line method, the critical load and safety against buckling is assessed. From the catenary arches tables [Dieste 1983], the geometry of the cross section and the directrix are evaluated: the aspect ratio of the cross section is Lv/Fv = (4-2·0.85)/(0.7+0.18/2)=2.91, so the second moment around the centroid $Ig$ = 0.3479·t·Fv$^3$ =0.031 m$^4$ . Regarding the directrix, the aspect ratio L/F=23.84/4.7=5 and from the tables: $\ell$ = S/2 = 1.09972 23.84/2= 13.109 m, the springing angle $\varphi_o$ = 40, therefore $\gamma$ = 1.19. From Fig. 7, $\chi$ = 7 and from eq. 4, $q_{cr}$ = 1918 kN/m. The equivalent linear distributed pressure due to self weight q = 16 0.18 4.67 = 13.46 kN/m. As a result, the degree of safety $\gamma_f = q_{cr} /q$ = 142, which indicates a quite large margin of strength.

**Figure 11.** Exterior and interior of the warehouse in Granda, Oviedo

A FE analysis could have assessed the safety factor more accurately, but a dimensional analysis shows that the magnitude is reasonable. A more careful study of the geometry and the major dimensions (L, F, width), reveals that the vaults in Granda are almost a quarter scale of the Sports Hall. If in calculating $\gamma_f$ a proportional factor $\lambda$=4 is applied to all dimensions except the thickness (which should remain the same) than the two factors are inverse proportionally, i.e. the safety factor in Granda should be 4 times that of the Sports Hall or 4 18.7 = 75.

A more detailed analysis of the shell will yield more reliable results, but the comparison of the two case studies gives already some insight to the behaviour and design processes of Sánchez del Rio's hybrid vaults. A scheme could have been established whose structural response and safety was well analysed. Then, the scheme could be proportionally adjusted to the site conditions but only a limited series of proportions could be applied as the units had to follow a modularised construction due to the use of standard ceramic blocks. It is a reasonable hypothesis, which could have allowed the designer to concentrate on the more important stages of manufacturing and construction, but it will have to be verified by further examination of case studies.

## 6. DISCUSSION AND CONCLUSIONS

The innovative system of the wave-voussoir devised by I. Sánchez del Rio was a successful solution to the roofing of large warehouses and factories, achieving efficiencies similar to the work of Eladio Dieste and other shell builders who developed vaulting methods from a careful examination of first principles. Although not as spatially flexible or expressive as the works of the latter, their degree of safety and construction modularity result from the same line of research as contemporary prefabricated or lightweight large shells such as the CNIT or the Duxford Air Force Museum. More research is therefore needed in order to understand what can be transferred from such ideas into modern practice, and this goes even for the earlier work of this important designer.

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