Periodica Polytechnica
Civil Engineering

62(1), pp. 162–172, 2018
https://doi.org/10.3311/PPci.10251

TECHNICAL NOTES

Two Post-tensioned Thin Folded Plates Designed by E. Torroja for San Nicolás Church

Ernesto Fenollosa[1*], Begoña Serrano[1], Adolfo Alonso[1], Fernando Arnau[2]

Received 11 November 2016; Revised 15 February 2017; Accepted 01 March 2017

Abstract
The long-span structure of the San Nicolás church was designed in 1959 by the renowned engineer E. Torroja. His innovative solution consists of two Z-shaped folded plates mutually independent spanning 29 m. In this paper the technical advances used by the engineer to carry out its daring structure are explained. A series of ribs provide stiffness to the slabs facing biaxial bending and torsion and support cantilevered roof. The set of post-tensioned tendons contribute to the strength capacity of the slabs as well as avoid the cracking of concrete. Rotation and the horizontal displacement of the plates are released by the twofold articulation of the main frame. The simplified structural design carried out by the engineer is explained and his results are compared with the structural analysis obtained by a virtual finite element model. The aggressive environment caused corrosion in reinforced concrete elements, but not in the post-tensioned concrete plates.

Keywords
folded plates, post-tensioned tendons, Torroja, F.E. model, corrosion

[1] Department of Continuum Mechanics and Theory of Structures
Universitat Politècnica de València.
46022, Valencia, Spain
[2] Universitat Politècnica de València.
46022, Valencia, Spain
* Corresponding author email: efenollo@mes.upv.es

1 Introduction

San Nicolás church is the posthumous construction of the renowned Spanish engineer Eduardo Torroja Miret [1]. It is a modest church located at the end of the Gandía harbour, on the southern part of Valencia (Spain).

Its innovative structural solution has turned this church into an interesting example of XX Century Spanish architecture [2]. Two thin folded plates independent of each other and supported by the main frames, spanning 29 meter and becoming a unique volume of space without pillars (Fig. 1). There are three continuous stripes through which light penetrates the interior providing the church roof with an incredible appearance of weightlessness. In this way all side chapels placed on the south side are completely open to the nave. On the north side the lack of supports allows the opening up of the temple to the cloister to increase capacity.

Fig. 1 Interior view of San Nicolás church. Own source.

E. Torroja carries out the structural design using only pencil and paper. Thanks to his intuition and expertise he simplifies the behaviour of the plates as two simply supported beams. He notices that the open asymmetrical section with a Z shape produces biaxial bending and torsion even for the gravity loading hypothesis. Its undesirable effect is counteracted by adding a sequence of ribs to reinforce the folded plates [3] and by the advanced post-tensioned tendons system in walls and roofs, developed by the engineer.

This paper will explain the design process followed by E. Torroja and will compare his results with the ones provided by the structural design using software tools. Like the studies performed in other masterpieces of Mid-Century Modern architecture [4, 5, 6], the structural analysis of folded plates under different load combinations will show its resistant behaviour, providing key information for designing of these special structures.

The model of the finite element (F.E.) elaborated is not limited to the folded plates in the nave. Main frames supporting the folded plates, the apse structure and the multiple pre-stressed tendons system are also included. The analysis of the whole model will make possible to test the working process of the twofold articulation at main frames that releases the rotation of the plates and permit the achievement of the desired structural behaviour. It will show the effect produced by the differences between the simply supported beam simplified model used by the engineer and the actual structure built. The effect of the cloister roof columns, the South plate continuation at the apse area, the extension of the plates and one of the ribs to the foundations will be shown.

Post-tensioned tendons have been modelled in their true position, instead of in the resulting curve as can usually be found at the bibliography [7]. This will display the efficacy of the anchors system used that minimize stress concentrations on these thin-walled structures.

Finally the damage to the reinforced concrete structure due to corrosion of the steel bars are displayed. However there was only occasional damage in the prestressed folded plates which reveals that post-tensioning forces keep the compressed concrete and therefore without cracking, by increasing the durability of the structure in aggressive environments.

2 The Church and the Engineer

Eduardo Torroja (1899–1961) has been one of the best Spanish engineers. He is considered along with Franz Dischinger (1887–1953) one of the pioneer using thin-shell structures [8] and he is together with Félix Candela, Anton Tedesko and Pier Luigi Nervi the designer of the most elegant ones. He combined teaching at Civil Engineers School in Madrid with his professional job. He was founder and the first director of the Building Sciences Institute (IETCC), which is dedicated exclusively to the study and research in the field of construction and its materials. He wrote his ideas about structural behaviour in the book "Philosophy of structures" [9] and in many texts he wrote about his thoughts on the interactions between artistic and technical disciplines.

He used the prestressing technique in an innovative way to get audacious structures in both concrete domes as the roof of the Algeciras Market (1933), or the hiperbolic shell of Fedala reservoir (1956) as shells of ceramic bricks in the church of Pont de Suert (1952). Even some authors credited with the invention of prestressing due to Tempul aqueduct, built before the first prestressed concrete structures of Eugène Freyssinet [10]. Tensioning structures allows him to achieve the desired structural behavior while avoiding concrete cracking, ensuring tightness and improving durability.

The idea of the church construction starts from the popular movement leaded by the parish priest: Father Juan Miñana in 1955. He decided to propose the building of the new temple to D. Eduardo Torroja Miret after getting to know the excellent references of the Pont de Suert church in Lérida [7]. This is another interesting construction by the recognized engineer.

The San Nicolás church project [11] was carried out in collaboration with the Architect Gonzalo Echegaray Comba, in 1959. E. Torroja died in 1961, a year before the construction was complete; this gives the building a special bond with its author due to the fact it was his last work.

In addition to the church itself, the complex consists of the Abbey House and the garden cloister between them. It is worth pointing out the modern bell tower with the shape of two sounding boards directed towards population and summer visitors.

The building from the outside is perceived as a unique volume of space with a parallelepiped shape, visible from the harbor. The church nave (Fig. 2) has a trapezoidal shape with the high alter facing east. It has a length of 40 m including baptistery and sacristy, a 13, 5 m. height and 12 and 10 width narthex and alter respectively. Attached to the nave are the side chapels and the sacristy facing south. On the north side the cloister is attached and on the west side the baptistery, chapels and the sacristy.

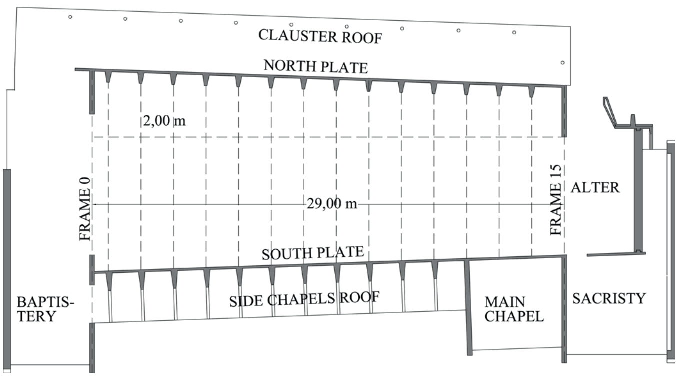


Fig. 2 Plan and structural scheme. Project of San Nicolás church (Echegaray 1959).

3 Description of the structure

3.1 The folded plates

The main part of the structure that defines this building is formed by two 15 cm thick concrete folded plates. They constitute both longitudinal façades of the temple resting on the main frames. This system allows spanning 29 m while the construction remains free and without support, as Torroja already did in the Frontón de Recoletos roof [12].

Both thin concrete plates acquire inertia due to the great wall height and to its end horizontal folding. They are Z shaped, its web is formed by the façades and the top flange by the roof of the nave. The bottom flange consists of the roof of the side chapels in the South plate and the cloister roof for the North one. However, the South plate inferior slab is not connected directly with the wall, instead it is just joined through the ribs. That is why it is usual to name this plate shape as inverted L.

E. Torroja improved the structural behaviour of thin plates by means of several constructive solutions like serie of variable section ribs, placed at intervals of 2,00 m. The temple is characterized by these ribs appearing inside the nave at the North plate and outside at the South plate (Fig. 2). The ribs lengthen below the roof of the chapels on the south façade while they are removed at the cloister roof on the north façade, they are replaced by reinforced concrete supports. The ribs inclusion is justified as a stiffening element in front of biaxial bending and torsion loads caused by the asymmetrical and uneven geometry of the plates and in order to avoid elastic instability [13, 14, 15]. However, they play a second role as rigid cantilever elements where the horizontal plates of the roof of the temple and the chapel are supported [16].

3.2 Prestressing system of the folded plates

On the other hand, live loads were introduced in the slabs by prestressed steel tendons (Fig. 3), composed by groups of three wires according to Barredo system [17] (explained in the next section). In the vertical walls and the roof of the side chapels 3 tendons are arranged by each duct, while in the roofs of the church and the cloister are arranged 2 tendons per duct. Furthermore, in the case of vertical plates, each parabolic guideline corresponds to two ducts flowing almost parallels, vertically separated 4 cm and with the axis 4.5 cm from the outer side one of them and from the inner side other.

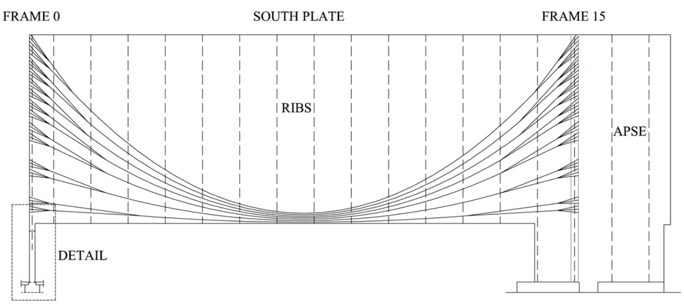


Fig. 3 Post-tensioning tendons system in South folded plate.

Tendons were tensed from both sides due to the curved path of the cables. The anchors are arranged on the edges of the folded plates, perpendiculars to the parable guideline and with additional reinforced bars to support the stress concentration.

Post-tensioning forces located in the vertical surface are counteracting most of the dead load, reducing biaxial bending and minimising tensile stresses that appeared on the lower part of the plate to acceptable values by the concrete tensile strength. Steel tendons are placed on the nave and chapels roof producing two horizontal forces in an opposite direction which reduce torsion efforts (Fig. 4).

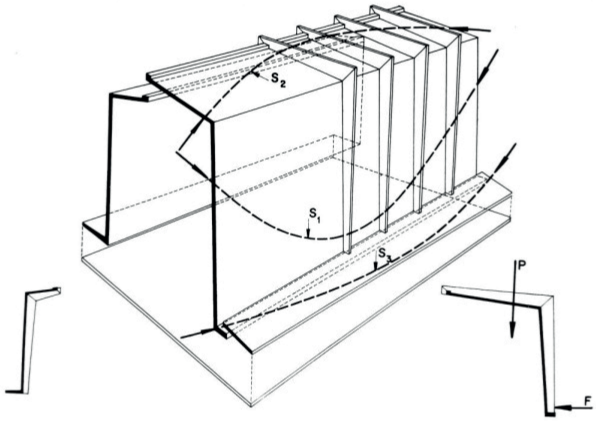


Fig. 4 Structural diagram.

Furthemore, as mentioned E. Torroja [16], the post-tensioning forces compress the concrete slabs, ensuring sealing without waterproofing elements, including roofs only 10 cm thick. The sealing increases the durability of the folded plates because in his almost sixty years , post-tensioned elements have not showed signs of corrosion, as if have other structural elements of reinforced concrete that required rehab at least twice [18]. Therefore, the prestressing of the slabs serves two functions: corrects undesired effects deriving its open section and provides the necessary sealing against water, being located the building in a marine environment.

3.3 Prestressing Barredo system

His deep knowledge of the behavior of structures and their great insight for the design and application of new techniques are evident in its collaboration with the civil engineer Ricardo Barredo for the development of various devices and post-tensioning systems after the second world war [19]. The Barredo Method consists of three wires per tendon with an anchoring system formed by an inner wedge which imprisons them against cone service (Fig. 5) [20]. Over time multiple variants that resulted in several patents were made, including the CGC anchor system.

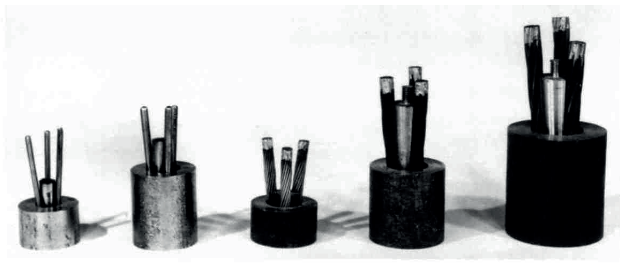


Fig. 5 Anchors of prestressing Barredo system.

Tensioning the tendons it is carried out by using special hydraulic jacks, stretching the three wires independently. The simultaneous tensioning of all wires housed in the same duct allows a work faster but requires a hydraulic jack larger and more unwieldy. Furthermore, the possibility of differential extensibility or any slippage of a wire during tensioning can cause overloading not provided of the rest. These drawbacks are overcome by the Barredo system using a hydraulic jack with three interconnected pistons could have different paths so that all wires are subjected to the same force.

The Barredo Method competed internationally with the most prestigious systems like BBRV, Freyssinet, CCL, Mangel, VCL, LEOBA or Dischinger [21]. Eduardo Torroja, responsible for its creation, used it in many of his works as the folded plates at San Nicholas church, whose structural effects and benefits in tightness and durability are shown below.

3.4 The main frames

Folded plates of the façades are rigidly supported by two rigid frames; Frame 0 placed next to the main access and Frame 15 in front of the high alter.

Frame 0 (Fig. 6) has two singular elements. One of them is the joint placed on its upper side which releases the rotation at the ends and makes both façade's plates independent. However, this joint has been post-tensioned allowing wind loads to be transferred. The other element is the twofold articulation Freyssinet type, one of them is placed at the lower end of the frame and the other placed below the façade's plates. That is why this frame behaves as a pendulum by releasing rotation and the horizontal displacement of the plates and simplifying its calculation model by simulating it as a simply supported beam. In addition it allows the expansion of the plates due to thermic variations.

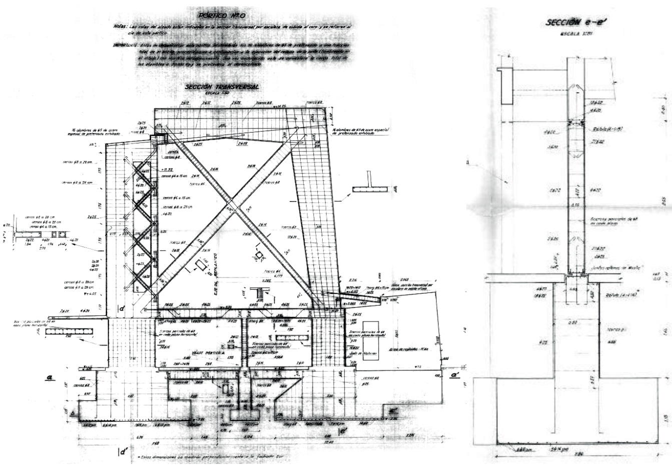


Fig. 6 Elevation and section of Frame 0.

Both frames are given great rigidity in their own plane in order to consider null the displacements of the plates under wind loads. However in the transverse plane, frame stiffness is considered negligible compared to the significant stiffness of the folded plate.

4 Structural analysis carried out by E. Torroja

4.1 Previous considerations

Materials properties

According to the information obtained in the structural analysis in the original project dossier, the compressive strength considered for concrete was f_{ck} = 13,5 MPa with a safety factor γ_c = 1,6. In addition E. Torroja admitted tensile stresses exist when not overcoming a 0,5 MPa concrete tensile strength.

He used a steel with an elastic limit of 240 MPa and a safety factor of γ_s = 1,2 for reinforcing bars and a steel with an elastic limit of 1500 MPa, a safety factor of 1,2, a percentage of prestress of 90% and 20% of long-term losses for pre-stressing forces.

Loading hypothesis.

E. Torroja [16] put the plates under three loading hypothesis:

- dead loads constituted by the folded plates and ribs weight, the roofs own weight (1 kN/m^2), the enclosures and claddings.
- live loads of 0,6 kN/m^2 that are applied over the roofs,
- wind loads over the façades with a pressure of 1,5 kN/m^2 and a suction of 0,5 kN/m^2.

In addition he takes into account live loads given by post-tensioned tendons. The force applied at each tendon was 176,72 kN.

Torroja takes into account just one load hypothesis in the North plate, the one that results from the simultaneous application of dead loads plus prestressing forces and wind force acting in north-south direction. However he considered four load hypotheses in the South plate:

- dead loads plus prestress forces
- dead loads, prestress forces and live forces
- dead loads, prestress forces and north-south wind
- dead loads, prestress forces and south-north wind

4.2 Torroja design model

E. Torroja addresses both plates design independently. He simulates their behaviour to the simply supported beams behaviour; assuming Frame 15 has a pin support and Frame 0 a roller support, thanks to the twofold articulation referred to above.

He takes on the Z shaped plate section on the North wall while he takes into account the inverted L shaped plate section on the south wall because the vertical surface is not connected to the chapels roof. From the plate geometry and the ribs, Torroja [16], determines the center position (Fig. 7), the area, the inertias and the inertia product of the sections.

E. Torroja calculates the normal stress values for four points in the North plate and for three points in the South plate (Fig. 8), under the load hypothesis described.

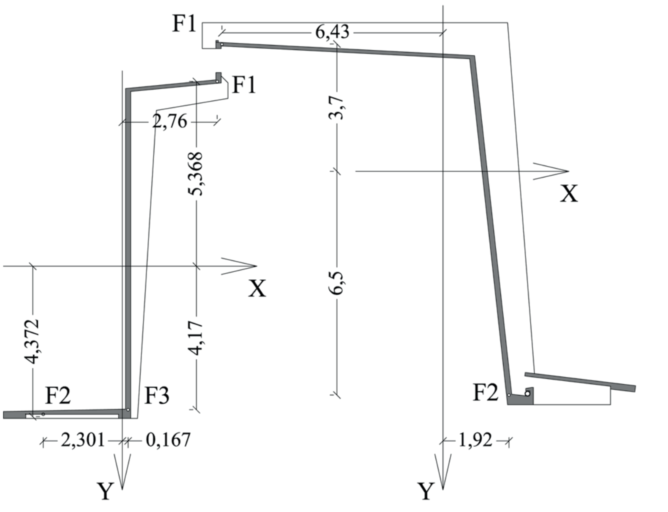


Fig. 7 Cross section of the folded plates. Loads centre and prestressing forces. Source: Prepared by the authors from the data provided by the original project design dossier.

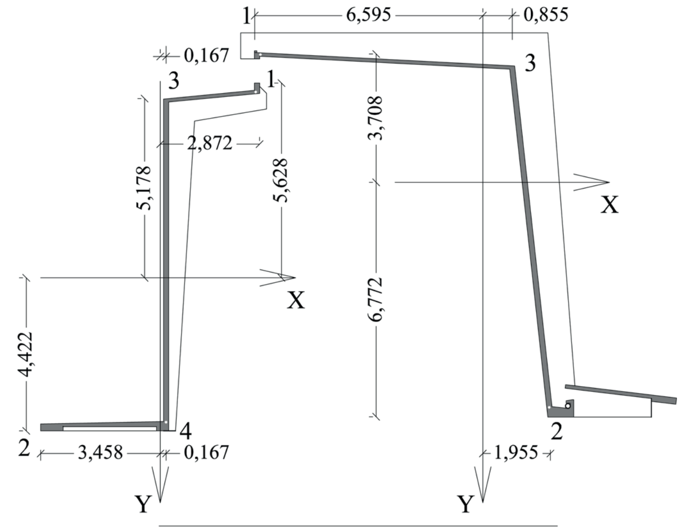


Fig. 8 Cross section of the folded plates. Position of representative points. Source: Prepared by the authors from the data provided by the original project design dossier.

4.3 Obtaining stresses

E. Torroja carried out the normal stresses evaluation assuming the plates have an elastic and linear behaviour. By accepting Navier-Bernouilli hypothesis, he uses the classic strength of materials equation for the non-main inertia axes. He didn't use any structural optimization process as is usual nowadays [22].

By keeping nomenclature, reference system shown in Fig. 7, and subscripts used by the engineer, the equations developed are Eq. (1) for the North plate and Eq. (2) for the South plate.

The value of the bending moments with respect to the vertical bending plane (MV) and horizontal bending plane (MH), as well as the prestressing forces (F) used in the design are collected in Table 1.

Further information about the design process carried out by E. Torroja can be obtained at the work of Nuñez-Collado et. Al. [23].

Unlike the roofs of the Frontón de Recoletos and the Algeciras Market, in this case any reduced model of the structure was made in order to check the analytical results obtained.

5 Finite element model

5.1 Model description

A virtual model of the temple structure has been made based on the building geometry: the folded plates and the stiffening elements, the frames that the plates rest on and the remaining apse structure. All of them had been split up by Finite Element (F.E.) triangular plane elements of 40 cm of medium size (Fig. 9). Frame 0 diagonals and the cloister supports had been modelled by bars.

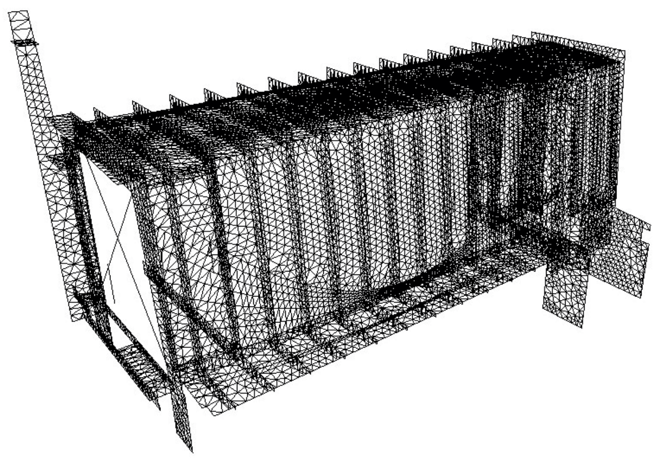


Fig. 9 F.E. structural model.

The inclusion of the main frames in the model is considered to be essential for comparing the simplified analysis made by Torroja with the structure actually built. They had been modelled by means of triangulated F.E. of 30 cm thickness.

$$\text{North plate:}\ \sigma = \frac{\left(M_V^{'} - 4.17 \cdot F_3\right)\cdot\left(y \cdot I_y - x \cdot I_{xy}\right) + \left(M_H^{'} - 0.167 \cdot F_3\right)\cdot\left(x \cdot I_x - y \cdot I_{xy}\right)}{I_x \cdot I_y - I_{xy}^2} - \frac{F_1 + F_2 + F_3}{W} \tag{1}$$

$$\text{South plate:}\ \sigma = \frac{\left(M_V^{'} - 3.70 \cdot F_1 - 6.50 \cdot F_2\right)\cdot\left(y \cdot I_y - x \cdot I_{xy}\right) + \left(M_H^{'} + 6.43 \cdot F_1 - 1.92 \cdot F_2\right)\cdot\left(x \cdot I_x - y \cdot I_{xy}\right)}{I_x \cdot I_y - I_{xy}^2} - \frac{F_1 + F_2}{W} \tag{2}$$

Table 1 Bending moments and prestressing forces stresses.

	M_V (kN·m)				M_H (kN·m)				F (kN)		
Plate	Dead load	Live load	Wind (N-S)	Wind (S-N)	Dead load	Live load	Wind (N-S)	Wind (S-N)	F1	F2	F3
North	10220	313.9	-159	-159	0	0	1407	-1029	345.6	41.45	1797.0
South	15877	629.0	-622	-622	-642.3	93.9	-648	286	622.2		2211.4

Frame 0 articulation, below its meeting with the folded plates had been modelled by F.E. in a row of half the thickness (15 cm) and the meeting of the frame with the foundations by means of a theoretical articulation. The joint at the top of the Frame 0 (Fig. 6) has been modelled; the two parts of the frame have been connected through a single vertex.

Frame 15, next to the high alter, has no articulation, not even an expansion joint and its meeting with the foundations has been modelled as fixed, like the other elements reaching the foundations

5.2 The software used and other design data

The structural analysis of the model has been run by the design software in a linear regime ANGLE [24]. Shell elements are assigned a behaviour as a plate (bending moments) and membrane (axial and shear forces) [25].

The F.E. have been assigned to the calculation parameters of concrete used by E. Torroja: an elasticity modulus of 24000 MPa and a Poisson´s ratio of 0.20.

The software directly assesses the structural elements self-weight, taking into account a specific concrete weight of 24 kN/m^3, as E. Torroja used it. The remaining loads of constructive elements, live and wind loads have been applied through linear or superficial forces with the same values and hypotheses as the ones used by the engineer.

Most of the performances explaining the church folded plates post-tensioning system, replace the tendons as a whole by a single curve representing the resultant forces. Modelling this way would lead inevitably to a stress concentration around the anchoring. In the developed research the authors had introduced different tendons in their real position (see Fig 3) and a pre-stressing force has been applied at each one of them. The stress along the cable has been assessed following the ACI [26] criteria and taking into account a global loss of 20%, already considered by E. Torroja. This loss includes deferred instantaneous losses caused by friction and the anchors. In addition a compression force has been applied that is tangent to the curve at the tendons anchoring point.

6 Results discussion

6.1 Stresses and strains caused by dead loads

Tensional state of the folded plates under dead loads is shown in Fig. 10. The results are as expected for a simply supported beam: compressional stress on the upper side of the plate and tensile stress on its lower side.

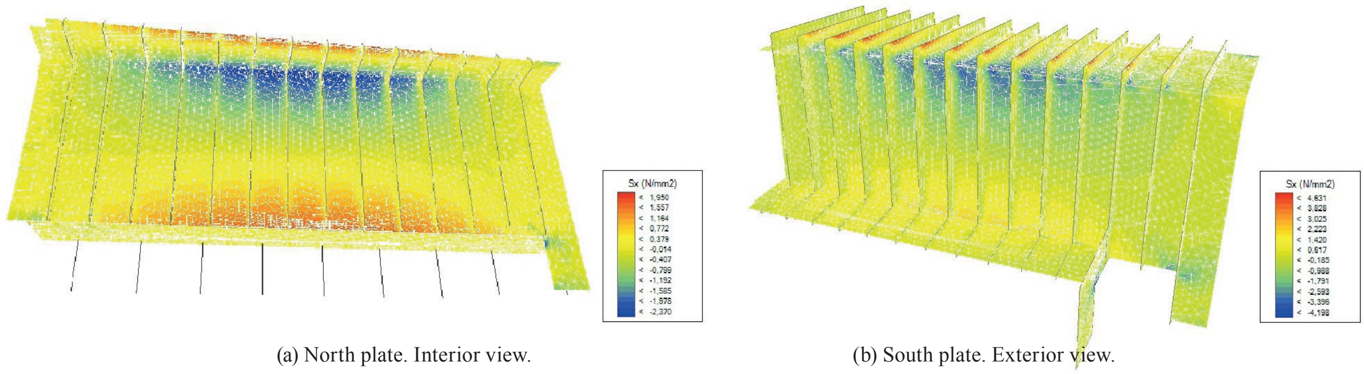
The vertical plate is performing as a wall beam and it is reinforced by the roof plates. In the same figure a stress concentration at the plate vertex and at the corner next to the supporting area can be observed.

The normal stresses values obtained by the F.E. model are compared to (Table 2) the ones from the design carried out by E. Torroja (Eqs. 1–2). The most significant difference is at Point 2 of the North plate where a tensile stress is obtained instead of the expected compressional stress.

Table 2 Normal stresses (MPa) caused by dead loads. Results comparison.

	North plate				South plate		
	Point 1	Point 2	Point 3	Point 4	Point 1	Point 2	Point 3
E.T.	0.46	-2.10	-3.16	3.15	3.80	5.63	-5.03
F.E.	1.95	0.24	-2.37	1.69	2.25	2.62	-3.76

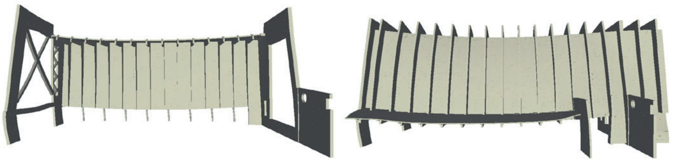
The explanation lies in the differences between the simplified design model used by Torroja and the real construction reproduced by the F.E. model. The images of the elastic deformation of both plates show what happened.



(a) North plate. Interior view. (b) South plate. Exterior view.

Fig. 10 Normal stresses due by dead loads.

Frame 0 (Fig. 11-a) shows the expected deformation: top and bottom joint rotation allowing longitudinal plate displacement. The support behaves as a roller. The low stiffness of Frame 15 (picture on the right) regarding to the folded plates causes it to behave as a pinned support.



(a) North plate. Interior view. (b) South plate. Exterior view.

Fig. 11 Deformations produced by dead loads.

Nevertheless, there are several elements distorting the model. The most obvious one is the South plate continuity in the apse façade. Besides, with a lower repercussion, there are several ribs in the south façade and even some portion of the plate itself in both façades that restrict the free rotation of the ending part next to the high alter. Those elements provide a stiffness that prevents the plate from free rotation on Frame 15. That is why the plates behave similar as the simply supported beam at one side and fixed at the other. This is especially evident in the South plate (Fig. 11-b). In this way the obtained stresses are smaller than the expected ones.

Finally the series of reinforced concrete supports placed below the cloister roof (North plate) explains the previously mentioned sign change of the stresses in this area.

The role of the ribs can be observed in Fig. 12. The roof plate is rigidly fixed to the main frames 0 and 15 and because of their stiffness cannot be displaced vertically. But at the central area, the plate remains fixed by the ribs with a fundamental work like a cantilever (as it can be seen in Fig. 10-b, tensile stresses on the upper side of the ribs). Maximum vertical displacement reaches a value of 18.4 mm. for the analysed dead load. As E. Torroja anticipated, the ribs play a double role as plate stiffeners and as rigid cantilever elements supporting the roofs.

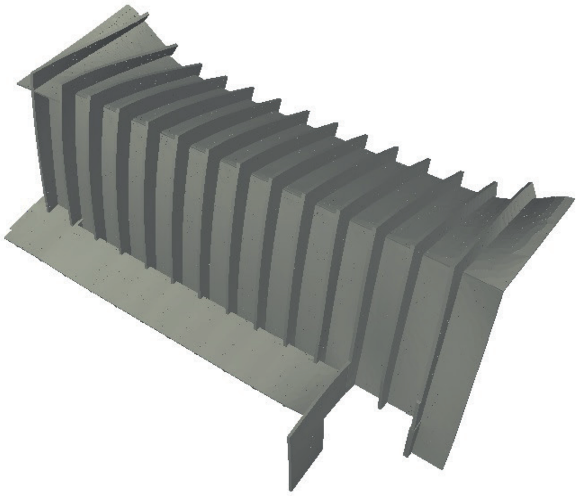


Fig. 12 South plate. Deformations produced by dead loads.

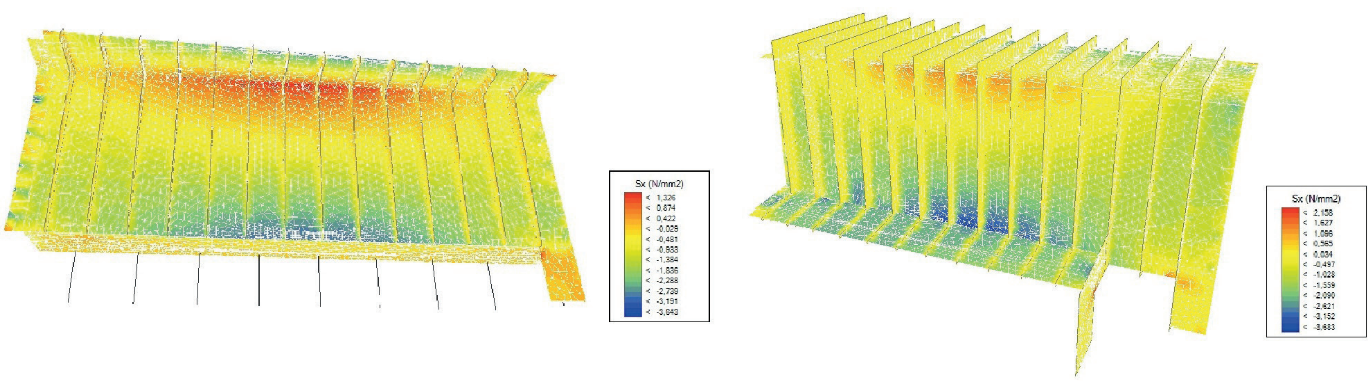
6.2 Stresses caused by the post-tensioned tendons

Normal stresses values obtained with the F.E. model (Fig. 13) are compared in Table 3 with the ones from the design made by E. Torroja. There is a closer approximation between the E.T and F.E. values in north folded plate than in the south one. Absolute values are smaller than the ones provided by E. Torroja and a sign change in the stress is produced in the North plate at point 2. The reasons are the same to the ones mentioned for dead loads.

Table 3 Normal stresses (MPa) produced by the post-tensioning forces.

	North plate				South plate		
	Point 1	Point 2	Point 3	Point 4	Point 1	Point 2	Point 3
E.T.	-1.84	0.91	1.35	-3.69	-3.60	-5.39	2.30
F.E.	-2.75	-1.01	1.32	-2.99	-2.49	-3.64	1.32

The compression stresses produced at the bottom side of the plates by the post-tensioned tendons is slightly higher to the tensile stresses caused by dead loads, while tensile stresses at the upper side do not reach the compression values because of the dead loads (Table 4). If both hypotheses loads are taken into account, as both loads are dead, all the points in the plates are under compression.



(a) North plate. Interior view. (b) South plate. Exterior view.

Fig. 13 Normal stresses produced by post-tensioning forces.

Table 4 Normal stresses (MPa) caused by dead loads. Results comparison.

	North plate				South plate		
	Point 1	Point 2	Point 3	Point 4	Point 1	Point 2	Point 3
Dead load	4.30	1.22	4.12	1.20	-4.63	-1.03	-4.17
Pre-stressing	-5.62	-1.98	-5.54	-1.94	1.91	0.23	1.88

Due to the asymmetric and open section of the folded plates a torsional moment takes place even for the dead loads. This effect can be seen in the North plate deformation shown in Fig. 14 and it becomes clear by the difference in horizontal displacements of the representative points. The torsional problem is corrected through the introduction of post-tensioned tendons at the web and flanges of the folded plates.

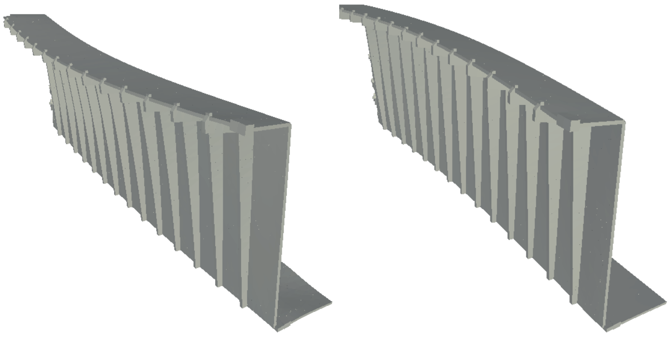


Fig. 14 North plate. Deformations under dead loads and post-tensioning forces.

Through the arrangement of multiple wires, with a division with three anchors for each tendon, following the Barredo system, prestressing forces are distributed. The stresses concentration shown by Nuñez-Collado [23] does not exist.

Higher stresses at the anchoring area are located at the access ending side of the North plate. Maximum values are -3.64 MPa. That is because this plate goes beyond Frame 0 plane the anchoring takes place over the plate of 15 cm thickness. In the others the anchoring is made at the main frames plane which had a 30 cm thickness, and spreading the stresses out in the most homogeneous way.

6.3 Deformations caused by wind loads

Horizontal displacements are shown in Fig. 15 by a color map. It shows how the Frames 0 and 15 are rigid enough in their own plane as to be considered as fixed. Longitudinal displacement does not exceed 1 mm in any of both frames upper vertex (Table 5).

The behaviour under horizontal wind loads is like two great deep beams (church and chapels roofs) supported by the main frames. A vertical plate performs as a two way slab fixed in its perimeter to the roof plates and to the main frames. Facing this load hypothesis, the ribs also play a stiffening role, reducing the horizontal displacement of the folded plates.

Table 5 Horizontal displacements (mm) produced by wind loads. (North-south displacements are considered positive).

		North plate				South plate		
Wind	Sect.	Point 1	Point 2	Point 3	Point 4	Point 1	Point 2	Point 3
Direc. North South	Fr. 0	0.84	0.30	0.82	0.27	1.03	0.38	0.99
	Cent.	4.44	2.43	4.47	2.44	1.16	1.39	1.19
	Fr.15	0.88	0.17	0.87	0.19	0.87	0.02	0.85
Direc. South North	Fr. 0	-0.91	-0.31	-0.89	-0.27	-1.03	-0.37	-1.00
	Cent.	-3.23	-1.84	-3.24	-1.84	-1.33	-1.76	-1.39
	Fr.15	-0.85	-0.15	-0.85	-0.17	-0.85	-0.02	-0.83

Horizontal displacements are bigger in the North plate due to the small dimension of the church and cloister roofs. The cloister supports insertion makes maximum deformations to be located in the upper side of the plate. On the contrary the South plate maximum displacements are located in the bottom side, due to the bigger dimension of the temple roof and to the low impact on the roof of the chapels linked to the vertical plate only by means of the ribs (see Fig. 15).

The extension of the South plate by the high alter area does not produce a significant interference.

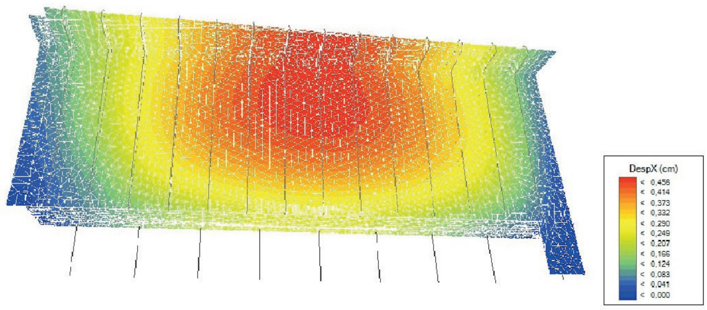
6.4 Load combinations

E. Torroja tested the structural design validity by means of putting tensions under different load combinations. While the North plate only considers one load combination, the South plate takes into account four different combinations. Results are indicated in Table 6, next to the values obtained in F.E. model analyzed at this paper.

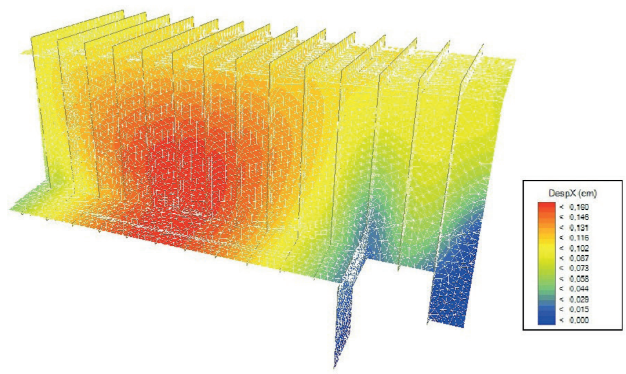
Table 6 Normal stresses (MPa) produced by the post-tensioning forces. Results comparison.

Plate	Point	Dead load + pre-stressing		Dead load + pre-stressing + live load	
		E.T.	F.E.	E. T.	F.E.
North	1	-1.38	-0.85	-1.37	-0.67
	2	-1.19	-1.26	-1.26	-0.76
	3	-1.81	-2.61	-1.91	-1.16
	4	-0.54	-1.43	-0.44	-1.23
South	1	0.20*	-0.24	0.40*	0.35
	2	0.25*	-1.02	0.48*	-0.44
	3	-2.73*	-2.44	-2.95*	-3.11

No tensile stresses are obtained in any of the points analyzed when dead loads are combined with pre-stressing forces and live loads. Tensile stresses are only obtained under wind loads at the points expected by the engineer.



(a) North plate. Interior view.



(b) South plate. Exterior view.

Fig. 15 Horizontal displacements caused by wind loads.

7 Maintenance works in the building

The first intervention of maintenance of the church was made in 1977, supervised by Fernández Canovas, regular contribuitor at Eduardo Torroja Building Sciences Institute. Longitudinal cracks were detected in all external ribs caused by spalling concrete cover due to corrosion of steel. The intervention consisted of eliminating the damadeg coatings, clean and protect the reinforcements and replace the covers with the repair mortar. Therefore, a final solution was not raised.

In 1997, the new detected damages prompted a comprehensive assessment of the conservation state of the building, which served as the basis to a complete rehabilitation of the church, on the occasion of the 50th anniversary of the construction of the building [18]. The technical report carried out by Intemac Laboratories suggests that the main structural damage appeared in reinforced concrete elements, especially in the external ribs, fronts of slabs and columns located in the cloister. This damage consisted in a generalized concrete cracking caused by the corrosion of the steel reinforcements. The cause of such corrosion was favored by three factors:

- Poor quality mortar with low cement content and high porosity
- Thin coating thicknesses and high density of reinforced bars that it difficult a homogeneous concrete work.
- Highly aggressive environment, due to the nearby sea, with high salinity and the presence of concentrations of chlorides in the concrete, derived from the contribution of the external environment, but also, because these chlorides were present in the mass of concrete, due to the use of unwashed sea sand.

It is important to note that in the plates, compressed by the action of the prestressing forces, the concrete cracking was lower, reducing consequently exposure of the reinforcement to the outside environment. The technical report was the base of the rehabilitation project developed by the architect Ignacio Lafuente Niño. The reparation process consisted in:

Removal of damaged concrete, mechanically and water jet high pressure.

- Cleaning the concrete reinforcement by sandblasting silica to remove oxides from corrosion. Later, the reinforcement and the concrete were cleaned by water jet at medium pressure.
- Protection of the reinforcement by applying two coats of paint passivating.
- Replacement the cover to reinforcement using cement mortar modified with polymers, applied by hand, spraying or projected.
- Waterproofing flat roofs over concrete slabs, renovation of rainwater drainage system and water-proof treatment of the facings.

The structural repair works and other construction elements and facilities, were carried out between 2002 and 2006, so they could celebrate the 50th anniversary of the church with a completely renewed image.

8 Conclusions

Through this paper the behaviour of folded plates with Z shape section which make up the structure of San Nicolás church located in Gandía has been explained. It is an innovative solution, due both to the use of the reinforced concrete thin folded plates supported by two main frames and the constructive solutions used in it. A ribs series provides stiffness to the plates to face bending and to avoid the effects of instability. A post-tensioned tendons system placed on the roofs reduces the torsion effects due to the folded plates open section.

One F.E. model has been elaborated. It reproduces the real structure built including the folded plates with the post-tensioned tendons, the twofold articulated main frames, the apse and the cloister supports.

Hand calculations made by E. Torroja using a simplified model of a simply supported beam have been compared with the F.E. model, for different loads combinations. Given the restrictions of the original design, the correlation between the two options, is good.

As far as dead loads are concerned, the South plate continuity in the apse provides stiffness to the end sides, reducing stresses on the calculation as the simply supported beam. The expected values of the stresses of the roof edge are altered by the cloister supports. The ribs add stiffness to bending and give support to the cantilevered horizontal plates of the roofs.

In relation to post-tensioned load, different tendons in their real position had been introduced in the study developed and for each one of them a pre-stressing forces has been applied. It has been shown that pre-stressing reduce the torsional effects due to the open section of the folded plates. The multiple tendons system and the anchoring elements used avoiding stress concentration at the edges of the plates.

For the most unfavorable load combination, the stresses are far from the material strength capacity. For all load combinations analyzed, folded plates work with compressive stresses by prestressing forces. Only a few points are submitted to slight tensile stresses.

An aggressive environment due the situation of the building, caused the corrosion of reinforcing steel causing concrete cracking. Corrosion was accelerated by the lack of maintenance, the presence of chlorides in the mass of concrete and the little covers of the reinforced bars.

Inspection and damage assessment by the Intemac laboratories show that the main damage occurred in reinforced concrete elements located outside, such as ribs or columns the cloister. Only occasional damages were detected in prestressed folded plates. This reveals that post-tensioned tendons keep the compressed concrete and therefore without cracking, preserving their reinforced bars of corrosion, despite scarce covers. Thus, is evidenced that the post-tensioned technique is a great advance to increase the durability of the structure, especially in aggressive environments.

References

[1] Corres, H., Leon, L. "Eminent Structural Engineer: Eduardo Torroja (1899–1961)". *Structural Engineering International*, 22(4), pp. 581–584. 2012. https://doi.org/10.2749/101686612X13363929517938

[2] Fundación docomomo Ibérico. "Documentation and conservation of the architecture and urbanism of the modern movement". 1925–1965, Official Associations of architects of Spain. 2013. http://www.docomomoiberico.com/index.php?option=com_k2&view=item&id=794:parroquia-de-san-nicolas&lang=en

[3] Antuña, J. "The evolution of the work of Eduardo Torroja: Shell roofs with and without reinforcement rings.". *Proceedings of the Second International Congress on Construction History*, pp. 179–194. Cambridge University, UK. 2006. http://www.arct.cam.ac.uk/Downloads/ichs/vol-1-179-194-antuna.pdf

[4] Ramakrishna, A., Raghavan, N., Desai, J. C. "Folded-Plate Hangar Roof Extension, India.". *Structuctural Engineering International*, 7(3), pp. 173–174. 1997. https://doi.org/10.2749/101686697780494879

[5] Draper, P., Garlock, M. E. M., Billington, D. P. "Finite-element analysis of Félix Candela's chapel of Lomas de Cuernavaca". *Journal of Architectural Engineering*, 14(2), pp. 47–52. 2008. https://doi.org/10.1061/(ASCE)1076-0431(2008)14:2(47)

[6] Adriaenssens, E., Brown, N., Lowinger, R., Hernandez, J. "Structural Analysis of Reinforced Concrete Folded Hyperbolic Paraboloid: A Case Study of the Modern Miami Marine Stadium.". *International Journal of Architectural Heritage: Conservation, Analysis, and Restoration*, 8(4), pp. 498–516. 2014. https://doi.org/10.1080/15583058.2012.694967

[7] Fernández J. A., Navarro, J. R. *Eduardo Torroja Ingeniero–Engineer.* Pronaos, Madrid, Spain. 1999. Bilingual edition in Spanish - English. http://www.naoslibros.es/libros/torroja-eduardo-torroja-ingeniero-engineer/978-84-85941-40-7/

[8] Krivoshapko, S. N., Bock, C. A., Mamieva, I. A. "Chronology of Erection of the Earliest reinforced concrete shells.". *International Journal of Research and Reviews in Applied Sciences*, 18(2), pp. 95–108. 2014. http://www.arpapress.com/Volumes/Vol18Issue2/IJRRAS_18_2_01.pdf

[9] Torroja, E. *Philosophy of structures*. University of California Press, Berkeley, USA. 1967. Third edition. https://books.google.es/books/about/Philosophy_of_Structures.html?id=ni_HN_IhVt4C&redir_esc=y

[10] Fuzier, J. P. "Eminent Structural Engineer: Eugène Freyssinet (1879–1962)". *Structural Engineering International,* 17(3), pp. 264–265(2). 2007. http://dx.doi.org/10.2749/101686607781645851

[11] Echegaray, G. *"Proyecto de Iglesia parroquial y casa abadía en el Grao de Gandía. [Exp. 855]"*. Project of a church and an abbey in Gandia's harbor, 1959. (in Spanish)

[12] Lozano-Galant J. A., Paya-Zaforteza, I. "Structural analysis of Eduardo Torroja'sFrontón de Recoletos' roof", *Engineering Structures*, 33(3), pp. 843–54. 2011. https://doi.org/10.1016/j.engstruct.2010.12.006

[13] Joó, A. L., Ádány, S. "FEM-based approach for the stability design of thin-walled members by using cFSM base functions". *Periodica Polytechnica Civil Engineering*, 53(2), pp. 61–74. 2009. https://doi.org/10.3311/pp.ci.2009-2.02

[14] Szychowski, A. "A theoretical analysis of the local buckling in thin-walled bars with open cross-section subjected to warping torsion". *Thin-Walled Structures*, 76, pp. 42–55. 2014. https://doi.org/10.1016/j.tws.2013.11.002

[15] Kowal, Z., Szychowski, A. "Experimental determination of critical loads in thin-walled bars with Z-section subjected to warping torsion". *Thin-Walled Structures*, 75, pp. 87–102. 2014. https://doi.org/10.1016/j.tws.2013.10.020

[16] Torroja, E. Proyecto de estructura para una iglesia parroquial. Memoria y planos. Structural project of a church.Memories and plans.1960. [in Spanish]

[17] Barredo, R. "Pretensado Sistema Barredo." "Barredo prestressin system." *Informes de la Construcción*, nº 70. 1955. (in Spanish)

[18] Serrano-Lanzarote, B., Fenollosa, E., Arnau, F. "Evaluación de daños y actuaciones de rehabilitación en la iglesia de San Nicolás de Eduardo Torroja (Gandía, 1962)". "Damage assessment and rehabilitation works in the church of San Nicolas de Eduardo Torroja (Gandia, 1962)". *Informes de la Construcción*, 68(541), e130. 2016. (in Spanish). https://doi.org/10.3989/ic.14.139

[19] Sanabra, M., Capellà, J. "The four ages of early prestressed concrete structures". *PCI Journal, 59*(4), pp. 93–121. 2014. http://www.pci.org/uploadedFiles/Siteroot/Publications/PCI_Journal/2014/DOI_Articles/JL_Fall_2014-13.pdf

[20] Llorente Zurdo, M. P., Anaya Díaz, J., de Miguel Sánchez, M. "An approach to patents of prestressed concrete in 20th Century's architecture". *Structures and Architecture: Concepts, Applications and Challenges*, (Cruz, P. J. S. (Ed.)). Taylor & Francis Group, London, pp. 912–919. 2013. https://doi.org/10.1201/b15267-128

[2] Anaya, J., Azpilicueta E., Casinello, P. "Historical Patents and the Evolution of Twentieth Century Architectural Construction with Reinforced and Pre-Stressed Concrete". *Proceedings of the Second International congress on construction history*, Queens' College, Cambridge University. 29 Mar., 3 Apr., 2006. pp. 1719–1739. 3006. http://www.arct.cam.ac.uk/Downloads/ichs/vol-2-1719-1740-anaya.pdf

[22] Balogh, B., Lógó, J. "Optimal design of curved folded plates". *Periodica Polytechnica Civil Engineering*, 58(4), pp.423–430. 2014. https://doi.org/10.3311/PPci.7856

[23] Nuñez-Collado, G., Garzon-Roca, J., Paya-Zaforteza, I., Adam, J.M. "The San Nicolas Church in Gandia (Spain) or how Eduardo Torroja devised a new, innovative and sustainable structural system for long-span roofs". *Engineering Structures*, 56, pp.1893–1904. 2013. https://doi.org/10.1016/j.engstruct.2013.08.003

[24] ANGLE. Structural Analysis Software for Finite Element, Developed by A. Alonso, Department of Continuum Mechanics and Theory of Structures, Universitat Politècnica de València, Spain. 2016.

[25] Min, C.S. "Design and ultimate behavior of RC plates and shells: two case studies". *Structural Engineering Mechanics*, 14(2), pp. 171–190. 2002. https://doi.org/10.12989/sem.2002.14.2.171

[26] ACI Committee 318. Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (318R-14), American Concrete Institute, Farmington Hills, Michigan, USA. 2014.