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PRESTRESSSED CONCRETE USAGE IN CHURCH BUILDINGS CONSTRUCTION IN POLAND

ZASTOSOWANIE BETONU SPRĘŻONEGO W PROJEKTACH BUDYNKÓW KOŚCIOŁÓW W POLSCE

Abstract

Prestressed concrete was used to raise building constructions in Poland at the beginning of 1950s. Firstly, large span post-tensioned roof girders were made in industry halls, warehouses, airport hangars ....and so on, and also they were used to construct large capacity liquid tanks and bridges. The new form of post-tensioning, which means unbonded tendons, was used in buildings after 2000. Lately, the first time in Poland, prestressing has been used in project of two churches. As a result of using the unbonded tendons attractive forms were received. The slender prestressed slab supported on the steel "sapling" was constructed in the first of them. A prestressed concrete circumferential ring allows to create a delicate structure of timber girders in the second church. In this paper, these solutions are presented in detail together with the obtained benefits and proper calculation results.

Keywords: church, post-tensioning, prestressed concrete, unbonded tendon

Streszczenie

Istniejący w projektowaniu kościołów trend dążący do tworzenia delikatnych i smukłych form wymusza poszukiwanie nowoczesnych form konstrukcyjnych. Wobec powyższego po raz pierwszy w Polsce zastosowano cięgna sprężające bez przyczepności w projektach dwóch kościołów. W pierwszym z nich zaprojektowano smukłą płytę antresoli dla chóru, wspartą na stalowych podporach. W drugim zastosowano sprężony wieniec obwodowy. W pracy przedstawiono przyjęte założenia, wyniki analizy statyczno-wytrzymałościowej oraz szczegóły przyjętych rozwiązań konstrukcyjnych.

Słowa kluczowe: beton sprężony, cięgno bez przyczepności, kablobeton, kościół

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1. Introduction

The first attempts of prestressing, which is the introduction of prior compression to the concrete, had already been made by the end of the 19th century. However, the first successful attempts had been carried out in 1930s, when steel alloys of low relaxation rates, capable of keeping lots of tension for a longer period of time, were developed. The true evolution of prestressed concrete structures in the world has happened after World War II. In Poland, prestressing has been used since 1950s for large-span roof trusses utilised in buildings, such as industrial halls, warehouses, hangars, etc., liquid tanks and bridges. Lately, post-tensioned concrete has started to be used for building structures. Possibility, the use of advanced unbonded tendons prestressing (first used in 1991) has given a new start for a wide range of applications of prestressing in the construction of high span slabs.

In the world, pre-tensioned precast concrete elements were used to construct churches. Sometimes, external prestressing with FRP material was used to strengthen existing facilities. Usage of post-tensioning with internal, unbonded tendons in the construction of churches is unknown to the authors.

Last year, for the first time in Poland, prestressing was used to construct two church buildings by the authors. Thus, delicate and slender construction forms were achieved. The architectural form the both of the presented churches was designed by the Cracow Architectural Design Studio *f-11* (architect: Ph.D. Arch. Marcin Furtak). In this paper, the structural details and results of theoretical calculation are presented as well as the profits obtained due to prestressing.

2. St. Jack's Church in Cracow

2.1. General profile of the construction

Saint Jack's Church in Cracow was designed as a modern geometrical shape (Fig. 1) on a plan of a rectangle with dimensions of 32.3×15.5 m, with a bell tower rising up to 25.0 m from the ground floor. The building structure consists of reinforced concrete transverse frames and filling masonry walls between them. The RC frame supports were created by

Fig. 1. The view of the St. Jack's church model

a 1.6×0.25 m reinforced wall with 0.35×0.38 m columns (Fig. 2a). The timber roof girders with a cross-section of 200×600 mm are hinge supported on the RC columns of the frames. Additionally, the timber girds are equipped with steel ties. Frame spacing is equal to 3.0 m. The structure of the church is based on a continuous footing. Inside of the building, an entresol was designed for the choir, which is presented in the paper in details. The designers had to face the task of creating a thin, slender slab without beams, with the limit of the points of support, achieving a delicate architectural form of the inside of the sanctuary.

2.2. Post-tensioned slab for the choir entresol

As presented, the prestressed slab for the choir entresol has a shape of a part of a ring with a width of 3.06 m (Fig. 3). It is supported on masonry walls on the external edges. The slab is supported on two steel additional interior steel supports. These are made of a steel tube φ159×16 mm, which, in two thirds of the height, are split into three smaller pipe branches of φ114.3×10 mm, resembling a tree in the final shape.

In order to reduce the interior span deformation of the slab caused by the weight of choir organ, the slab needed to be additionally suspended to the roof structure with steel hangers of φ30mm each. The maximum length of the span in the axis of supports equals 12.2 m. Such large spans suggested that additional edge beams should be made. However, the architectural vision required a thin construction of the slab and transparent glass filling of the area between the slab and the barrier. That was the reason to search for a more effective structural solution. The conducted analysis has shown that in the case of reinforced concrete slab of 250 mm in thickness, the elastic deflection from long-term loads was about 14mm, which allows us to expect long-term deflection at the level of 60–70 mm. The possible thickening of the slab made the structure less attractive and caused an additional
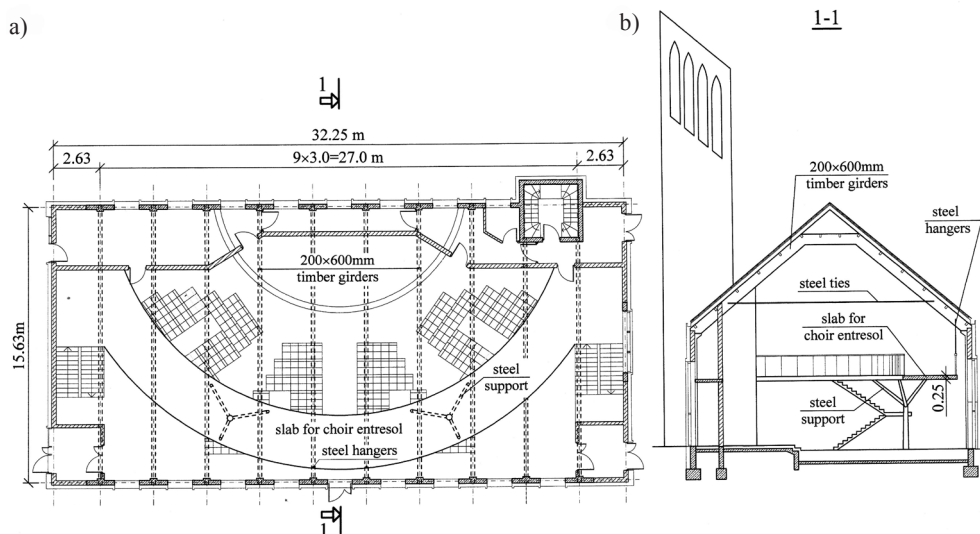

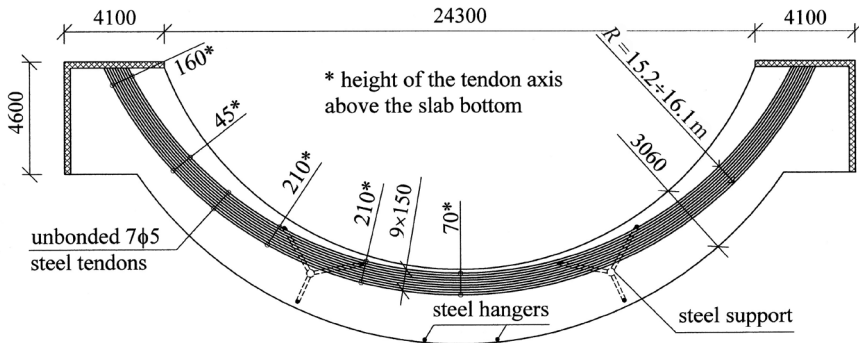Fig. 2. St. Jack's church plan (a), vertical cross-section (b)

Fig. 3. Post-tensioned slab for choir as well as prestress arrangement

Growth of the massiveness of steel supports. The decision was to construct a slab of 250 mm thickness, post-tensioned with unbonded tendons. After a few attempts, a final tendon arrangement was assumed, as presented in Figure 3. Ten cables made of Y1860 steel were used. The cables were planned on the circular curve of the average radius equal to 15.65 m and maximum overhang of 140 mm. Each tendon is tensioned with a force equal to 220 kN.

2.3. Calculation analysis

For design purposes, a slab and steel supports model was created in the FEM system. The slab was built with three and four nodal elements, of an edge length not exceeding 300 mm. For the purpose of the analysis, the concrete class of C30/37 parameters was assumed. All analysis was conducted in the linear-elastic range of concrete behaviour.

Figure 4 shows a FEM model and Figure 5 presents the assumed loads. The analysis includes an average value of the prestress force after time-depended losses, taking it as 80% of the initial value, which is 176 kN. Prestressing was modelled with the use of a substituted load. Horizontal action of the cable at the value of 11.6 kN/m and vertical, resulting from cable curvature on the sections between supports (Fig. 5), was taken into account. In the anchorage zone, the pointed bending moments, equal to 7.0 kNm were added. Beside the
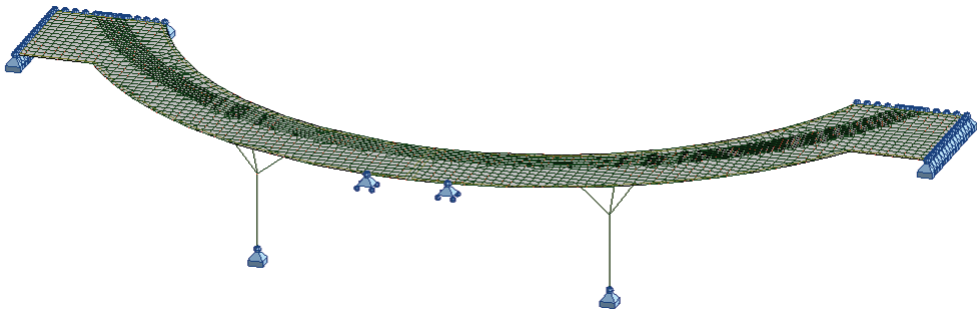

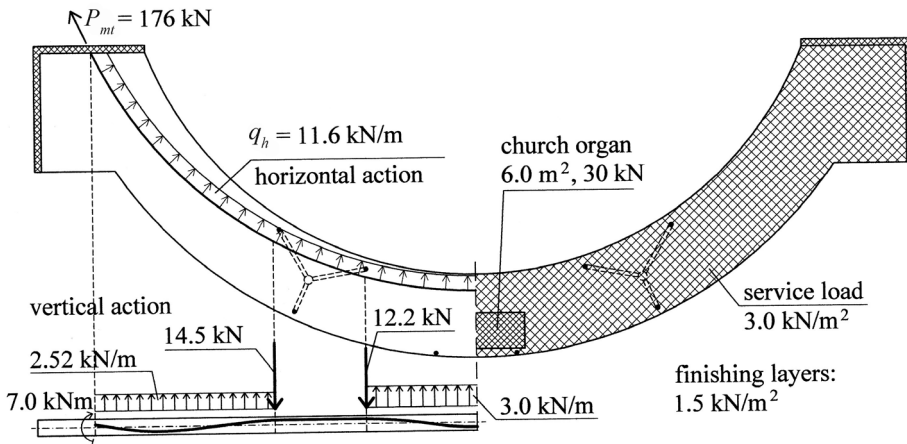Fig. 4. Choir entresol FEM model

Fig. 5. Slab load assumed in calculation

self-weight of the slab, the weight of all the finishing layers of 1.5 kN/m², the weight of the organ of 30 kN and service load of 3.0 kN/m² (Fig. 5), were also added.

Figure 6a shows the values of the bending moment, which takes place due to the self-weight of the slab in the circumferential direction. Figure 6b shows the bending moments caused by prestressing. The moment caused by self-weight (40.8 kNm/m) was completely reduced by the prestressing moment, which equals 49.9 kNm/m. In a design situation, assuming that all possible loads are present and that all prestress losses occur, the maximum bending moment in the span equals 23.9 kNm/m (lower moment). Figure 7 shows the course of all the longitudinal forces in the circumferential direction due to the prestress action. Value of the longitudinal force in a most loaded cross-section differs from 188 kN/m on the external edge to 522 kN/m on the internal edge.
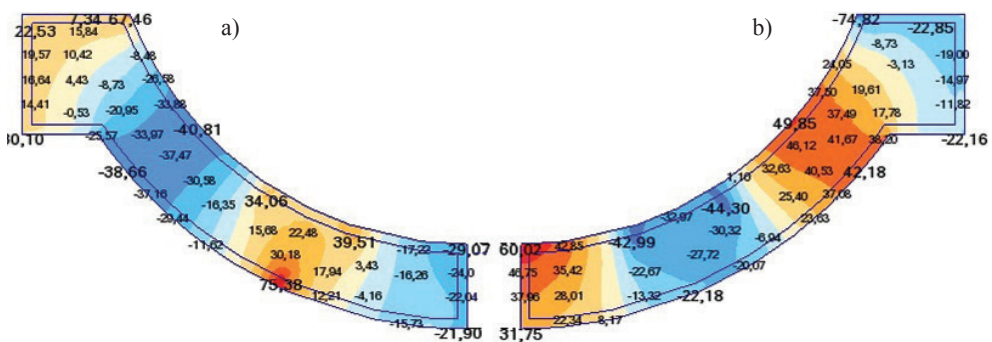

Fig. 6. Circumferential bending moments in kNm/m due to self-weight (a), due to prestressing (b)
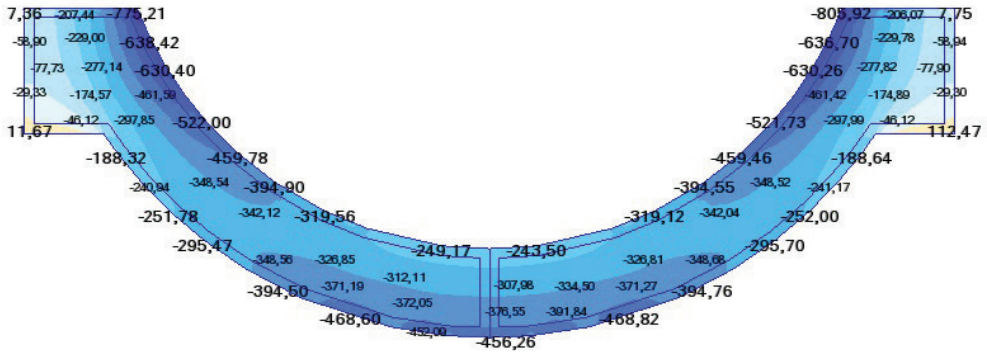
Fig. 7. Normal forces diagram in circumferential direction due to prestressing in kN/m
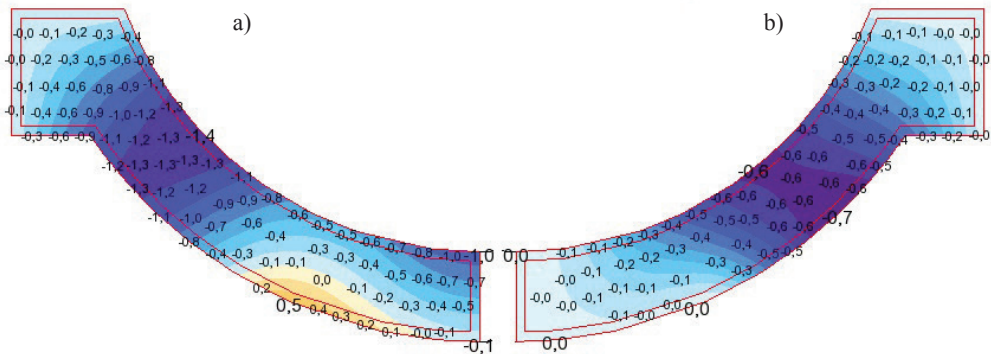

Fig. 8. Elastic deflection in mm due to dead load (a), due to dead load and prestressing (b)

The concrete cross-section stress analysis in the circular direction for the most bended cross-section of the external span ($M = 23.9$ kNm/m, $N = 522$ kN/m, prestress eccentricity: $z_{cp} = 80$ mm) showed that values of compressive stress equal 0.37 MPa for the upper edge and 3.8 MPa for the lower edge under the design situation.

One of the main problems to solve was to reduce the deformation of the thin slab. Values of the elastic deformation for long-term loads without prestressing are shown in figure 8a. In the case of absence of prestressing, the value of deflection equals 14 mm. Based on the technical knowledge, for the cracked reinforced concrete structures, it may be expected to get a four-five times higher values of long-time deformation than the one resulting from elastic analysis. Such approach suggests that the value of a final deflection is about 55÷70 mm. In the case of presence of prestressing, that value equals 7 mm (Fig. 8b). Following the guidelines included in [1], for prestressed slabs, in case of crack prevention, a long-time deflection can be estimated by using appropriate factors increasing the elastic deformation. That factor value equals to 3.0 for dead loads and prestressing and 1.5 for life load. Based on this method of evaluation, the value of the final deformation of prestressed slab is 21 mm for span of the slab equal to 8.3 m. The assessed deformation is a 1/382 of the total span, and this value is acceptable.

3. St. Peter's and Paul's Church in Bodzanow

3.1. General profile of the construction

Another sacred building in which prestressed concrete was used is the Church of St. Peter's and Paul's (Fig. 9) in Bodzanow (nearby Wieliczka). The shape of the church was designed on a plan of a 25.5 m diameter circle (Fig. 10a). The structure of the building is made of 12, steadily positioned on the perimeter, RC columns of the cross-section of 0.4×0.6 m. On the columns, radiating roof girders made of laminated wood are sustained (Fig. 10b).

The preliminary analysis showed that the usage of girders, fully connected in the central node, without any additional perimeter tie rods, would require cross-sections of approximately 2.2 m in height. Another version assumed taking steel peripheral tie rods, running along the chords, which would allow to decrease the height of the cross-section of the girders to 700 mm. In the end, a ring beam made of prestressed concrete of a 300×500 mm cross-section was chosen, hidden in the peripheral infill wall.

In the building, a ring-shaped entresol, made of reinforced concrete, was built on the level of the first floor, sustained on the external edge on 12 columns, and on the internal edge on 6 columns, made of concrete (Fig. 10a). Additionally, on the internal edge, between the pillars, 6 suspensions to the roof girders were used.

Fig. 9. Visualisation of St. Peter's and Paul's church

3.2. Prestressed ring beam

The wooden roof girders form the dome with circumferential tie-beam. A good solution for this tie-beam is to make it as a prestressed element. All over the world, it is common to use a post-tensioned ring beam in liquid tanks, but this beam is often used in circular buildings with radial roof girders, too [2]. In Poland, this solution was used in the renovation the Cetennial Hall in Wrocław in 2009 [3, 4].
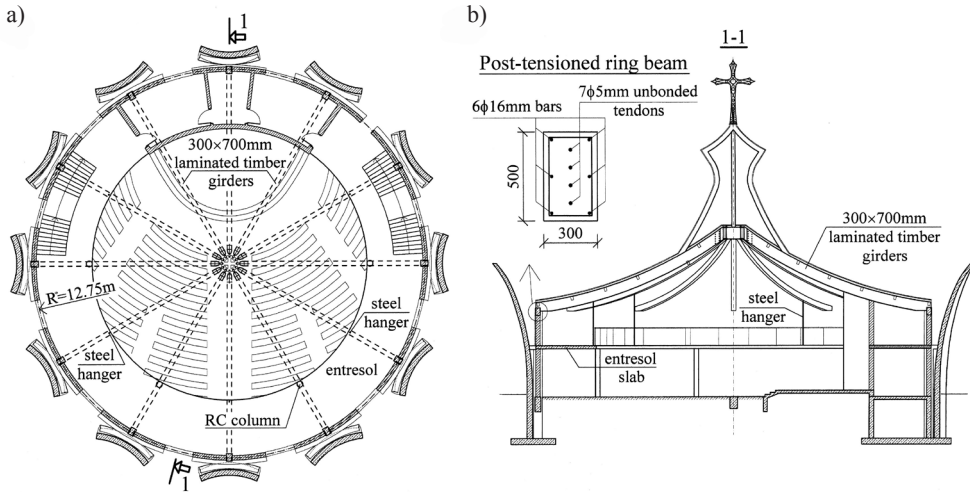
Fig. 10. St. Peter's and Paul's church plan (a), vertical cross-section and post-tensioned ring beam (b)

A prestressed ring beam of 300×500 mm cross-section (Fig. 10b) was designed of C30/37 class concrete. Prestressing with 4 unbonded tendons, positioned in the vertical axis of the cross-section, was applied. Tendons of 7ϕ5 mm made of Y1860 steel were used. The ring beam is reinforced in the longitudinal direction, by 6 bars of ϕ16 mm and by transverse stirrups of ϕ6 mm every 200 mm. It was assumed that the prestressing will be implemented once the reinforced structure will be done and before the installation of the roof girders.

The analysis was carried out on the model created in the FEM system, which is shown in the Figure 11. The model consisted of concrete columns, entresol slab, peripheral ring beam and roof girders. The bar and the shell elements of the geometrical and material characteristics, which matched the ones applied in the project, were used. The following two models were analysed: prestressed model with and without the roof structure. The prestressing force was considered as the circumferential radial load applied to the ring beam and directed to the centre. The force after the total loss was assumed, which has value of 75% of the initial force (15% – immediate loss, 10% – time-dependent loss).

Figure 12 shows the distribution of the longitudinal forces inside the ring beam and bending moments after the prestressing (dotted line), due to roof girders action (dashed line) and final values (continuous line). The axial force of 621.1 kN and bending moment with changeable sign caused by the stiffness of the columns, with the value of –13.6 kNm above the columns and +6.6 kNm between the columns, were entered due to prestressing. As a result of the forces coming from the roof girders, the compressive axial force was reduced to the value of 391.0 kN, the bending moments escalated to the value of –81.9 and +42.1 kNm.

The analysis of the stress inside the ring beam (Fig. 13) showed the occurrence of tensioning in the cross-section above the columns, with a value of 4.0 MPa (continuous line). This value does not guarantee that there are no cracks. The ratio of bending moment to the cracking moment is 81.9/45.0 = 1.82. Such value is small and safe for the increase of deformations resulting from the cracking. The calculated width of the cracks according to [5] is 0.15 mm.

Figure 14 shows the deformation of the ring beam after prestressing (dashed line) and the final deformation (continuous line) resulting from the action of the load coming from the roof gorders. Prestressing leads to the regular movement of the ring beam to the centre, with a value of 2.0mm. The load coming from the roof girders leads to the deflection of the ring beam. When the axial distance of the columns (measured along the chord) is 6.8 m, the value of the deflection is 3.0mm. Even the presented values come from the elastic analysis, the possibility of the cracking the cross-sections above the columns and the small ratio of the bending moment to the cracking moment, which is 1.82, suggest a small influence of the cracking to the increase of the deformation. Even, when the values of the deflection will increase several times in comparison to the presented values, the result will be safe for the structure. It should be noted that the analysis of the stresses, cracking and movement was carried out assuming a full load with snow, according to [6] which is almost a half of the total load coming from the roof. According to this, the obtained values are not permanent, but temporary.
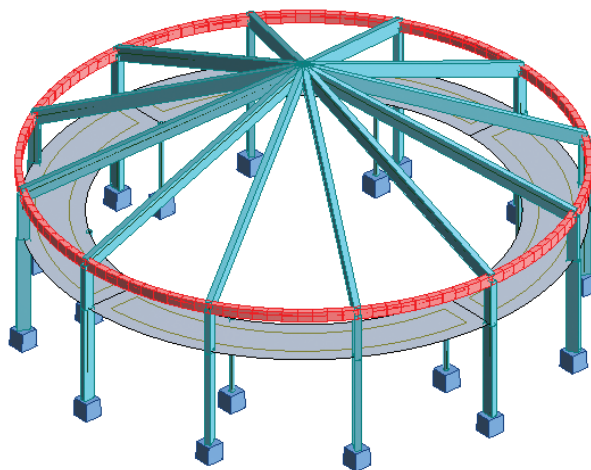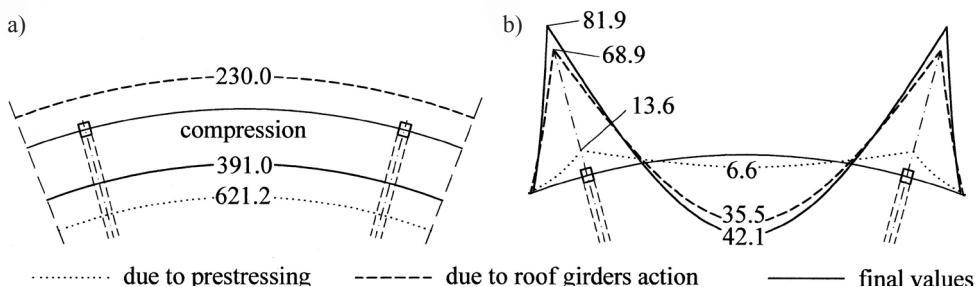

Fig. 11. Church structure FEM model

Fig. 12. Diagrams of longitudinal force in kN/m (a) and bending moment in kNm/m (b) in the beam ring
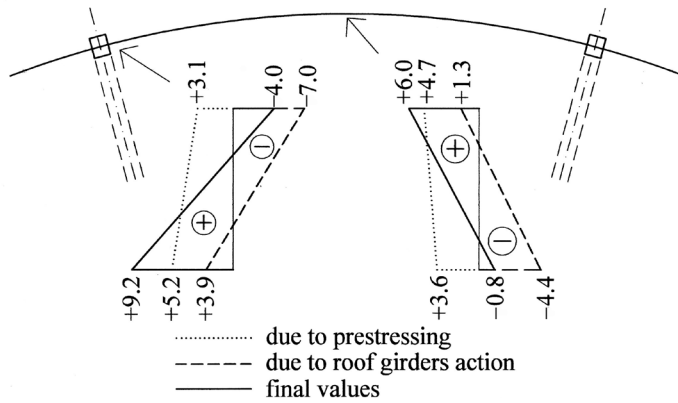
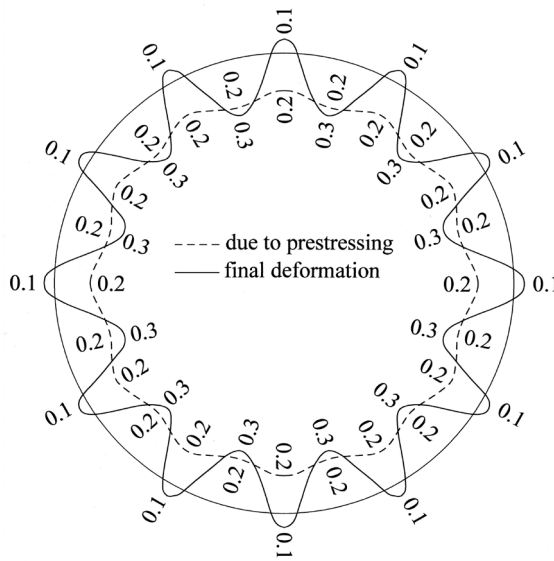Fig. 13. Concrete cross-section stress diagrams (in MPa)

Fig. 14. The ring beam deformation (in mm)

4. Conclusions

The paper shows the first in Poland examples of the usage of elements made of prestressed concrete in church structures. The construction of both of the presented churches is in progress now. In the first case, prestressing with the unbonded tendons allows to avoid additional beams and create a proper, thin entresol slab for the choir. This resulted in an architecturally attractive entresol, supported by steel columns shaped in the image of forked boughs of tress. In the second case, usage of the prestressed concrete circumferential ring

beam allowed to decrease its cross-section (comparing to the previous, reinforced concrete solution), which made it invisible (hidden in the wall). This solution also allowed to resign from the previously designed steel tie rods, running along the chords between the girders or from creating any other form of peripheral concentrations.

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