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STRUCTURAL FAULTS OF HISTORIC ST. GEORGE'S CHURCH IN GLIWICE OSTROPA – ANALYSIS AND STRENGTHENING METHODS

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Abstract

St. George's church in Gliwice-Ostropa underwent, in its history, numerous reconstructions resulting in combination of elements of various historical and artistic value. Original nave of the church, of wooden ring beam structure, was reconstructed in the 70's of the twentieth century by means on external wooden structure, and brick fragments of the structure (presbytery and sacristy) were strengthened with reinforced concrete elements preventing expected impact of mining exploitation. The paper presents results of static-strength analysis of the new wooden structure spatial model with particular consideration for spatial rigidity in different variants of floor stiffening. Moreover, basic structural faults have been specified and required strengthening has been proposed. Additionally, the paper presents damages to improperly strengthened and secured walls and floors of sacristy, including proposal of remedy works.

Streszczenie

Kościół Św. Jerzego w Gliwicach-Ostropie podlegał w swojej historii licznym przebudowom, w efekcie łącząc w sobie elementy o różnej wartości historycznej i artystycznej. Oryginalną nawę główną kościoła, o drewnianej konstrukcji wieńcowej, obudowano w latach 70-tych dwudziestego wieku zewnętrzną konstrukcją drewnianą, a murowane fragmenty obiektu (prezbiterium i zakrystię) wzmocniono elementami żelbetowymi, mającymi przeciwdziałać przewidywanym wpływom eksploatacji górniczej. W artykule przedstawiono wyniki analizy statyczno-wytrzymałościowej przestrzennego modelu nowej konstrukcji drewnianej, ze szczególnym uwzględnieniem sztywności przestrzennej w różnych wariantach usztywnienia stropu. Ponadto, wyszczególniono podstawowe błędy wykonawcze oraz zaproponowano niezbędne wzmocnienia. Uzupełniająco, przedstawiono uszkodzenia niewłaściwie wzmocnionych i zabezpieczonych ścian i stropów zakrystii, wraz z propozycją działań zaradczych.

Keywords: **Wooden structures; Masonry structures; Spatial analysis; Rigidity; Cracks; Strengthening.**

1. INTRODUCTION

Wooden churches on the Polish territory are the unique example of sacral architecture combining traditional wooden building with frequently used very bold structural solutions. In many cases they enrapture not only with woodwork craftsmanship but also with beautiful polychromes constituting one of the most precious collection of wall paintings [1]. These are mainly Roman-Catholic churches but in many other areas other faiths are strongly represented including Augsburg-Protestant, the Orthodox faith, the Uniate church, old-ritual, Moses and Mariavite church. At present there are 1729 wooden churches, 730 bell towers, 315 chapels and 11 morgues in Poland [2]. Province of Silesia, despite its relatively small area, is on the forth place as to the number of monumental wooden sacral structures.

Wooden churches, in their history, were destroyed many times, mainly during the wartimes – the example here is the history of St. George's Church located in Gliwice Ostropa described under chapter 2. But even in peace times of the history they often suffered from fire or natural disasters. For instance, in the last decade of the twentieth century there were over 50 churches in Poland that burnt including such precious ones as the Orthodox church in Grabarka as well as churches in Tarnów, Witków, Tylice and Rożnowice [2].

Originally a wooden church was made by means of connecting two one-room structures (central nave and presbytery) in shape similar to a quarter. Once a bell tower was added three levels system was created (front tower, nave and presbytery) oriented east-west. Such a system became popular in Silesia since the middle of the XVII century. Typical solution here was a tower of columnar-skeleton structure (shuttered with boards or single) whereas nave and presbytery were erected as rim structure. Quite often presbytery, and sometimes also sacristy adjacent to it, were erected as masonry structures made of stone or brick – we are dealing with such a solution in case of the subject church in Ostropa.

2. HISTORY OF THE CHURCH

First church in Ostropa was build in around 1340 and it was burnt during hussite war in the first half of the 15th century [3]. St. George's church built in the same place in 1640 was partially destroyed during Swedish wars (1655-1656) and then reconstructed in 1667-1668. The church was consecrated in 1719 by bishop suffragan Eliasz Sommerfield.

In the following years the church played branch role and from 1807 it was a parish. After a new brick church was build in Ostropa in 1926, St. George's church was no longer utilized, sinking into oblivion with time.

Present shape of the church is the effect of numerous changes and modernization out of which the most essential was reconstruction after Swedish wars and modernization in the second half of the twentieth century.

Principle division of the structure lets separate four basic parts (Fig. 1):

- wooden tower on the western side,
- main nave, whole wooden,
- brick presbytery on the eastern side,
- brick sacristy, adjacent to northern wall of the presbytery.

Modernisation mentioned above, performed in the 70's of the twentieth century, involved the main nave as well as presbytery and sacristy. Unfortunately, many design and construction errors were made during this modernization resulting in further deterioration of some structural elements condition, and locally in their failure state.

In needs to be added that main nave interior present huge historical and artistic value as the substantial part of its surface is covered with polychrome dated to years 1667-1668.

Figure 1.
General view of the church from the north-east side

3. WOODEN STRUCTURE OF THE MAIN NAVE

3.1. General description

In the effect of reconstruction after Swedish Wars, the main nave of St. George's church in Gliwice Ostropa was constructed as a ring beam structure, covered with relatively flat roof of post-purline structure, with one vertical post and two skew ones. It survived as such by the 70's of the twentieth century, when during modernization whole roof was dismantled and the open "box" created by original walls was encased with wooden frame.

According to the design assumption the structure was to carry all external loads including horizontal forces caused by wind. However, observations showed big local displacements of original northern wall parts which could be a result of horizontal forces acting on it.

Because of employed solutions, and in particular lack of appropriate stiffening of individual transverse systems of the new structure, it was necessary to analyse it as a spatial system with possibly faithful modelling of rotation freedom in particular joints. During detailed visual inspection, required in order to review original reconstruction design, the whole range of departures and errors made during erection of the new structure were noted, threatening with damage of the roof beams joints and in effect with failure of the whole superstructure.

The subject nave is adjacent to the wooden tower from the western side and to the brick presbytery from the eastern side however, it is entirely separated from them by means of expansion joint. Therefore it can be considered an individual structure.

3.2. Structure

During modernisation mentioned above and carried out in the 70's of the twentieth century, completely new wooden structure was designed and constructed comprising of:

- northern stud sidewall, made of posts and spandrel beams, supported, by means of wooden ground beam, on reinforced concrete continuous footing and topped with wooden grit braced with struts in the extreme fields; the wall is covered with shingle on battens,
- southern sidewall of the structure as above,
- western transverse stud wall (at the contact with the tower), made of posts and spandrel beams, highly cross braced in some fields,
- rafter framing, whose main load bearing elements are four queen-post roof truss beams with additional cross bracing; rafters and ceiling beams have been distributed between girders in spacing of around 1 m; rafters have been based on walls top beams and suspended from the longitudinal floor beam based on girders joints; the roof slope has been shingled and ceiling surface has been covered with boards.

Structure supporting signature, covered with onion-shaped helmet, has been based on longitudinal floor beam, in the girders joints, on the eastern side.

Schematic cross-section of the structure has been presented in Fig. 2 and Fig. 3 shows beam scheme of the spatial superstructure frame.

3.3. Results

Computational analysis of monumental wooden structures is usually related to two aspects of its modelling – general, usually as a bar structure with idealised nodes [4], [5], as well as detailed, where nodes geometry is considered [6], [7], [8], [9]. In the subject case only skeleton as bar structure was analysed at the assumption of actual characteristics of geometrical cross-sections and nodes being perfectly articulated.

Calculations of spatial superstructure frame have been made with the use of Robot program. Even superficial analysis of the structure indicates that calculating it in the flat system is impossible, as the transverse systems are four-hinged frames, therefore mechanisms. The problem was omitted in the archive technical design by separate calculation of roof girders and posts. In practice geometrical stability of the system is ensured by western wall heavily cross braced in particular fields as well as angle braces in the extreme fields of the sidewalls (Fig. 3).

Nave was calculated, using in the joints section, offsets making allowance for actual eccentrics in connections. Material parameters were employed as for timber class C22.

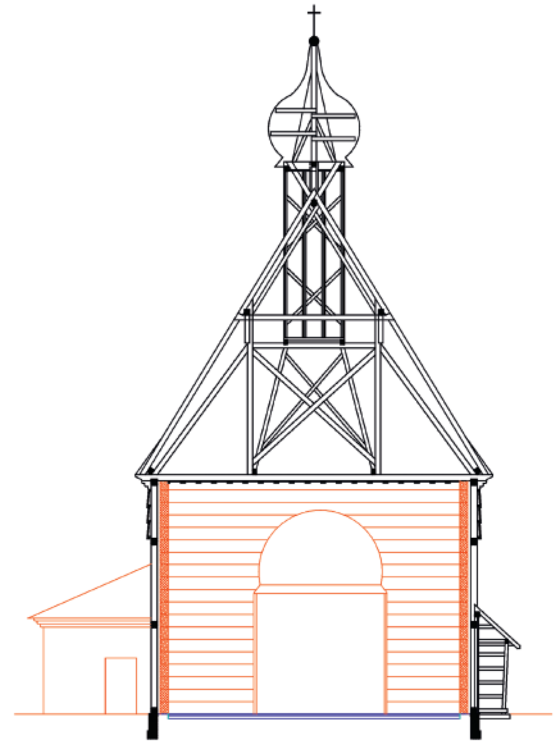

Figure 2.
Cross-section of the nave structure

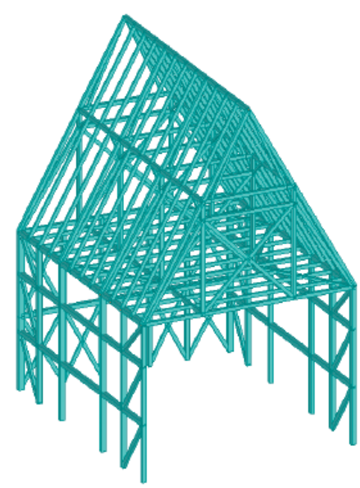

Figure 3.
Spatial model of the nave new structure

Different options of system bracing on the floor level over the nave were employed in calculations. These were as follows:

- originally designed system with braces in the floor plane (not constructed),
- system with membrane in the floor plane, ensuring

stability of the mutual joints system (solution partially doable by fixing the formwork, which is not ideal membrane as it works only in one direction),
- system with braces and membrane,
- system with braces and anchorage of the intermediate joints of the bottom flange of the extreme girder on the eastern side in presbytery brick wall, over the rood arch; during reconstruction described above the wall was strengthened by means of reinforced concrete ring beam.

In all cases structural elements effort was within acceptable limits, whereas there were substantial differences in the joints displacements. Obviously, the greatest displacements were noted in joints of transverse system most distant from bracing wall, adjacent to presbytery.

From computational point of view, system of angle braces employed in the original design, in which relocations of joints in the top beam of the extreme load bearing system reached the value of 0.136 m (Fig. 4), turned out to be the worst. It is unacceptable solution as it leads to basing new structure posts on original sidewalls, and at the same time to loading walls with horizontal forces. Distance between posts and original wall does not exceed 0.10 m and the contractor left installation wedges causing transfer of forces from the posts as concentrated, acting on individual beams of ring beam wall.

Use of ideal membrane on the floor plane level over the nave results in limited relocations of joints up to the value under 0.03 m (Fig. 5), safe for the original structure. Simultaneous use of membrane and braces practically does not change this value.

The best results were obtained in case of anchorage of intermediate joints of the eastern girder in the ring beam of presbytery western wall – relocations of top beams joints do not exceed then the value of 0.004 m (Fig. 6).

Currently incorporated ceiling made of boards meets the membrane assumptions, but only unidirectional, which may result in quite big relocations of the new structure in the presbytery neighbourhood.

However in practice, part of the horizontal forces is transferred to original ring beam wall which cooperates with eastern wall of the nave at the contact with presbytery. The above situation causes spot loading of individual beams in sidewalls, which may significantly in-crease their deformations.

3.4. Structural errors

Detailed visual inspection of the structure showed generally good technical condition of elements and carrying skeleton as the whole as well as lack of

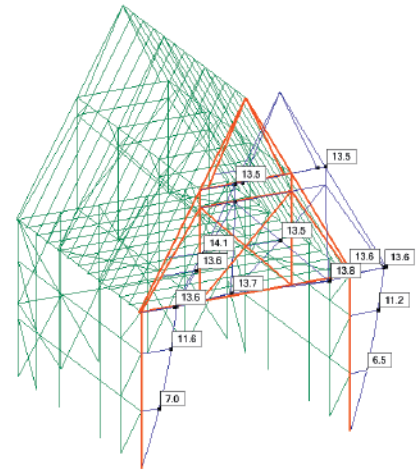

Figure 4.
Displacements extreme transverse system in the original structure

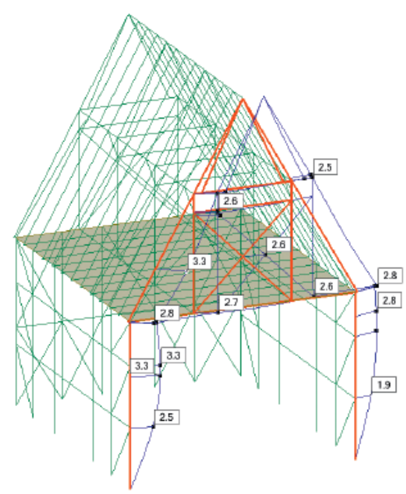

Figure 5.
Displacements extreme transverse system in the original structure

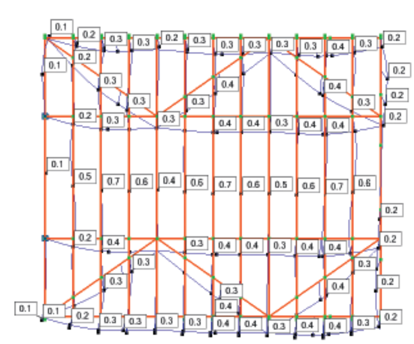

Figure 6.
Displacements of joints on the floor plane, with anchorage of extreme joints in the ring beam of presbytery wall

essential structural or biological wood defect.

However, at the same time the whole range of departures from the original design solutions were noted, resulting many times in real threat of structural elements failure, particularly within joints area. The most substantial faults have been presented below and some of them are shown in the subsequent figures. These are:

- lack of braces designed in the floor plane,
- improper connection of king posts with stretcher – steel connection clip was replaced with clamp of relatively small load capacity causing wood cracking (Fig. 7),
- improper suspension of the intermediate floor beams – bolts were replaced with clamps as above (Fig. 8),
- lack of local bracing of joints.

Ground beam was poorly anchored to the foundation (design error), and linking between posts and ground beam does not ensure transition of tensioning forces.

Substantial utilization fault of the roof is lack of roofing tightness resulting in snow or water getting into the attic space and in effect temporary, intensive dampness of the structural elements.

3.5. Required repairs and strengthening

Analysis of carried out calculations as well as tests and visual inspection of the structure showed the requirement for performing the following works ensuring appropriate rigidity and load bearing capacity of the structure:

- additional anchorage of the ground beams to foundations s well as posts to ground beams,
- proper joining of stretchers with king posts (roof beams) and intermediate floor beams with longitudinal beams,
- strengthening of all joints showing features of rigidity or load bearing capacity loss,
- tightening of the floor plate (membrane) or anchorage of extreme girder joints to reinforced concrete ring beam over the presbytery western wall.

Moreover, replacement of shingle roofing is required – it will let wood conservation and protection as well as proper tightening of the attic space.

In practice, it turned out that all roof structural elements mentioned above (except for stiffening of floor shield) were solved properly in the original structure design of dated 1965. Hence, detailed recommendations for construction include drawings directly taken from that design (scanned fragments have been presented in Fig. 9). Floor stiffening was designed in the form of horizontal shield made of OSB plates fixed to the ceiling beams as well as additional transverse beams. Moreover, it was assumed that nodes in walls frame structure would be strengthened with the use of overlays made of flat bars fixed with screws.

Figure 7.
Improper connection of the truss roof king post with stretcher – with the use of the clamp

Figure 8.
Improper suspension of the intermediate floor beam – with the use of the clamp

Additionally, cleaning of all wooden structural elements was recommended and preservation with the use of preparation Ocean 441 as well as replacement of the existing shingle with the new one made of spruce, double laid. Above referred works were completed in the second half of 2008.

4. MASONRY STRUCTURE OF PRESBYTERY AND SACRISTY

4.1. Description of the structure

Sacristy was made of full brick with the use of lime mortar. Load bearing system consists of three exter-

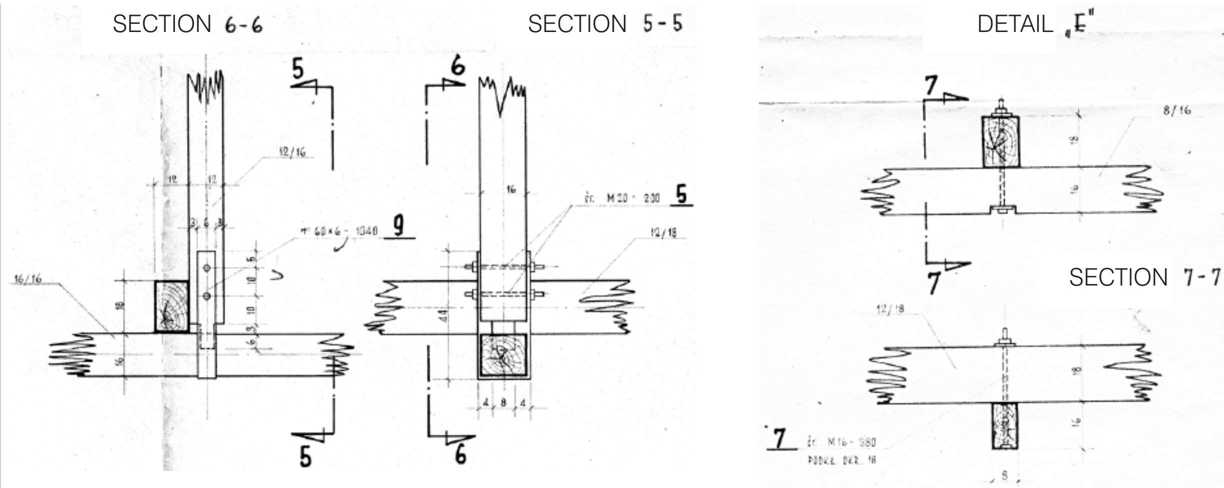

Figure 9.
Nodes design solutions (original design documentation) – compare Fig. 7 and Fig. 8.

nal 0.87 m (including plaster) thick walls and presbytery northern wall (1.04 m thick including plaster). Brick cross-barrel vault was based on the walls. Door way and window openings were made in sacristy walls, including door connecting it with presbytery.

Structure of sacristy foundations is not known, whereas reinforced concrete band of the cross-section 0.25×0.65 m, reinforced with six bars Ø14 mm, with stirrups Ø6 mm spaced every 0.30 m was constructed around them during modernisation carried out in the 70's of the twentieth century.

Northern wall of sacristy was supported by brick slope not connected with the wall.

Double pitched wooden roof covered with shingle was constructed over the sacristy. Present structure of the roof is the result of the above mentioned modernisation. Brick presbytery with annexed sacristy has got 1.04 m thick brick walls with pilasters supporting cross vault with telescopes over the window openings. Continuous footings are made of stone with the use of lime mortar. They were strengthened with identical reinforced concrete band as it was described above in case of sacristy. Outside the walls were supported with three brick slopes of different size, not connected to the original structure. Presbytery walls were topped during modernisation in the 70's of the twentieth century with reinforced concrete ring beam of section 0.20×0.25 m, reinforced with four bars Ø14 mm, with stirrups Ø6 mm spaced every 0.30 m. The roof over the presbytery was constructed during above mentioned modernisation as a timber structure covered with shingle.

Both masonry structures are linked.

4.2. Structural defects

Even cursory inspection of available elements of sacristy structure reveals its before-damage state actually threatening with collapse of part of walls and vaults. Numerous cracks appear within the whole masonry structure covering practically all walls and substantial part of the vaults. The most significant ones have been presented in the following photos (Fig. 10-13). Moreover Fig. 10 shows practically whole damage to the slope supporting northern wall of sacristy.

Based on available sources it can be assumed that the most significant cracks within sacristy walls and vaults appeared in the 70's and 80's of the twentieth century as they were not mentioned in the earlier sources. Therefore, observed structural defects appeared right after construction of the above motioned reinforced concrete bands and ring beam over the presbytery. Practically all observed cracks in the sacristy walls and vaults were caused by subgrade deformations resulting from mining exploitation. The nature of cracks shows it; intensive ones in vaults and upper parts of the walls where protection was not provided, and disappearing ones in bottom part of walls secured by reinforced concrete band. Similar type of cracks did not appear in presbytery where strong reinforced concrete ring beam was constructed at the level of rafter framing base.

Observations of numerous control seals, installed in sacristy in 2005 indicate that no further cracks opening development took place in the last three years. Currently no mining exploitation is performed under Ostropa.

Damages within brick slopes were caused by intensive dampness and frost penetration.

4.3. Required repairs and strengthening

There are many theoretical studies as well as practical solutions to the problem of strengthening brick vaults, due to their popularity. Some of the basic methods of strengthening include: incorporation of steel stays making the system of load bearing walls more rigid, adding to the vaults by means of reinforced concrete coatings, strengthening with the use of glued metal plates and steel bars or mates and CFRP tapes, incorporation of steel or wooden structures enabling vaults suspension and finally mineral and resin injections [10], [11], [12], [13].

Figure 10.
View of the sacristy northern wall

Figure 11.
View of the sacristy western wall

Figure 12.
Cracks in the sacristy north-east corner

Figure 13.
Window head in the sacristy western wall

In order to ensure adequate load bearing capacity and rigidity of the sacristy structure, the following treatments were selected from the above as the proper ones:

- cleaning and injection of all cracks with the use of adequately modified fine-grained cement mortars or resin; in case of the biggest cracks local reinforcement of walls is recommended,
- construction of reinforced concrete ring beam or steel stretchers hidden in the top part of walls, capable of carrying tensile forces in case of further subgrade deformations; such activity is not required if obtained data proves lack of such danger.

Recommended works require dismantling and later reconstruction of the roofing over the sacristy. It should be used for control and possible repair of wooden rafter framing. After completion of the above recommended repair works, temporary observation of the structure is recommended including installation of control seals in case of next cracks appearance.

Damaged parts of the slope require dismantling and reconstruction with simultaneous construction of effective horizontal insulation.

5. CONCLUSIONS

The paper presents an example of structure combining historic and entirely contemporary elements. During modernisation the church underwent in the 70's of the twentieth century, the whole range of unacceptable design simplifications was employed and many working errors were made. The effect is that the existing structure is not rigid enough which results in excessive deformations of the wooden skeleton as well as serious damages to the masonry structure. Based on computational analysis and detailed visual inspection results, scope and general method of works construction ensuring adequate rigidity and load capacity of the structure were determined. In practice a detailed program of required works has been prepared. They were completed in the second half of 2008.

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