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Structural strengthening of the Dobrzyca Palace, Poland

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ABSTRACT: In the presented paper authors show the idea of strengthening the construction of historical Palace in Dobrzyca (Poland). The complex consists of the English style park, the Palace, the Pantheon and the Monopter. There were necessary restoration works and, in the first order, protection and constructional stabilization, because of the bad technical state of the Palace construction. The Authors elaborated complex programme and schedule of strengthening the construction of the Palace which guarantee its stabilization through equilibration of internal tensions in structural elements. As a consequence of it some difficulties occurred in arrangement the appropriate rigid constructional system. Steel braces were applied in those areas. Additional bracing of building structure, in the level of ceiling above the ground floor, was obtained by initially compressed band jointed with steel braces and reinforced concrete ring beam in the level of the wall cap. As a result of carried out strengthening spatial stiffness of building considerably raised, protecting it from repeated constructional damage.

1 CONSTRUCTION AND TECHNICAL STATE OF THE PALACE BUILDING

Construction of the Palace building in Dobrzyca is traditional. The walls were executed as multi-layer ones. External surfaces are made of ceramic bricks, meanwhile the interior was made mainly of stones, also rubble, cobbles, etc. There are brick barrel vaults, surbased vaults with lunettes and single and double-span segmental vaults above cellars. The part of vaults is strengthened by brick upper ribs. The backfilling over the vault is made of sand and rubble. The construction of over-ground floors has classical closed beam wooden structure with blind floor and backfilling which consist of sand mixed with clay. The ends of beams are based on the walls. Parts of corridors and halls have brick vaults. There are mirror vaults above ball room and main staircase. For smaller spans above staircase were executed wooden vaults. In the beginning of the XX century there was applied steel construction (I 300, I320) to support wooden vaults in a ball room. The roof of the Palace have wooden and collar beam construction. It is gable with lucarnes and is symmetrical in relation to line of development. Present roofing felt onto board lagging fixed to rafter will be exchanged to copper sheet.

Technical state of the Palace building construction is diversified (Figure 1). Spatial stiffness of construction is reduced, that was caused by inappropriate building works on different stages of building's development and lost of primary strength features of applied materials. As a result of these, many cracks and rifts in walls and lintels can be observed as well as construction separation, plaster detachment, considerable biotic damages of wooden elements and general constructional degradation of building. The walls near landing of main staircase as well as walls of the first floor and wall coping belong to the most destructed zones.

Figure 1. General view of the Palace in Dobrzyca before executing strengthening and renovation works.

Temporary repair works were executed in the Palace in earlier years, but there was not realized complex solution of strengthening and building's stabilization. The Authors of this paper elaborated complex programme and schedule of strengthening coherent to conservatory and constructional regards which satisfied general solution of building stabilization. This programme have got positive opinion and it received admission to realization. It was assumed, because of antique interior decoration of the Palace, that in

realization strengthening will be realized only in inter-floors areas and fragments of external part of building. Those works include stabilization of bearing construction by preservative way. It was decided to disassembly historical but very ruinous floors, what made possible constructional interference and realization of strengthening in inter-floors spaces. It was tried not to disturb existing statical scheme but only stiffen building by equilibration of internal stresses. Changing of statical schemes were limited to necessary minimum, trying to keep maximum historical substance of building.

2 STRENGTHENING WORKS IN THE PALACE BUILDING

It is possible to distinguish three levels of strengthening related to levels of floors above cellar, ground floor and the first floor (with area of rafter framing). Relatively good technical state of cellar walls, despite thrust of vaults, caused that application of radical actions were not necessary. Strengthening of floor above cellar can be divided into two stages. The first stage included strengthening of building with using existing flanged beams I220 which were installed in time of earlier executed repairs. Those beams make up supports for concrete plates of floor, which should be executed with cavity above vaults in aim of reduction of their loadings. On the basis of constructional analysis of vaults and theirs technical state, it was assumed that their carrying capacity is sufficient to transfer loads from backfilling of the first storey floor, dead load and planned live load. The beams were used as strengthening tie rods and only steel anchors were added to its ends. The steel anchors were fastened in bearing internal and external walls. In the second stage strengthening anchors as a classical bracing anchors on a level under floors of ceiling above cellar were executed. In aim of relief of cellar floor ceramzit was applied stabilised by lime as backfilling of floor.

Internal steel tie rods and initially tense reinforced concrete band which are placed around the building are the main constructional strengthening system of the building. The steel bracing in inter-floor space are made of rods which had 24 mm of diameter. Its anchoring was twofold: classical in walls by means of holdfast elements or in external reinforced concrete band. The steel bracings were designed also to transfer forces from anchoring of reinforced concrete bands near portico walls. In consideration of thickness of walls, by which of short tie rods could not passage in rooms of vestibule were sticked its ends into wall. For control of its tensioning adjusting bolts were used. Similar system was applied with success by Authors for walls strengthening of monastery in Lubiąż. There were executed investigations of bearing capacity of sticked braces, that showed interesting results which confirmed effectiveness of the method for strengthening of historical masonry constructions.

In order to liquidate cracks and fractures in damaged walls and lintels of the building a method of initial prestressing was applied so as to reach a state of artificial compressing which caused elimination of possibility of tensile stresses rising. The best constructional action is application of initially tensed reinforced concrete band. The level of ceiling above ground-floor was the most proper place for such reinforced concrete band, where the largest damages of walls, fractures, cracks of walls and lintels were observed. Conservatory agreement was obtained for realization of such bands. The best place for location of reinforced concrete band was horizontal rustic with stucco-worker's elements (decorative garland), which was disassembled (it will be reproduced during repair of façade). Reinforced concrete bands were executed on external contour of the walls on a level of ceiling above the ground floor. That band consists of four round rods which have 24 mm of diameter which were initially tensioned using small forces. Special corner anchors for tensioning of the band was designed. In consideration of length of bands, they were joined by stabilising indirect elements from U-sections which improved tension possibilities of those system as well as bounded with band system of internal bracing. The bands co-operates with bearing walls by means of adhesiveness of concrete, masonry and steel rods. Initial tense of steel rods was obtained by tensioning the rods to force which equal compressive strength of masonry (usually stress equals about 0,1–1,0 MPa) and every time are determined depending on technical state of masonry and its structure. Initial tensioning of band created in wall such state of internal stresses, which will effectively counteract working of external forces and it enlarges bearing capacity as well as general stiffness of construction. Arrangement of construction strengthening on a level of ceiling above ground floor is presented in Figure 2. Selected constructional details of initially prestressed band are introduced in Figure 3. The main node of band, presented in Figure 3, is useful to anchoring of main steel tie rods of bracing system whereas indirect node is local form of strengthening of floor wooden beams by its anchoring in band. Local strengthening of beams in connection with global system of tie rods strengthening and reinforced concrete band, assured building sufficient usable stiffness on a level of ceilings above ground floor. In Figures 3, 4 detail of executed band during of concreting is presented. In frames of ceilings strengthening above ground floor routine strengthening works and protection of wooden floor were executed:

- the sandy backfilling was exchanged for rock wool, relieving it and enlarging bearing possibility,
- improvement of beams fixing and exchange of biotic damaged ends of beams were made,

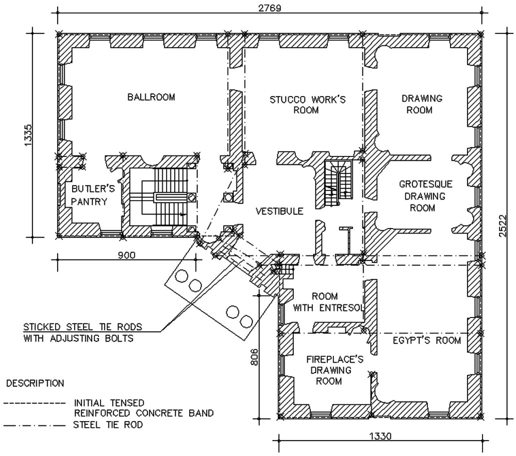

Figure 2. Arrangement of construction strengthening on a level of ceiling above ground floor.

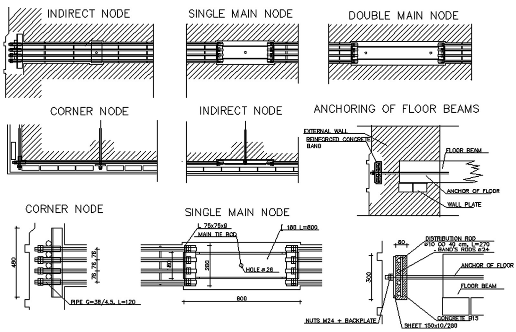

Figure 3. Selected constructional details of initially prestressed band and nodes of band.

Figure 4. General view of initially prestressed band.

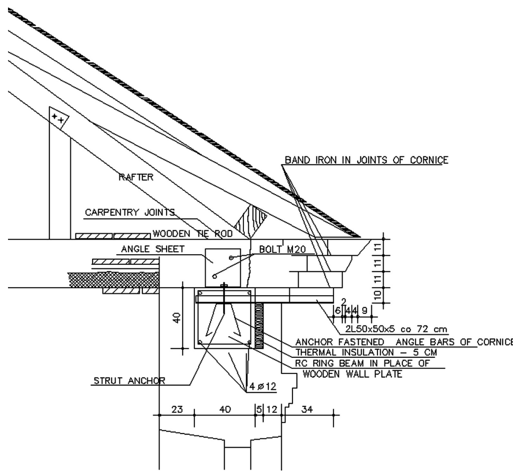

Figure 5. Constructional detail of reproduced brick cornice of roof.

- wooden elements and walls were impregnated by using fungicidal and insectidal means.

On a level of ceiling over the first floor it appeared that the best constructional and economical solution was to realize strengthening of building's bearing system by means of reinforced concrete ring beams. At selection of ring beams section, it was taken under consideration that their task is not only to stiffen the building, but also to transfer of tensile forces which are caused by non-uniform settlement or loading and strut forces from roof. As a result of application of reinforced concrete ring beams in wall coping more uniform stress distribution in strengthening masonry was obtained. Realization of ring beams was carried out in traditional way and based on the insertion suitably strengthened elements in longitudinal and transverse bearing walls (in the place of biotic damaged wall plates) which fasten circularity arrangement of walls in closed and rigid spatial contour. That ring beam co-operates with walls by means of friction and adhesiveness what effectively increase stability and spatial stiffness of construction on lower roof's level. Additional advantage of executed reinforced concrete ring beam was using of it to reproduce damaged roof cornices. In ring beam angle bars supporting reproduced brick cornice of roof were anchored (suitable constructional detail is presented in Figure 5), in the place of damaged and ruined wooden elements for cornices. Additionally, during strengthening of roof construction built-in reinforced concrete ring beam for anchoring floor's beams by means of angle sheets and strut anchors was used. Such as in case of ceiling strengthening above ground floor many routine protecting and impregnating works were executed.

3 NUMERICAL MODEL

3.1 *Distribution of tensions in connection on anchorage length of brace in wall*

Similar model was applied with success by Authors for strengthening of monastery in Lubiąż.

The following assumptions were applied:

- numerical model is analysed in linear range,
- thickness of adhesive joint and its surface are neglected small,
- shear stresses occurred on thickness of adhesive layer are constant,
- bending of joined elements does not occur,
- model of connection is axial-symmetrical.

The following function of shear stresses τ_k in joint was assumed:

$$\tau_k(x) = g(x) \cdot \sigma_s(x) \tag{2}$$

where: $g(x)$ = parameter of accumulation of tensions as a linear function of ordinate x, $\sigma_s(x)$ = normal stresses in steel bracing.

On the basis of relations determined in [1] and analyses of results of experimental investigations, parameter $g(x)$ was described by equation:

$$g(x) = \frac{x}{l_z} \cdot \frac{d_p}{l_z} \cdot \frac{E_s}{E_m} \cdot \frac{A_s}{A_m} \cdot \alpha_t \tag{3}$$

where: α_t = temporary coefficient of shear tensions, intensity understood as a relation of average destructive tensions to average tensions for given force F:

$$\alpha_t = \frac{R_t \cdot l_z \cdot \pi \cdot d_p}{F} \tag{4}$$

After regarding boundary conditions:

$$\sigma_{s|x=0} = \frac{F}{A_s} \tag{5}$$

the following relations defining distribution of shear stresses $\tau_k(x)$ in adhesive joint and normal stresses $\sigma_s(x)$ in steel brace was obtained:

$$\tau_k(x) = x\,\frac{d_p^2 E_s R_t \pi}{l_z E_m A_m} \cdot e^{-2x^2 \frac{d_p E_s A_s R_t \pi}{l_z E_m A_m F}} \tag{6}$$

$$\sigma_s(x) = \frac{F}{A_s} \cdot e^{-2x^2 \frac{d_p E_s A_s R_t \pi}{l_z E_m A_m F}} \tag{7}$$

In Equations 3 to 7 assumed the following notation:
A_m = contributing surface of wall section ($A_m \cong 8268 \times 10^{-6}\ \mathrm{m}^2$),
A_s = surface of brace section,
E_m = Young's modulus for masonry,
E_s = Young's modulus for steel,
F = over-turn force in brace,
R_t = average shear strength of weakest link of connection,
d_p = diameter of brace,
l_z = anchorage length of brace in wall.

3.2 *Determination of carrying capacity of connection and possible damage model*

Obtained formulas which concern distribution of normal stresses in steel brace (Eq. 7) and shear stresses τ_k in joint (Eq. 6) allow to estimate qualification of two damage models of sample (for assumed material characteristics and anchorage lengths) in relation to value of over-turn force. Possible to reach damage models (state of ultimate bearing capacity) are as follows:

A. Carrying capacity of brace:

$$R_{max} \le max\ \sigma_s(x) \tag{8}$$

$$max\ \sigma_s(x) = \sigma_{s|x=0} = \frac{F}{A_s} \tag{9}$$

so, calculated from Equation 9, maximum force carrying through joint can be written as:

$$F_{max1} = A_s R_{max} \tag{10}$$

where: R_{max} = limit of strength of applied steel.

B. Carrying capacity of adhesive joint (or shear strength of masonry):

$$R_t \le max\ \tau_k(x) \tag{11}$$

$$max\ \tau_k(x)\ \ \frac{d_{\tau k}(x)}{dx} = 0 \rightarrow x = \frac{1}{2}\sqrt{\frac{l_z}{d_p} \cdot \frac{E_m}{E_s} \cdot \frac{A_m}{A_s} \cdot \frac{F}{R_t \pi}} \tag{12}$$

$$max\ \tau_k(x) = \frac{1}{2}\sqrt{\frac{d_p^3}{l_z} \cdot \frac{E_s}{E_m} \cdot \frac{1}{A_s A_m} \cdot \frac{1}{e} \cdot F \pi R_t} \tag{13}$$

so calculated from Equation 6 maximum force carrying through joint can be written as:

$$F_{max2} = \frac{4e}{\pi} \cdot \frac{l_z}{d_p^3} \cdot \frac{E_m}{E_s} \cdot A_s A_m \cdot \frac{\tau_{\max}^2}{R_t} \tag{14}$$

where: R_t = average shear strength of weakest link of connection, τ_{max} = maximum shear stress of adhesive joint (joint or wall).

The real maximum force carrying through connection is smaller force calculated according to Equations 10 and 14:

$$F_{max} = min\ \{F_{max1}, F_{max2}\} \tag{15}$$

3.3 *Optimum anchorage length*

Optimum anchorage length can be calculated after the formula of carrying capacity of adhesive joint (Eq. 14), substituting for maximum force F_{max2} value of force F, which can carrying that connection (in practical applications with suitable coefficients of safety).

$$l_z = F \cdot \frac{\pi}{4e} \cdot \frac{E_s}{E_m} \cdot \frac{d_p^3}{A_s A_m} \cdot \frac{R_t}{t_{\max}^2} \tag{16}$$

under assumption:

$$F \le F_{max1} = A_s\, R_{max} \tag{17}$$

Evidently it must be fulfilled condition $F \le F_{max1}$, where F_{max2} is maximum force calculated for given type of brace (Eq. 10).

When we are using maximum of carrying capacity of steel as a one from elements of connection – substituting Equation 17 to 16 – we will receive formula onto optimum anchorage length.

4 RENOVATION WORKS OF WALL'S PLASTERS

The problem of plasters repair in the Palace building was also very interesting. The plasters of historical buildings are often reproduced without suitable investigations of plaster base. It often leads to execute new plasters with wrongly chosen or accidental materials what causes in consequence fast renewed damages, efflorescences and superficial discolouring. In aim of determination the method of plasters reconstruction realization of its investigations in consideration of possible appearance of salt, chlorides, sulphates, nitrates and PH units is necessary. Contents of individual components of plasters impose system of its reproducing, protecting from renewed damages. In aim of the structure of existing façade a samples from it were taken (three from external elevation and four from walls and cellars vaults). On basis of them investigations onto proportional content of salt, water and PH unit were executed. Results of investigations are presented in tables below.

Table 1. Description of plaster samples examination.

Sample	1	2	3	4	5	6	7
Place of taken	external façade.			vault of cellars		wall of cellars	
Height	4,0 m above level of ground	2,0 m	2,0 m	–	0,8 m above level of cellars	–	0,8 m
Side of the World	S–W	S–W	S–E	S–W	S–W	N–E	N–E

In consideration of appeared nitrates and chlorides in masonry renovation of plaster on renovated base which had total thickness about 3 cm was accepted. Renovated plaster to height of cornice was recommended for external elevation. During reproduction of rustics it was necessary renovated plaster which had total thickness above 3 cm. Plastering of each walls and vaults by means of purely lime renovated plaster there was recommended in cellars.

5 CONCLUSIONS

In this paper only chosen problems of strengthening of the Palace building construction are presented. Elements and methods of structural strengthening were applied after achievement of compromise with conservatory services as to possibility of interference into historical buildings substance (disassembly of floors, as well as local damages of façade appropriated for

Table 2. Results of plaster examination.

Sample	1	2	3	4	5	6	7
PH	8,2	8,2	8,1	8,2	8,5	8,4	8,3
Contents of water %	0,6	1,0	1,1	1,0	3,4	1,8	1,8
Contents of chlorides (scale 0–3)	0	0	0	0	2	0	1
Contents of sulphides (scale 0–3)	0	0	0	0	0	0	0
Contents of chlorides (scale 0–3)	1	1	2	1	2	1	1

Figure 6. General view of the Palace in Dobrzyca after executing strengthening and renovation works.

reconstruction and repair). Protecting works described above are in the majority executed. Strengthening and repairing works are executing now in the limits of ceiling over first floor and rafter framing. Destabilized chimneys were totally re-erect. The fragments of roof framings are exchanged and repaired surfaces are covered by copper sheets. It should be noticed that presented above realization complex constructional protection of building will permit the stabilization of its bearing arrangements and will allow making further conservatory works. In results of executed strengthenings up to the present, spatial stiffness of the building was considerably raised, protecting it from renewed constructional damages. Possibility of reconstruction and preservation of historical interior decoration of the Palace as well as its façades was obtained in this way.

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