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Failure and overhaul of a historic brick tower

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1. Introduction

Preserving in a good technical condition of historic buildings as cultural heritage is one of the priority tasks of local authorities, practically in every country. Unfortunately, this often does not translate into real help for owners or users of such buildings. As a result, both the current technical condition assessment and the necessary renovation work are not performed for long periods of exploitation. Users and owners of historic buildings often have problems with their proper use due to their deteriorating or even bad technical condition. This is usually related to the age of this type of construction and is associated with various factors and causes of damages. Very often only the appearance of visible damage becomes a signal to carry out an in-depth analysis of the technical condition of the object and conduct a renovation or strengthening works.

Generally, there are several negative influences, which deteriorate the technical condition of the buildings (not only historical one) and cause structural damage various in range and intensity. They can be divided into following groups connected with:

1. natural processes, which result from the aging of the structure [1],
2. structural solutions, i.e., irregularities or errors at the design, construction or renovation stage [2,3],
3. environmental conditions, which include typical weather impact (wind actions, rainwater, hail) [4,5], chemical pollutions [5],
4. natural seismicity [6,7],
5. geotechnical conditions, which cause changes in the underground water level or irregular settlements [8,9],
6. human activity, i.e., dynamic influences and urban traffic inducted vibrations [10,11], influences (paraseismic shocks, blasting vibrations) connected with coal mining exploitation [12–14] or improper use or overloading.

Mostly, the deteriorating condition of the building is related to several of the above factors and not only one. That is why it is so important to properly recognize the causes of damage because only correct diagnosis provides a correct and permanent repair. In many countries were elaborated and published special recommendations, guidelines, instructions or other documents, which help the engineers with proper analysis of damages, especially in the case of historic masonry buildings. First, more important elaborations on this subject were published in the 1990s in the UK, by Building Research Establishment [15,16]. More detailed guidance in Europe

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was elaborated and introduced into engineering practice at the beginning of the third millennium. Based on the Principles for the Analysis, Conservation and Structural Restoration of Heritage Buildings and Structures (ISCARSAH Principles), approved in 2003 by CIB W23 "Wall Structures" Commission, the final Guide for the Structural Rehabilitation of Heritage Buildings was published in 2010 [17]. Also, other authors presented similar documents, e.g. [18–20].

In southern Europe the main problem with all types of buildings, especially masonry heritage structures is connected with the seismic activities. Various analysis, which include prospective failure mechanisms [21,22] or influence of different retrofitting methods/elements [23–25] on the masonry structure subjected to dynamic influences are conducted to improve the seismic stability or protect the structure against unexpected earthquakes. Other types of dynamic influences, which are characterized with relatively low magnitudes are paraseismic activities, observed in the areas with mining exploitation and human-induced vibrations (produced by urban transportation systems). Also here, the appropriate protection of existing buildings [26–28], very often similar to that used in buildings exposed to low or moderate seismicity [29] is a key matter. As the most important in this field as well as very useful in case of preservation of masonry historical buildings situated in natural seismic regions, are the documents and guidance elaborated and published as the final result of the European international grant NIKER (implemented in years 2010–2012), coordinated by University of Padova, especially results of realization of Workpackage 3 [30] and Workpackage 10 [31] or American Getty Seismic Adobe Project (final report was completed and published in 2000 [23]). The rehabilitation of heritage masonry buildings and another type of structures should be analyzed not only from the technical point of view but also from the safety point of view. The problem of risk assessment is very important [32].

The analysis of the technical state of each building, including heritage masonry structures should be based on the specifics of the materials and structural solutions representative for buildings erected in a given period. Generally, the masonry buildings built in Europe about 100 or more years ago have typically wall load-bearing construction systems consist of solid single-layer walls located in both orthogonal directions. Such an arrangement is very good, and it guarantees the appropriate overall safeness of each part of the multi-sectional building as well as the whole object. The load-bearing walls of 100 years old buildings, especially in Upper Silesia terrains in south Poland were usually made of clay solid bricks with lime or cement-lime (rather weak) mortars with external, façade layer made of clay clinker hollow bricks. These types of bricks usually have a horizontal perforation. This construction of the external wall is correct and guarantees the good behaviour and in-plane deformation due to temperature influence. Empty spaces in facing bricks protect the inner part of the wall against thermal deformations, but at the same time significantly reduce the durability of the façade even when the smallest cracks or damage occurred.

In order to present the problems with the old historic structure a history of the university building located in Upper Silesia and utilized for over 100 years has been presented. After such a long period of the facility operation, without proper inspections and maintenances, the tower which was a part of the building has revealed unexpected and dangerous local failure. The main objective of this study is to analyze and discuss the causes of this local and, as the inspections that have been conducted have demonstrated, extensive damage to one of the walls. In addition, the stages of remedial work were discussed, from the standpoint of keeping the historical significance of the object.

2. Construction of the building and brick tower

Originally, the building was the location of the Royal School of Machinery Construction and Metallurgy (Königliche Maschinenbau und Hütten Schule), which was established in Gliwice in 1896 and existed until the first days of 1945. It was erected in the years 1906–1907 according to the design of Wilhelm Kranz. In January 1945, the building and its tower were partially destroyed within the façade and architectural details. In the first half of 1945, it was a military hospital, and since September 1945 it has been used by the Silesian University of Technology for needs of Faculty of Chemistry. The view of the facility in the past time, before World War II (WWII) is shown in Fig. 1a.

The building has a plane outline with the dimensions of 61 × 24 m. The main part of the building has three above-ground floors, basements, and a two-level attic whose lower part is in use. The tower, located in the vicinity of the southern corner and structurally connected to the other parts of the building, consists of several above-ground floors, with the levels of the first three matches the rest of the building. The tower is covered with a hip roof topped with an openwork spire. The structure of the tower slightly extends outside of the face of the entire building. Within the outline of the external walls, the dimensions of the tower are 8.53 × 6.29 m, with the height of the masonry part equal to approx. 33.6 m and the entire height (including the spire) equal to approx. 48.0 m.

Like the rest of the building, the load-bearing structure of the tower is made in the traditional technology as a masonry structure made from clay solid bricks. At the higher floors (in the independent part), the external walls are 0.65 m thick, and at lower floors (within the main building) the thickness of the walls increases gradually to as much as 0.82 m. On the inside, the walls are covered with lime plaster, and on the outside, they are clad with horizontally hollow clinker bricks with the side surface of 120 × 70 mm and with the width of 120 mm, supplemented with various types of cornice and window shapes typical of Neo-Gothic architecture. The geometry and structure of the tower is shown in Fig. 1b and c.

The floor slabs in the tower are reinforced concrete multi-core Klein type slabs, made from hollow ceramic blocks typical of the period when the building was erected, which are 100 mm high and reinforced with band iron and supported on steel I-beams that correspond to the current IP260 profile, placed every 1.50 m. The beams are supported in seats made in longitudinal walls of the tower. The wooden floors were supported on ground beams, with the free space filled with slag mixed with rubble.

The tower is covered with a wooden truss roof whose load-bearing structure is composed of two girders, each consisting of a ceiling beam (tie beam), two hangers, a strut, and two braces. In the vertical direction, the load-bearing structure is a spatial purlin roof structure. The intermediate ceiling beams are simply supported elements, located on the walls of the tower.

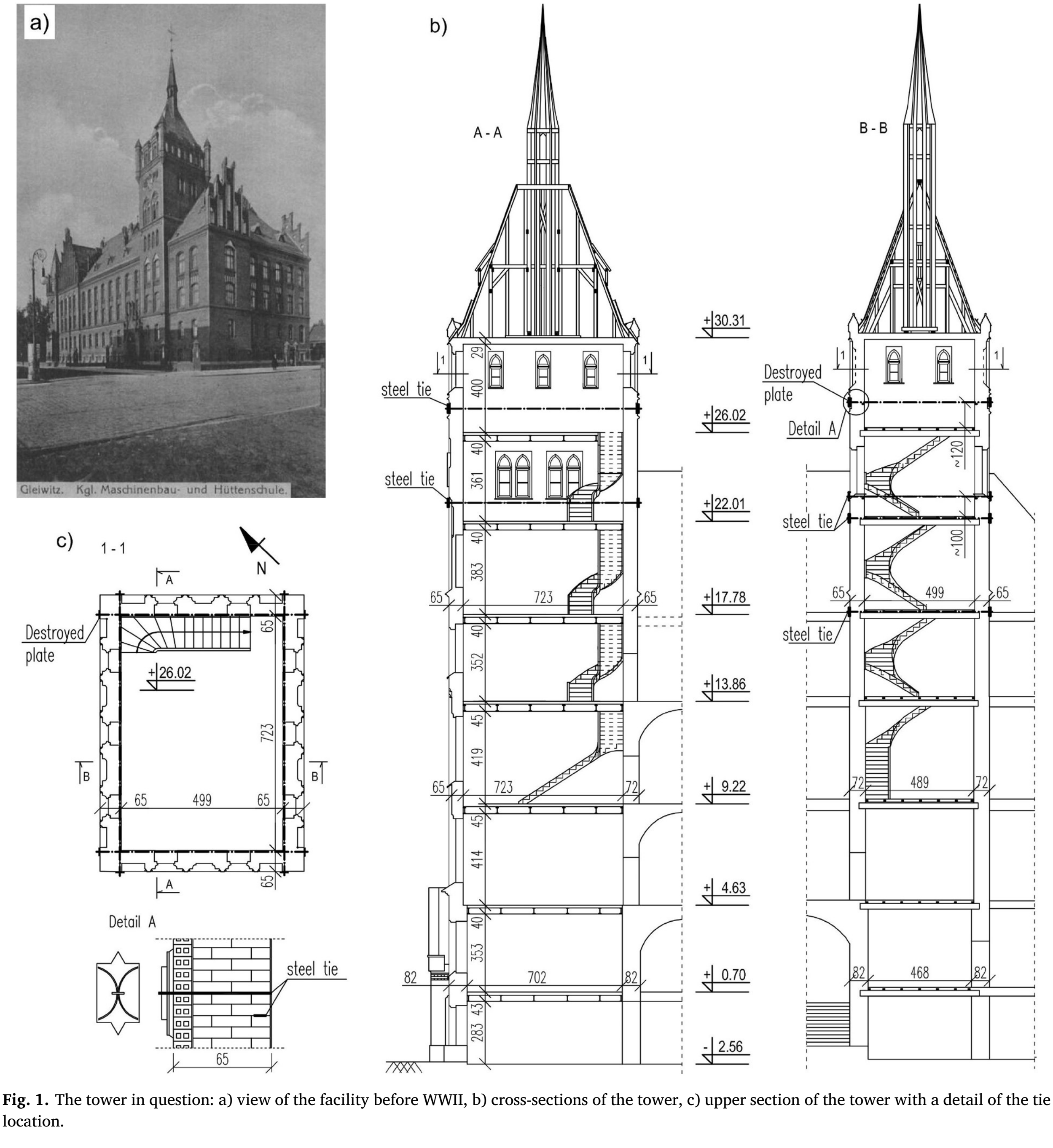

Fig. 1. The tower in question: a) view of the facility before WWII, b) cross-sections of the tower, c) upper section of the tower with a detail of the tie location.

The roof cover is made from double plain tiles installed on battens, counter battens, and fool boarding. The small tower and the lower parts of the roof are covered with sheet metal.

In the second half of the 1950s, an intensive mining exploitation was carried out in the area of Gliwice city. This mining activity resulted in frequent vibrations and dynamic paraseismic shocks. According to archival photographic documentation, it was at this time that the highest part of the tower has been braced using the steel ties located into all external walls. The ties made from 80×8 mm flat bars have been built in both directions (see Fig. 1b and c). The flat bars were anchored in external steel plates with the dimensions of 290×380 mm and the thickness of 12 mm, which are visible on the outer surface of the walls. The ties are installed inside the floors or in chases made in the internal layers of the walls. The circumferential ties protected the upper part of the tower against mining shocks. Additionally they connected the façade with the load-bearing wall. The introduction of the ties is typical and wide being in use in coal mining terrains in European countries for over a hundred year. This type of protection is effective also in case of masonry buildings (especially adobe masonry) located on typical seismic areas [23,25].

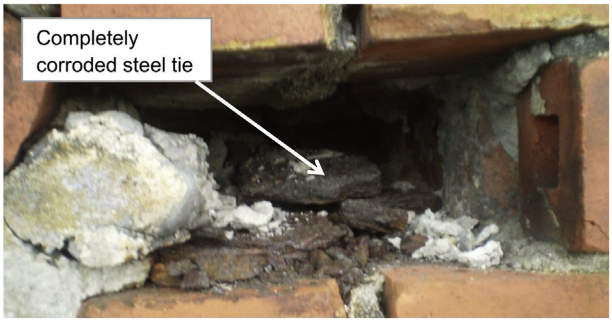

Fig. 2. Total corrosion of the steel tie.

3. Unexpected local failure

In the autumn of 2010, without any warning whatsoever, one of the external anchor plate of the steel tie fell off of the structure of the tower. After breaking through the main section of the roof of the building adjacent to it, it became stuck in the ceiling above the highest floor. The detached steel plate was an anchor of the steel tie, which was placed over 29 m above the ground (it corresponds to the level of 27.02 m in the building). The tie was one of the ties in the group of circumferential horizontal braces (see Fig. 1c) more than 1.0 m above the ceiling of the highest floor.

There was a serious suspicion of the very bad condition of the other anchoring elements. Therefore, the user of the building, concerned by this situation, ordered an in-depth analysis of the technical condition of the tower, which started in November 2010.

4. Methods of determination of the technical condition

4.1. Exterior inspection

Due to an emergency situation, an in-depth inspection of the tower was conducted. The basic visual inspection was made on the outside of the tower, using a firefighter lift. Due to the difficult access (one of the key crossroads in the city had to be partly blocked), the inspection was performed only on the front (south-western) wall and the adjacent north-western wall.

During the visual inspection, several very disturbing and serious damages in the structure of the tower were identified. They were divided into two principal groups: damages of the façade layer and damages of the structural components.

The damages of the façade layer were extensive and characterized by different intensity. The most important are presented below.

- Total destruction of the steel tie (Fig. 2) in the place of the detached anchorage plate. The failure occurred as a result of a deep corrosive process.
- Bad condition of some other anchoring elements. A few plates was seriously degraded due to corrosion, which caused delamination of the separated steel sheets. In original, the anchorages were not fastened to the façade but, instead, only pressed against it by force caused by the tie, which was equal to zero when the corrosion was serious. Fig. 3 shows the condition of the remains of the anchor plate, which was detached during the inspection. This plate was also located at the level of 27.02 m, on the western corner of the tower. The corrosion products coming off in sheets.

- Very deep vertical cracks along the western corner of the tower (Fig. 4). On the outcrops of some cracks, there were traces of epoxy mortar injected in the past, a large part of which leaked to the outside.
- Dislocation (movement to the outside) of a significant part of the western corner of the tower cut off by the cracks (Fig. 5).
- Very significant detachment and bulge of the façade layer of the wall (on the surface area of several square meters) above the windows of the second highest floor (Fig. 6) on the north-western side; the delamination of the wall was located within the external edges of horizontal perforation of the clinker bricks.
- The numerous local loosened clinker bricks and cornice elements in both inspected walls (Fig. 7), at the level of the highest floors, and falling pieces of the clinker bricks had damaged the main section of the roof cover.
- Local dislocation of adjacent brick layers.

The damage to the structural elements was present only in the external load-bearing walls and the steel bracing ties installed in them. A detailed description of those problems is provided below.

- Destruction of the solid bricks in the load-bearing layer covering only the outer part of the wall (Fig. 8); this type of failure was in large part identified later, during repair works, after the damaged façade was removed.
- Significant destruction of the mortar, down to the depth of approx. 50 mm.

Fig. 3. Remains of the corroded steel anchor plate

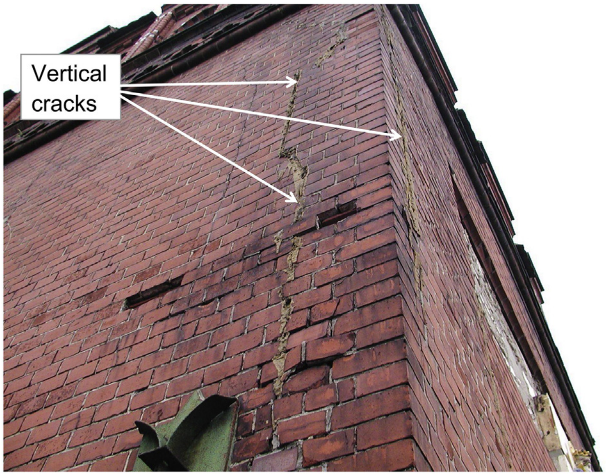

Fig. 4. Vertical cracks in the western corner of the tower.

Also, traces of previous repair works performed using bricks other than the original ones and stiff cement-lime mortar or resin-epoxy mortar were identified on the façade locally (Figs. 5, 7). Some of these works were done just after the end of the World War II.

Other walls and all structural elements that were not available during the first inspection were checked in detail during renovation works. The tower was then completely built up with a scaffolding, which allowed easy access to the entire surface of the walls. The technical condition of the tower inspected at that time was good; much better than in the parts checked at the first stage. This directly resulted from the geographical orientation of the building. The strongest damage (presented above) was observed only in the north-west wall including northern and western corners.

4.2. Interior inspection

Next, a thorough inspection was performed of the interior of the tower, namely the spaces occupied by employees and classrooms/lecture halls. During the inspection, few damages, with low intensity and range, were identified. The problems observed were intensive moisture of pillar between window openings of one of the external wall of the north-western façade of the masonry tower whose location corresponded to the location of the most serious damage in the façade.

Verification of the technical condition of the steel ties was carried out on local uncovered areas inside the building. All ties were checked at the anchorages and at selected points along the length. Generally, the technical condition of ties located at the levels of

Fig. 5. Dislocated part of the newly built-in bricks (on the stiff cement mortar).

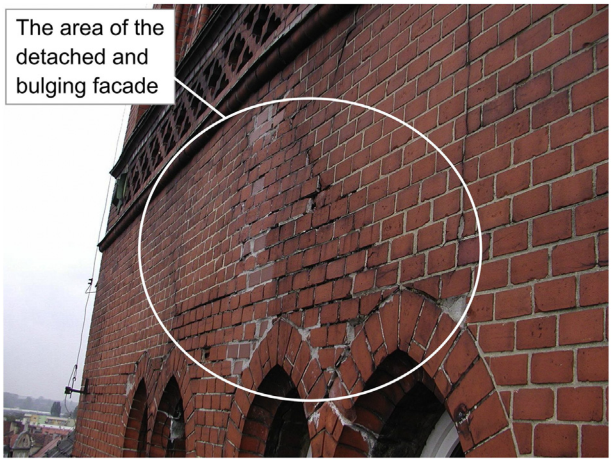

Fig. 6. Detached and bulged part of the north-western façade layer.

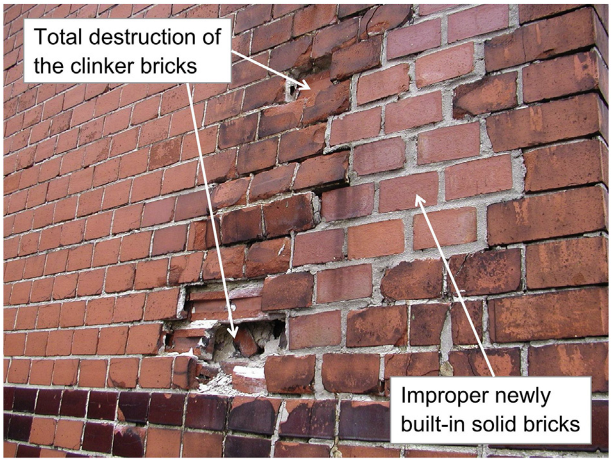

Fig. 7. Very strong destruction of the clinker bricks.

17.5, 21.9 and 22.8 m was good; only local corrosion damage of small intensity was found. At the level of 27.2 m, in the northern and western corners, the ends of the ties were completely destroyed. The corrosion of the flat bars was so large that only single pieces of steel remained (see Fig. 2).

Fig. 8. Destruction of the solid bricks visible on the surface of the load-bearing layer.

4.3. Non-destructive investigation

Consent of the conservation authorities for the performance of low-destructive tests and collection of an appropriate number of samples from the structure was not obtained. Thus, the authors used a less accurate method to determine the basic parameters of the wall - non-destructive tests (NDT), which are acceptable in such situations and provides sufficiently reliable results [33,34]. The compressive strength of the masonry elements (bricks and horizontally drilled façade shapes) was determined using Schmidt rebound hammer which tests the hardness of the surface layer. The value of compressive strength of mortar in wall joints was determined using a pendulum rebound hammer. The results of the tests are summarized in Table 1. Of note is the rather large spread of the average values. This, however, is completely understandable considering the testing technique (non-destructive sclerometry) and the type of material tested.

4.4. Evaluation of the technical condition

An analysis of the identified damages required an analysis of two aspects of the condition of the structure: detailed (threat of local failures) and general (threat to the safety of use of the entire structure).

Due to the possible existence of further local failure (parts of the façade falling off), the technical condition of the tower was declared to be very bad. The face layer, detached and delaminated in significant portions of the façade, posed the risk of an uncontrolled drop onto the roof and the sidewalk in front of the main entrance to the building (see Fig. 6). Similarly, further steel anchors of the ties could fall off.

In the main aspect, most of the problems described herein concern the façade layer of the wall. Nevertheless, significant damage of the load-bearing layer of the structure was identified locally, as well as extensive corrosion damage to a part of the steel ties, which lead to the suspicion that the technical condition of the remaining tie elements is very similar. Also, the unfavorable location of analyzed ties at a significant distance from the rigid horizontal shields (1.15 m above the ceiling of the highest floor) has caused additional bending of the walls. In the long term, both observed damages of the structural components posed the risk of reduction of the load-bearing capacity and the rigidity of the entire structure of the tower.

5. Analysis of the failure process

Based on the visual inspection of the whole structure, three main causes of the observed damage of the tower were identified:

- exposure of the tower to paraseismic shocks in the years 1950–1980,
- many years of exposure of the structure to the normal environmental conditions and
- the influence of traffic-induced vibrations.

Table 1
Results of non-destructive tests for masonry units and mortars.

Layer of the external wall	Masonry unit $f_{b,av}$ [MPa]	Mortar $f_{m,av}$ [MPa]
Load-bearing part of the wall	15.4–17.6	5.4–7.1
Façade part of the external wall	14.7–18.2	6.8–7.5

The first type of damage to the tower occurred as a result of mining activities. Due to mining operations conducted in the past, the tower was exposed to numerous paraseismic shocks. They caused strong vertical cracks in the corners of the tower. This damage covered only the façade layer, which was not bound to the load-bearing wall (it was checked during the renovation works). Those cracks have been repaired in the past, but with low effectiveness. At present, no new cracks of this nature were observed; all noticed cracks arose in the past.

The two types of damage resulted from impact of typical weather conditions. The highest intensity of damage was observed on the wall most exposed to rain and wind. The process of damage development to the structure took place in several stages, described below.

Also, there were some independent factors (vibrations and improperly performed repairs), which accelerated and intensified the observed damage.

5.1. Initial damage in the façade layer

After the first damage occurred in the mortar in the wall joints, the water penetrating the leaky mortar in the façade layer reached the horizontal perforation in the façade bricks and froze there. The repeated freezing and thawing cycles have led to the clinker brick break in the plane of the edge of the furthest perforation and, as a result, caused further unsealing of the façade. This was particularly noticeable in the north-western façade, which was the most exposed to the rainwater and, in consequence to moisture. At the same time, corrosion progressed of the ends of the ties, directly beneath the anchor plates (in the façade layer). What contributed to the corrosion was the fact that inserts made from soft fiberboard were used between the anchor plates and the face of the wall, which easily absorbed water and collected water in the vicinity of the anchors.

The same process took place in the places of improperly repaired cracks of paraseismic origin. These cracks also contributed to the penetration of rainwater into the walls.

5.2. Damage to the load-bearing components of the structure

After the façade layer became damaged, water could easily penetrate to the interior of the wall, which was made from regular bricks and very absorbent mortar. The result was deep frost damage (locally reaching the depth of over 0.4 m) and corrosion of steel ties along their lengths, i.e. in the joints of the load-bearing layer.

The use of a clinker façade layer had the disadvantage that it made it very difficult for the moist wall to dry in locations where the clinker layer and the mortar were in good condition. As a result, also non-defective parts of the façade layer, which was exposed to steam at increased pressure, became detached. The problem was aggravated by the lack of any mechanical joints between the façade layer and the load-bearing layer - they were not anchored.

5.3. Impact of improperly performed repairs

An additional cause of the damage in the façade layer was the improper performance of repairs in the past. In those repairs, bricks with cement mortar were used, which resulted in parts of the wall characterized by lower deformability, thus causing detachment of the newly repaired layer of the wall from its internal part. Additionally, cement and cement-lime mortars are characterized by significantly lower steam permeability; as a result, moisture is evacuated from the wall not through the mortar in the joints (as it should) but through masonry elements, thus causing their damage and the resulting failure. Moreover, those components were not properly bound to the original brick structure. As a result, the original façade was locally burst through thermal stress and steam

Fig. 9. Improperly made repair works, which are escalating the further destruction.

pressure.

The visible attempts to inject epoxy resin mortar into some of the cracks (Figs. 4, 5, 9) were unable to join the cracked parts of the structure. On the other hand, due to the significant difference in rigidity between the original mortar and the injected material and due to the injection under pressure, the repairs contributed to the local deterioration of the condition of the structure. The rigid bands of the epoxy mortar acted as wedges, especially at fast changes in the temperature of the façade.

5.4. Negative influence of vibrations generated by transportation system

Additionally, there was another negative factor, which caused a significant escalation of the degradation process of the tower structure. The analyzed building is located at the corner of the crossroads in the city centre. For several dozen years, very intense transit traffic through the city was taking place near the building. It is also important that the intense traffic took place from the western and southern sides of the analyzed building. This meant that the structure was exposed to some dynamic vibrations induced by the traffic of wheeled vehicles. Unfortunately, the intensity of these vibrations has never been measured in the past, therefore the authors were not able to estimate their impact on the behavior of both the entire building and the masonry tower. Of course, this problem is well known [10,11,35,36] and the effect of this type of vibration on a masonry structure is comparable with the influence of seismic activities [29], which act with a small magnitude. Traffic vibrations gave the negative impact on the building, but it was not possible to determine how great this influence was. Those effects have significantly speeded up and intensified the cracking of the façade. The damage development was also facilitated, because the façade layer was previously unstiffened with strong cracks caused by paraseismic shocks. All this provided easier penetration of the rainwater and much faster degradation and failure of the external brick layer.

6. Discussion

The inspection of damages demonstrated that none of all cracks observed on the façade of the tower (mainly on the north-western and south-western walls) reached across the entire thickness of the walls. They covered only the outer façade layer made of horizontally hollow shapes. The location and nature of the damage observed excluded development of the cracks as a result of exceeding of the load-bearing capacity of the walls or a decrease of local stiffness. The safety of the tower, in terms of carrying capacity of the main structural elements, was not at risk. Therefore, it was not necessary to carry out the relevant analyzes and calculations.

It was considered that the emergency situation of the tower structure was caused by typical and very obvious factors - rainwater, frost and dynamic vibrations, which cannot be avoided.

There are many publications, which described the negative influence of the moisture and natural salt crystallisation on the properties of the masonry components [37,38]. All those processes are leading to decay of the porous material, since it escalates all degradation processes as free-throw cycles, biological attack and wind erosion. This phenomenon, which varied in intensity, has happened in the described case. The location and concentration of the damage practically on one north-western wall are due to weather conditions typical of the region. According to meteorological data, westerly winds prevail in Upper Silesia. In addition, wind is often accompanied by rain, which causes rain to be whipped against two façades of the tower: the south-western façade and the north-western façade. The north-western façade is not exposed to sun; therefore, it dries much more slowly than the south-western façade. This explains the distribution of the intensity of damage that was observed. A similar scenario applies to frost corrosion, which has the strongest influence on the wet parts of the wall – in this case, the north-western façade. These parts of the wall were additionally exposed in the winter to the cooling effect of winds. Thus, corrosion damage to the steel tie ends was the most intense on the north-western side.

The condition of the tower described herein also resulted from the dynamic influences caused by traffic vibrations and mining activities, which have occurred in the past at high intensity. The building was erected in the years when vehicular traffic was limited to horse-drawn carts and there were no coal mines in operation nearby. Thus, the tower was not suitable to carry any dynamic loads, as evidenced by the lack of ties during construction and the fact that the clinker layer was not bound to the load-bearing walls.

Gliwice is located at the cross-section of the east-west and north-south traffic routes. Practically until the end of the 20th century, all heavy transit traffic in both directions passed through the city centre. The building in question is located at one of the main cross-sections in Gliwice and, therefore, the intense traffic of heavy vehicles took place from the western and southern sides of the tower. At the same time, intensive coal mining operations were carried out in the area of the city, which resulted in paraseismic shocks, particularly dangerous to high-rise masonry structures. This is why the steel ties were built into the existing tower structure.

The result of those vibrations was vertical cracks, which separated the western corner of the building, and an increase in the intensity of damages directly related to the adverse weather conditions. Those cracks were repaired in the past but, unfortunately, in a very improper way (see 5.3).

Currently, both dynamic influences are significantly weakened. The main heavy traffic was moved out from the city centre thanks to putting into service of two major highways (A4 - east-western direction and A1 - north-southern direction). Also, at the end of the 1990's, mining exploitation within the Gliwice city ended. Therefore, no new damage caused by the dynamic vibration was observed.

The question that needs to be asked is: Was it possible to avoid the situation where the damages accumulated to the extent that caused the threat of a failure or even a catastrophe?

It must be concluded that the persons conducting yearly inspections of the building (as required by the Building Law) did not act with due diligence because a significant part of the damage to the façade could be seen from the ground level using binoculars. Moreover, the administrator of the one-hundred-years-old building whose structure is described herein should have been aware of the inevitability of some destructive processes and, consequently, should have ordered regular preventive examinations of the structure. It is nearly certain that a properly planned and implemented preventive program, for example: very effective in detecting of a local detachment non-destructive thermography [39], periodic static penetration tests [40] or more complex diagnostic with monitoring [41,42] would have resulted in early detection of threats, which in turn would have translated into much lower cost of repairs.

7. Concept of the renovation and taken remedial activities

7.1. Analysis of the possibility to perform repairs

In the case described herein, the only practically available method of performance of repairs was to restore the original condition using the components of the façade recovered during the demolition of the defective parts or using other components of similar shape, texture, and colour. Similarly, the structural elements (the parts of the load-bearing wall and the ties) had to be made from materials similar to those used to make the original ones. For this purpose, the main strength parameters of the solid bricks, clinker shapes and mortars used in load-bearing and external layer were approximated using NDM (see 4.3). The above was due to the need to keep the façade in a condition as close to the original as possible (the façade is subject to protection by historical conservation law).

Another problem was the season of the year - the damages were found in late autumn, which prevented the immediate start of the repair works. It was also necessary to prepare a design and obtain a permit from the historical conservation officer. Due to this fact, the works were divided into temporary works and principal works, as described below.

In the analysis of the scope of the necessary renovation works, the static and strength calculations of the tower were omitted. The structure showed no signs of exceeding of the load-bearing capacity or loss of local stability and all structural elements will be repaired in a way that restores their original parameters. The facility has been used for the same purpose throughout its lifetime and, consequently with the same live loads. In the last twenty years, practically no paraseismic shocks of significant value were recorded and the heavy vehicle traffic was removed from the city centre. All this, as well as the fact of reconstruction of all the damaged parts of the load-bearing walls, made it possible to decide not to perform the computational analysis. The only effect of calculation would be to confirm the correct design and construction of the original object. Despite the significant reduction of dynamic impacts, the decision was made to leave, with a local reconstruction, all the steel ties. These elements, after so many years, mate with the structure, hold the clinker shapes in small range and are an element of architectural decor.

7.2. Temporary works

Due to the season of the year, the repair of the tower could not start immediately. Consequently, a recommendation was made to immediately install nets on the most dilapidated parts of the façade. Also, construction of platforms beneath the most damaged area (Fig. 10) was ordered to prevent clinker and ceramic elements falling onto the roof and the surroundings of the building (see Fig. 6).

Another action that had to be performed immediately was to cover the main entrance to the building located in the wall of the tower with a wooden roof. This place was directly exposed to the risk of falling fragments of the façade. At the same time, works started on the design for the repair, together with the necessary approvals (the building is listed in the register of historical monuments). The works described above were performed on an immediate basis in the course of preparation of the expert assessment report.

Fig. 10. Temporary steel-wooden platform and protecting nets.

Fig. 11. Reconstruction works.

7.3. Final repair works

The repair works started in the middle of 2011 and ended in the autumn of 2012. They included three main directions of activities:

- repair of the load-bearing layer and reconstruction of the façade layer;
- replacement of the corrosion damaged steel ties;
- installation of additional horizontal ties.

Additionally, on this occasion, many conservation works were performed, including the renovation of several decorative elements on the façade.

The scope of the repair of the façade and the load-bearing wall included: manual demolition of all the damaged or loose façade clinker bricks, chipping off and removal of destroyed bricks in the load-bearing layer and then securing of exposed surface and replacing the missing parts of the load-bearing layer (with solid ceramic bricks of class 15 and lime-cement mortar of class M7). Stage of demolition and protection works are shown in Fig. 11. After the load-bearing layer of the external walls was repaired, the missing parts of the façade layer were replaced with undamaged shapes that had been disassembled. Also, all surfaces of the façade were mechanically anchored to the load-bearing layer with additional steel reinforcement, which is a typical action. The repair system based on the special spiral rods made of austenitic stainless steel Grade 316 was used. At the final stage of the works, the entire surface of the façade was cleaned, pointed, and secured with water-repellent products.

The key difficulty at this stage of works was the fact that a significant number of the façade hollow bricks were damaged to the degree that prevented their reuse. Due to the significant costs of production of individual shapes, a decision was made to cut the missing elements from clinker bricks of texture and colour similar to those of the original elements. Moreover, original clay façade bricks recovered during the repair of another building from the early 20th century were used.

Due to the risk of the anchoring plates of the existing ties falling off, additional anchoring of the original plates in the wall was performed. Threaded pins, with the diameter of 8 mm and the length of 0.50 m, fastened with adhesive, were used, which ensured full anchoring of the plates in the load-bearing layer of the wall. The pins were fastened with a resin and then protected with movement joint putty. All plates were stabilized by nuts and washers.

Strong corrosion damage identified on the ties located at the level of 27.02 m and their improper location (more than 1.15 m above the plate) required the installation of an additional set of four ties located immediately above the floor of the highest floor. Due to the historical nature of the building, the restored elements were identical with the original ones. The flat bars of the ties were located in the joints between the bricks (at the internal surface of the walls) and the remaining gaps were filled with appropriate mortar dedicated to the renovation of historic buildings. The appropriate tension in the ties was achieved by pushing wedges between the face of the wall and the anchoring plate and then by stabilizing the plate by filling the free space with cement mortar with strong thixotropic properties. The old steel ties have left and only the loosened parts of the rusty steel pieces have been removed. New anchorage plates were mounted. The new set of four ties was built-in to replace the corroded elements. The location and construction of those ties are shown in Fig. 12.

In the course of the performance of the works, the decorations, having the form of a shield with an eagle, guild crests, and an inscription, which were damaged after the World War II, were also restored by the recommendations of the conservation officer. Luckily, the threats could be identified and eliminated on time and the repaired tower, enriched with restored decorations, can be used safely for many more years. The general view of the tower at present is in Fig. 13. In Fig. 14a new anchorage plates located above the highest floor (higher anchor plates include the old tie system - level 27.02 m; lower plates include the newly mounted

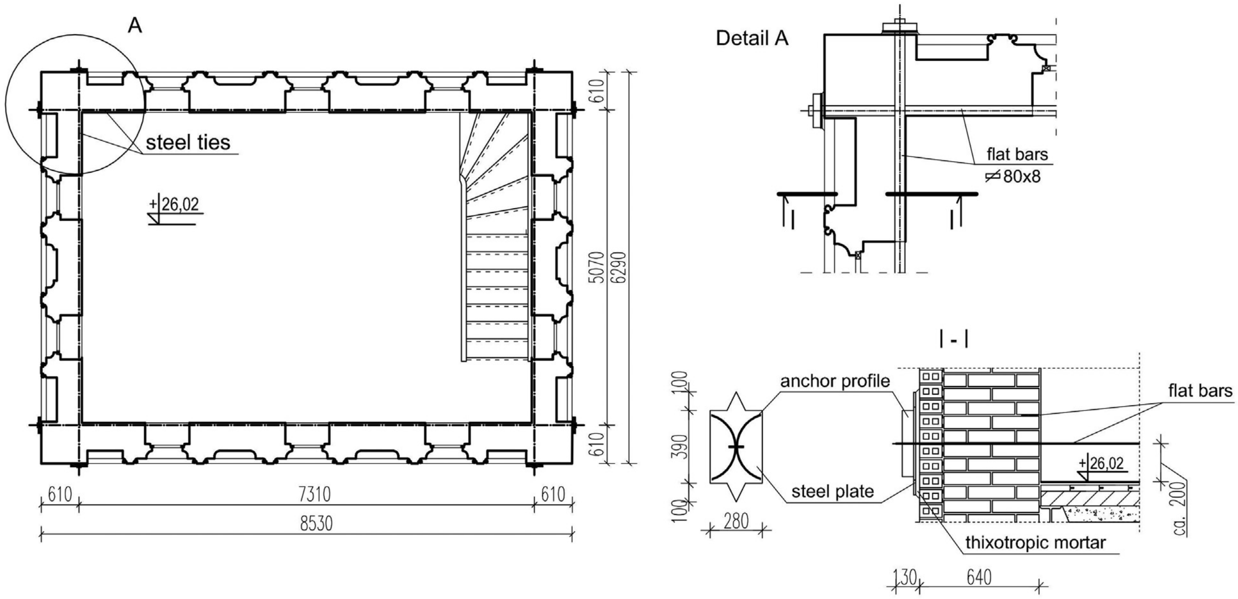

Fig. 12. Detail of the new steel tie; location and overall view.

Fig. 13. General view of the renovated tower.

group of ties) are visible. Both Fig. 14a and b show renovated architectural ornaments.

8. Summary

The case of the brick tower described in the article is an example of a situation where local damage (in this specific case a single tie anchoring plate falling off) brought attention to a serious threat of a failure or even a catastrophe leading to the destruction of the entire structure. The scope of the damage to the façade layer of the walls of the tower posed a real threat of clinker hollow bricks falling off from a great height, and minor damage to the load-bearing layer observed now could with time lead to serious damage of

Fig. 14. Renovated details: a) new anchorage plates, b) architectonic details.

one of the walls.

The damage described here was present mostly on north-western wall of the tower, which – due to its location – was exposed to the worst weather conditions, compared to other walls. In Upper Silesia, westerly winds dominate and, therefore, the north-western façade of the building is most exposed to flooding with rainwater, with very limited possibility of drying out (there is no sun exposure). The bad condition of the wall was getting worse due to the several dozen years of exposure to the dynamic impact of vibrations associated with the intensive heavy road traffic in the vicinity of the building. For some time in the past, the structure was also exposed to paraseismic shocks (resulting from mining operations), which resulted in vertical cracks located in the corners of the tower. The cracks have been repaired, but it was done improperly and with poor quality, which led to intensification of the damage. The causes described above were, in this case, aggravated by the improper construction (no connection to the structure) of the façade layer made from hollow clinker bricks, which is very effective when the façade is fully water-tight but quickly becomes damaged when water penetrates inside the shapes. The failure mechanism was discussed in the previous chapters.

This emergency situation has appeared due to three important negligence, which are:

- owner's ignorance about treats, even such obvious like atmospheric factors, to which historic buildings are subjected,
- lack of the periodic inspections that would lead to the specific repair recommendations,
- proper maintenances, which includes among others an effective repair work.

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