Evaluation Aspects of Building Structures Reconstructed After a Failure or Catastrophe

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Abstract. The article presents the characteristics of several steel structures, among others modernized industrial dye house, school sports hall, truck repair workshop, that have been rebuilt after a disaster or a catastrophe. The structures were analyzed in detail, and the evaluation and reconstruction processes were described. The emergencies that occurred during exploitation of the buildings were the result of multiple mistakes: incorrectly defined intervals between inspections, errors during periodic inspections, incorrect repair work recommendations. The concepts of reinforcement work implemented by the authors, enabling the long-term future failure-free operation of the objects, were presented. Recommendations for monitoring of the facilities, applied after reinforcement or reconstruction, have been formulated. The methodology for the implementation of specialized investigations, such as geodetic, optical, geological, chemical strength tests, both destructive and non-destructive, has been defined. The need to determine the limit values of deformations, deflections, damage or other faults of structural elements and the entire rebuilt facilities, as well as defining conditions for objects’ withdrawal from operation in subsequent exceptional situations was indicated.

1. Introduction
Throughout their life, buildings and engineering constructions should not only meet safety requirements concerning their structures but also be safe for their users. As a result of a number of irregularities at the stages of design, building construction and/or operation, building hazards are likely to occur, leading to construction failures or catastrophes. Based on the analyses carried out by the authors, it was found that the most prevalent causes for the emergence of hazards, failures and construction disasters were the errors made by the participants on all stages of the construction process and also by unpredicted accidental factors. Damaged buildings that have been subjected to reinforcement or repair work may be approved for exploitation [1]. The "sine qua non" condition should be detailed guidelines for building administrators to carry out continuous monitoring of the actual condition of basic structural elements. Over several dozen years, the authors monitored the condition of reinforced buildings of varied constructional solutions. The resulting conclusions have been documented by examples of analysis of changes in the technical condition of the structures rebuilt and put into use after the occurrence of a disaster or threat. The investigated structures are located in the north-east and central Poland.
2. Durability assessment of skeleton walls with steel framework

The modernized building previously housing an industrial dye house located in the plush factory building complex was built about 50 years ago. As the building had been out of operation for several years, it was neither heated nor properly maintained. Originally, it was erected as a two-storey building without a cellar, with reinforced concrete frame construction. Steel columns were made of I-beam rolled profiles 200 mm high and spaced 6.0 m apart. The horizontal bracing in the plane of the wall were made of welded rafters composed of two 120 mm closed section profiles. The lower parts of the skeleton walls were made of prefabricated reinforced concrete slabs, insulated with a layer of cellular concrete, fixed to steel columns by means of steel sheets. In the zones above the windowsills and under the ceilings of the ground floor, wooden windows, partially opening, were mounted. The spaces between the windows were filled with C-shaped glass elements.

2.1. Condition of external wall elements. Research

During operation, the steel support structure of the walls was subjected to advanced corrosion processes. Thickness losses of steel profiles were 30-40\% and locally even up to 100\% (figure 1). The facility had been shut down for a dozen years. The launch of another investment process involved adaptation and repair work.

Figure 1. Illustrations of total degradation of the steel rafters and beams/columns. Welding faults

In terms of the requirements of the applicable regulations and principles of building art, it was necessary to carry out an assessment of the technical condition of the structural elements and then to design and accomplish reinforcement work that would enable further safe operation. However, for financial reasons, a new owner decided to cut repair work to a minimum, improving only the aesthetics of the rooms, using the degraded load bearing structure with the gypsum plasterboards fixed to it. During the replacement of the damaged window frames, a phenomenon of excessive deformation of precast wall panels was observed, which posed a disaster threat to the workers carrying out renovations. Construction work was immediately halted as it was necessary to carry out analysis of construction design, as well as consider possibilities and ways of conducting reinforcement work.

2.2. Evaluation of the construction’s safety
The safety condition of the structural elements of the external walls was determined by the bearing capacity of the steel columns and beams and the quality of welded joints. In the examined construction, the degradation of rolled profiles was accompanied by destructive phenomena in the joint zones connecting the columns and rafters. Basing on the investigations, it was found that the support zones of the steel columns, resting on the foundation walls, had lost their continuity and endangered the safety of users as a result of advanced corrosive processes in a destructive environment. The state of unstable local balance was preserved by combining the lower sections of the columns with reinforced concrete prefabricated external wall panels. The welded joints of the frame rafters with the steel columns did not ensure safe transfer of the operating loads. The technical condition of all connections was assessed as requiring immediate reinforcement [2]. As a result of the inventory studies and measurements of the actual thickness of the steel profiles, it was found that apart from the described defects, resulting from the complete degradation of the steel support sections, the thickness of the remaining elements could allow safe transfer of the operating loads.

2.3. Implementation methodology of securing works

After the dismantling work, the corroded reinforced column support zones were strengthened by introducing reinforced concrete bases. Stirrups were welded to the side of the columns to make it possible to join steel and concrete elements. The reinforcement procedure of the support zone is illustrated in figure 2a.

![Figure 2. a) Reconstruction concept of degraded support zone, b) Reinforcement construction of corroded welded joints](image_url)
The stabilizing structures of the rafter sections on the ground floor level were implemented using sheet metal to reinforce the connection zones. Stiffening ribs made of steel sheets of 8.0 mm thick were used, as shown in figure 2b. The surfaces of all the steel elements exposed as a result of dismantling were thoroughly cleaned of the rust and dust layers by abrasive blasting by sandblasting and protected with an anti-corrosion coating. The façade layer of the outer walls was made of sandwich panels.

The outer partition, shown in figure 3, made it possible to sustain correct insulation properties and to comply with the applicable fire protection requirements. The building was conditionally allowed for maximum five years of operation, provided that its condition was subjected to inspections within a period of one year. Visual inspections to be performed every half year were recommended.

3. Failure of a hall’s roof steel covering structure after fire and its reconstruction
A steel covering structure of a school sports hall located in an educational building complex was investigated. The administrator of the sports hall had prepared the hall for an examination session laying the carpet on a wooden floor and setting up wooden benches and chairs. In the evening, on the day preceding the examination, the room ventilation system, including floor ventilation, was turned on. The carpet covering the entire floor surface completely covered ventilation openings, which resulted in preventing heat dissipation devices from conducting heat away from the under-floor space. The rise in temperature, in the absence of a proper response from the device's safety system, led to a short circuit in the electrical system, which triggered a fire, in a mechanism analogous to the one presented in [3].

3.1. The destructive processes and evaluation of the construction condition
The steel structure of the sports hall was a column-and-beam construction, similar to the one described in [4]. HKS-400 beam columns were spaced in the longitudinal direction at intervals of 6.0 m. Rafters of the total length of about 22 m were made of three IKS-800 welded I-beam sections that had been joined together at the construction site without steel overlays over the weldings. Steel beams and rafters were butt-joined with M24 bolts. The structural stability of the hall was provided by a system of bolts, masonry walls and lattice roofing. Steel purlins made of 140 mm rolled I-beams carried the load elements of the cover, the weight of the snow and wind forces. The main hall cover was made of layered roof panels with styrofoam core. In the design documentation, it was assumed that steel
purlins, spaced every 3.0 m, connected by system joints with sandwich plates, provided sufficient protection against the loss of stability of the frame rafters.

![Image](image1.png)

**Figure 4.** View of the damaged elements. Parts of the loosened lower sheet metal

The action of the fire temperature and consequent rapid cooling resulting from fire extinguishing operations caused permanent deformation of steel structure components and their loosening, as well as a total loss of the load capacity of the sheet metal carrying the tensile stress of the cover. High-temperature fields also affected framework rafters made of welded I-section profiles. During the fire, the roof structure was loaded with snow (figure 4). The investigation procedures undertaken directly included a local inspection, interviews concerning fire origination, approximate duration of the fire, visual inspection of the destroyed sandwich plates visible from the inside, identification of the elements covered with soot layers, and constituted sufficient basis for assessing the existing technical condition as threatening the safety of the people performing repair works.

At the bearing capacity verification stage, calculations were made in accordance with the current standards, although taking into account the actual strengths of the materials under the conditions of the likely reduction in bearing capacity caused by the fire. The criteria and conditions of safety assessment concepts of both measurements and investigations were developed to make such an assessment possible. The quality of bolt joints and butt welds connecting the three steel plate elements of each rafter was assessed. Investigations of the upper parts of frame node zones were also examined (figure 5).

![Image](image2.png)

**Figure 5.** Schematic diagram of the deformation measurement stand
Confirmation of the usability limit state was performed by verification of actual deflections. During the measurements, the roof was not loaded with snow. In order to determine the displacements of the tested elements, loaded according to the design and standard assumptions, a coefficient was calculated taking into account the effect of additional loads with characteristic values. The actual deflection values, as determined in the measurements, were compared with the limit values for girders and purlins.

3.2 Repair of the sports hall
As a result of the conducted research, calculations and analyses, it was confirmed that there were no changes caused by the fire resulting in the necessity of dismantling the support framework structure. The repair works started with demolition of damaged sandwich panels over the entire surface of the cover. The surfaces of the steel elements were prepared for resurfacing and anti-corrosion and flame resistant coatings. Bolts with threads destroyed by the fire were replaced and then welding works were carried out in the frame rafters. Stiffening ribs to reinforce IKS-800 girders made of 8 mm thick steel sheets, spaced every 1500 mm were designed [5]. The safety of repair works was secured by providing scaffoldings under the reinforced rafters. The reinforcement of the butt welds connecting sheet metal sections with welded overlays, taking over the normal and tangential stresses, was designed as shown in figure 6.

The construction of the roof was strengthened by the use of additional stiffening purlins made of rolled steel bars I-section profiles. The upper surfaces of the purlins were covered with sandwich roof plates and firmly fixed to the system connectors. The plates ensured the required conditions of thermal insulation. The reconstructed hall was subsequently put into service. The need of geodetic monitoring of the condition of the deformation of the reinforced roof covering loaded with snow during the winter season was indicated.

![Figure 6. Concept of strengthening of a steel roof covering construction](image-url)

4. Reinforcement and monitoring of industrial building after cover disaster
Longstanding analysis of building hazards, failures and construction disasters have showed that random or accidental factors such as an excessive snow load most adversely affect the safety of buildings with steel and wooden girders. They usually include industrial halls, shopping centers, warehouses, logistic facilities, sports halls and production facilities [6]. The conclusions presented above have been confirmed by another case, where after several days of heavy snowfall and atmospheric temperature oscillating around 0°C, there was a failure of a steel structure covering a production hall of a technical building located in an industrial plant.
4.1. Analysis of structural design solutions of the building. Identification of the extent of the damage

The workshop and production building was designed with a pillar-bar structure [7]. The pillars were made of reinforced prefabricated concrete with variable cross-section. Each column had two braces for the support of crane beams and roof girders. The roof covering was made of corrugated sheet metal, based on steel purlins, transferring the load to welded beam girders type HKS-360. The girder elements were joined in the middle of the span with four M24 bolts (see figure 7). The 9.0 m long purlins were made of I-section rolled profiles 200 mm in diameter, without any bracings. Gable roof girders with a span of 12.0 m, formed with a drop of 5%, were spaced every 9.0 m. External walls were built of autoclaved aerated concrete (AAC) blocks 38 cm thick. In the first phase of the disaster, the roof structure loaded with a layer of snow was damaged. The destruction process was initiated as a result of cracks in the bolts joining the girders in the middle of the span. The pivot points of the girder segments, while falling, after severing bolted butt joints, were located on the column supports. The ends of the HKS beams, anchored on the column supports, while rotating, caused damage to the reinforced concrete structures of the support zones. Steel anchors in the pillar heads plasticized, which caused local narrowing or severing of the bars. The concrete supports of the columns were completely destroyed. The diagram of the degradation mechanism is shown in figure 8.

Figure 7. Structural elements the roof cover

Figure 8. Degradation mechanism of roof covering structure

After the disaster, some parts of the roof cover reached a state of unstable equilibrium after the beams rested on the non-bearing technical floor, as well as the building’s longitudinal walls built of AAC blocks. Some other fragments of the collapsed roof structure also damaged the equipment, as well as the machinery installed in industrial hall before finally being stabilized.
4.2. Safety assessment of the designed structure

As a result of the investigations and calculations, it was found that the main structural defect causing the collapse were errors in the design of butt joints of the HKS-360 steel girders. Computationally, the use of class 10.9 bolts was justified, while in the drawing documentation bolts of class 5.8 had been indicated, and the contractor had used bolts of classes 5.6 and 4.8. The forces acting on the designed joints were much higher than the bearing capacity required to carry loads safely.

Another factor affecting the safety of the roof covering structure was design flaws concerning steel purlins. Static calculations concerning the design of 9.0 m long purlins made of 200 mm high I-beams failed to account for the effect of a possibility of local buckling resulting in the strength reduction [8, 9]. The load carrying capacity of such elements was much lower than the capacity required to carry the actual loads in a safe way. Additionally, the connections of the purlins with girder beams were faulty.

4.3. Concept of object reconstruction

The industrial object destroyed by the disaster had to be rebuilt without any delay due to the requirements of its technological cycle [10]. As a result of the conducted investigations and analyses, it was determined that the loads from the rebuilt roof structure should be transferred to reinforced concrete columns in an axial manner using existing elements of the structure. Damaged short reinforced concrete supports were discarded as load bearing elements. A rigid reinforced concrete crown was designed for the upper part of the walls. Inside the crown sockets were introduced to support the HKS girders. The reinforcements of the central and support beams of the girders and lateral stiffening between the purlins have been designed. The concept and implementation of reinforcement of the butt joint of girder elements is illustrated in figure 9.

5. Monitoring guidelines for buildings put into operation after reinforcement

As a result of the analysis of the conclusions arising from the presented examples of assessment and monitoring of the condition of the damaged or disaster-prone buildings, recommendations were made to the partakers in all stages of the construction process. In view of diverse construction of the surveyed buildings, each case should be provided with specific recommendations and guidelines for monitoring and evaluation of the structural elements. Such guidelines should be formed by the building designers or by qualified construction experts.

For objects that have been reconstructed, reinforced or repaired, the most important issues should include:

- verification of as-built documentation and statements of site managers that the construction had been completed in accordance with the approved design documentation;
ongoing supervision of the execution of the directives and recommendations resulting from previous inspections. Particular attention should be paid to observing the imposed restrictions on operating loads;

- explicit indication of the frequency of inspections and technical assessments of building structures as well as clear definition of the operation period;

- description of the so-called “weak spots” in inspected buildings and engineering structures, listing aggressive atmospheric and technological factors.

A separate issue is the indication of the methodology of carrying out specialized tests, such as geodetic, optical, geological, chemical and durability tests, both destructive and non-destructive, to the extent necessary for an effective evaluation of the condition of the facility, and above all its state of emergency.

It is also necessary to indicate the limit values of deflections, damage or other deformation of structural elements, as well as entire structures, and precisely define the conditions of their exclusion from operation in exceptional situations.

6. Conclusions

The exploitation and maintenance of building structures approved for operation following their reinforcement, repair or reconstruction caused by emergency requires the owners/administrators not only to carry out periodic inspections of the structures and implement the resulting recommendations, but also to perform ongoing check-ups of basic structural elements of building structures. Such activities can be described as continuous monitoring, which may, in the future, protect the objects and, most of all, their users, from another unexpected or unpredictable failure.

The authors have repeatedly argued the fact that the administrators, mostly for financial reasons, ignore the recommendations resulting from the expertise, decisions of the authorities and binding regulations concerning periodic inspection of buildings and their proper maintenance.

Another important problem is to observe the deadlines for building’s approval indicated in pertinent opinions and expertise, as well as to provide timely, specialized works under the supervision of authorized employees. It is also advisable to engage the authors to supervise the execution of their concepts or designs of reinforcements or reconstruction of the objects.

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