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Original Research Article

CONSTRUCTION DISASTER HAZARD CAUSED BY A DEFECTIVE BUILDING STRUCTURAL SURVEY

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Abstract

The present paper encompasses an analysis of a production hall reinforced concrete roof structure in terms of its limiting snow load. Errors which were made at the survey and thermal retrofitting design stages for the structure in question have been identified. These contributed to a construction disaster hazard which could occur in the event of intensive snowfall. Reinforced concrete roofing structures are least prone to failures caused by the impact of the weather, however they do occur. This results from both structural wear and also often from the addition of extra loads. These loads may be caused by thermal retrofitting as well as installation of additional equipment on the roof, including the currently popular photovoltaic systems. Using the structure subject to the analysis as an example, the authors have proposed a procedure for diagnosing and analysing reinforced concrete roofing structures in terms of upgrades and installation of additional equipment thereon.

Keywords: construction disaster, structural failure, structure diagnostics, snow load, roofing structure

1. INTRODUCTION

Building diagnostics throughout their entire lifespan is one of the key elements of a maintenance system which primarily aims to ensure structural and use safety [1, 2]. An analysis of the structural state of building elements is usually performed to determine whether the building is fit for further use and also to find the causes of a failure or construction disaster. Building failures and disasters originate at different stages of a building's life cycle, both during the design and construction period or as a result of improper use. Natural hazards and extreme meteorological phenomena which cause the load-bearing and serviceability limit states of structures to be significantly exceeded are frequent causes of building failures and disasters [3]. For buildings built several decades earlier, there is often a lack of design documentation which comprises both the technical solutions and the construction materials used, and often no calculations specifying the assumed impact forces. For these buildings, as part of retrofitting

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or when planning to install additional elements to the structure which add to the load, it is necessary to carry out a survey of the structure and determine the permissible forces.

Heavy snowfall causes high loads, especially on roofs. Standards for determining loads caused by atmospheric phenomena have changed over the years. These were often determined by trial and error, which led to many inaccurate regulations. Only an analysis of snow load failures and disasters fully uncovered their causes and illustrated the nature of the hazard [4]. Unfortunately building disasters caused by fortuitous events, among which weather phenomena play a leading role, are a common occurrence. The disaster which befell on the exhibition hall in Katowice, resulting in its collapse in the winter of 2006, is considered to be the biggest building disaster in Poland, where snow loads contributed to its occurrence. 65 people died and more than 170 were injured during the Katowice International Trade Fair in progress at the exhibition hall. An expert report by the Silesian University of Technology Team (a team appointed by the General Office of Building Control) pointed to design errors, including incorrect design of the structural system and the flat roof structure, and revealed significant discrepancies between design and execution. The flat roof contributed to snow building up on the roof surface and the formation of a layer of ice, which further stressed the structural elements [5]. Another example of a misjudgement of the actual load-bearing capacity of a structure, is the disaster of two warehouse halls in early 2010, built using the ABM system, which comes in the form of cold-formed panels of steel sheets joined to form cylindrical shells. Prior to the disaster, there was only a 10 centimetre layer of snow on the roof of the hall in Gdańsk, which generated a load significantly less than that predicted by the snow load standard at the time [6]. Biegus and Kowal [7] cite an incorrectly adopted computational model during design as the cause of the disaster, which resulted in overestimated results for geometrical characteristics of the cross-sections. Thus, the structure did not meet the strength requirements; its load-bearing capacity was exceeded by 114%. In addition, the operational limit state of the roof was not adhered to with the deflection significantly exceeding the limit [7].

The above examples show how dangerous it is to exceed the permissible climatic impacts on a structure.

2. LAYOUT OF ROOF LAYERS BASED ON A STRUCTURAL SURVEY OF THE BUILDING

According to the provisions of the contract [8] concluded between the facility manager and the contractor, the latter should prepare an expert opinion and a building survey which includes digital documentation of the F20 building on the premises of WSK "PZL-Rzeszów SA w Rzeszowie" with due diligence. The expert opinion was to be carried out to an extent sufficient to achieve the purpose it was intended to serve, taking into account the regulations, the applicable Polish Standards and the principles of technical knowledge. The detailed scope of the expert opinion should include all elements to allow strength calculations of the facility to be carried out and to be able to structurally adapt of the facility to new functions. In addition, the static-strength calculations for the existing elements were to take into account the critical thickness of snow cover at which the roof would need to be cleared of snow. On the basis of that analysis, conclusions and recommendations were to be made regarding the possible reinforcement of individual structural elements and/or the subsoil [8].

The survey of the reinforced concrete structure of the building was used to draw up both an expert opinion of the building [9] and to perform an architectural and construction survey [10]. For the purpose of these studies, site control measurements and inspections were carried out, test-holes were made for selected elements and photographic documentation was prepared. Authors of the aforementioned documents produced a technical description of the building and its structural elements, including foundations, columns, walls, floor and ceiling assemblies, floor and roof assemblies, stairs, floors, finishing works and systems. In addition, they drafted a description of the damage and an assessment of the technical condition of the structural elements of the building in question, i.e. floor and ceiling assembly slabs and ribs, columns, walls and other elements, i.e. roof eaves, roof coverings, and thermal insulation, for which they additionally made, in a graphic form, appropriate drawings of the surveyed elements. As described in studies [9, 10], the dimensions of the cross-sections of the reinforced concrete structural elements (columns, rafters, ribs, floor and ceiling slabs as well as floor and roof slabs) were measured during a site visit. Apart from measuring the cross-sectional area of the elements, measurements were also taken of the thickness of the reinforcement cover. In addition, their works included non-destructive testing of the compressive strength of the concrete of the accessible columns and rafters (for randomly selected locations), using the sclerometric method – N-type Schmidt hammer.

From a formal point of view, an analysis of the expert report [9] as well as the survey [10], found no inconsistencies with the scope of the contract [8]. On the basis of this documentation, a design for the reconstruction of the F20 building was drafted, which entailed thermal retrofitting involving the addition of a 19 cm thick layer of mineral wool and a roof membrane. The design also included strengthening the reinforced concrete ribs of the roof structure with steel beams in the snow drift load zone. According to the calculations in the redevelopment design, the structure was to meet all the recommended standards for use, including [6].

During the course of the building works, whilst making a series of roof openings, the site manager identified additional roof layers not included in the design documentation. According to the construction site documentation, all participants in the investment process (Investor, Construction Works Contractor, Design Supervisor, Investor Supervision) were fully aware whilst the works were being carried out in 2011 that additional non-structural layers of the roof covering were present in the test holes made at locations of openings for skylights. These layers placed significant strain on the reinforced concrete roof support structure of the building

Due to emerging doubts about the permissible snow load values, the Investor commissioned expert reports of the roof structure. The first indicated that internal forces were significantly exceeded and recommended that the structure be reinforced. Due to the identified significant extent to which internal forces were exceeded (by more than 300%), a further study was commissioned [11], the results of which are presented in this article.

3. COMPUTATIONAL ANALYSIS OF THE ROOF STRUCTURE

The reinforced concrete roof load-bearing structure of the F20 building was built in the 1950s. In study [11], the safety of the building roof load-bearing structure in question, rebuilt and extended between 2010 and 2011, was verified. The load-bearing capacity verification concerned the rebuilt part – Figure 1, not the extension, i.e. a three-aisle building with a footprint of approximately 4,200 m2. The building features a three-aisle layout, with two side aisles approximately 9 m high and an approximately 14 m high central aisle.

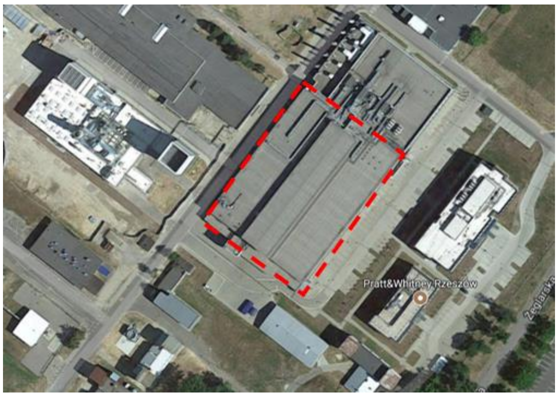

Fig. 1. Location of building F20. The part rebuilt between 2009 and 2011 is marked in red [11]

A test report was prepared in order to develop a detailed computational model of the structure, taking into account the actual geometry of the sections and the parameters of the incorporated materials [12]. The scope of works for this report included, inter alia:

- local test-holes at structural elements of the roof, including rebars;
- collection of concrete core samples for laboratory analysis, including for the needs of survey measurements of the thickness of the existing roof layers;
- laboratory analysis of the material samples;
- measurements of the geometry of the reinforced concrete elements of the roof load bearing structure.

As a result of these investigations, significant errors (defects) were found in expert opinions and the survey. These errors included both incorrect thicknesses of the reinforced concrete floor and roof assembly slabs of the central aisle and side aisles. The survey documentation [9, 10] specified the thickness of the reinforced concrete floor and ceiling assembly slabs to be 8 cm, in reality the thickness is 7.5 cm for the central aisle and 5.5 cm for the side aisles. In addition, an incorrect – incomplete – arrangement of the flat roof layers was found: only two layers of felt were specified as waterproofing, while in reality there were additional layers under the felt: for the side aisles (lower parts) – 10 cm thick *suprema* panels and a 6 cm thick concrete (levelling) layer, and for the central aisle – 8 cm thick *suprema* panels and a 7 cm thick concrete (levelling) layer. Figure 2 below shows an example of the layout of the roof layers above a side aisle.

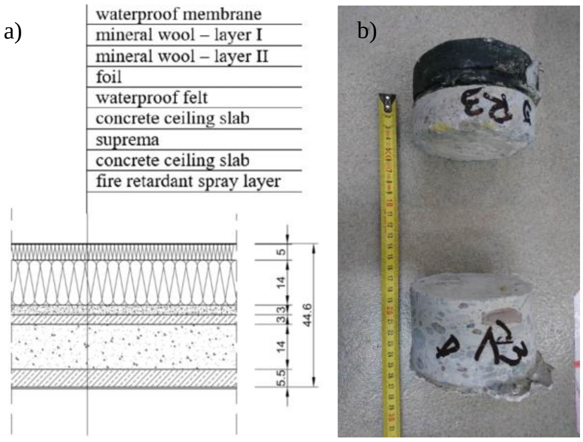

Fig. 2. Layers over the side aisle, a) layering, b) drill-hole elements

As a rule, the following design parameters were assumed for the overall structure, based on the results of report [12] and expert experience with reinforced concrete structures designed and constructed in the mid-1950s:

- C12/15 concrete strength class,
- reinforced concrete roof slabs reinforcement in the form of ø 6 mm smooth bars at 12 cm spacing (based on ferrometer scanning of the reinforcement),
- thickness of concrete in reinforced concrete roof slabs: 5.5 cm for the side aisles and 7.5 cm for the central aisle,
- main reinforcement of the lower aisle rafters in the form of steel of A-II strength class; quantity of reinforcement – two ø 26 mm rods and four ø 22 mm rods (in the four test-holes carried out, a different distribution of reinforcement was found, with three rods repeated in two layers),
- primary reinforcement of the central aisle rafter in the form of steel of strength class A-II; quantity of reinforcement – six ø 26 mm bars (ø 28 mm bars reinforcement were found in one test-hole by a column),
- static diagram of roof slabs – multi-span continuous beams supported on ribs),
- static diagram of roof ribs with girders – frame with rigid nodes: this assumption was made due to the large differences in the stiffness of the cross-sections of these elements and the additional haunches of the ribs at the joints, which further reduce the span moment in the ribs.

The reinforcement varied greatly. For example, there were two variations in the most stressed structural elements of the side aisle, i.e. the ribs:
- variant I – 4 smooth ø 16 mm bars, design cross-section load bearing M_{Rd} = 30.58 kNm,
- variant II – 3 ribbed-finned steel bars with a square section of 12 mm x 12 mm, design cross-section load bearing M_{Rd} = 30.65 kNm.
The less favourable variant I was used for the numerical analysis.

A computational analysis of the roof load bearing structure was carried out during the next stage. A schematic diagram of the bar numerical model of the reinforced concrete roof grid of the side aisles, together with the results of the numerical analysis, are presented in Fig. 3.

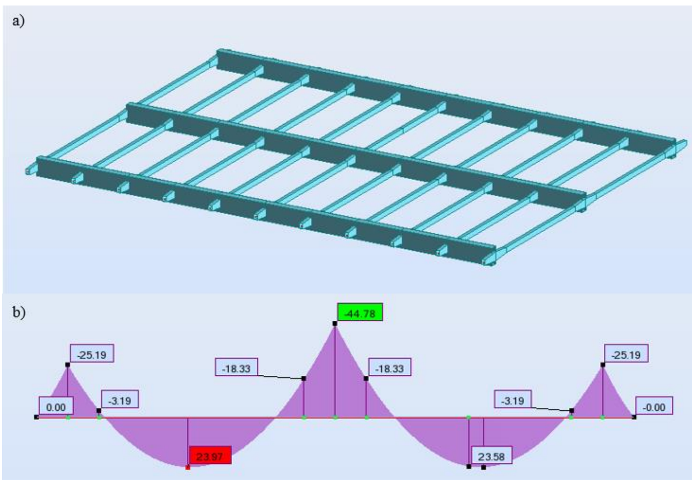

Fig. 3. a) Schematic diagram of the reinforced concrete grid of the roof of the side aisles, b) calculation results – determination of the maximum rib span moment [11]

Then the maximum moment value for the roof rafters was calculated by adding columns to the numerical model – Figure 4.

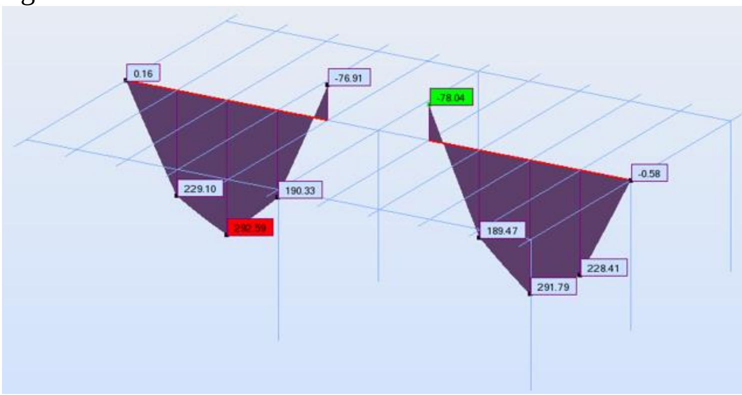

Fig. 4. Analysis results - maximum span moment of a side aisle roof rafter [11]

For the central nave, a static diagram of the roof structure, which also constitutes a reinforced concrete grid, whose reinforced concrete cross-sections were adopted in accordance with the actual cross-sections of the structure; among other things, the variable cross-section of the ceiling ribs was taken into account. The numerical model takes into account the section of the structure between the hinges of the Gerber beams, i.e. a section in the form of a two-span beam with supports on both sides. Structural weight loads were applied as continuous over the full length of the floor and ceiling assembly slab beams and concentrated forces at their ends from the middle of "suspended" spans. A schematic diagram of the bar numerical model for the central nave roof load bearing structure is shown below in Figure 5.

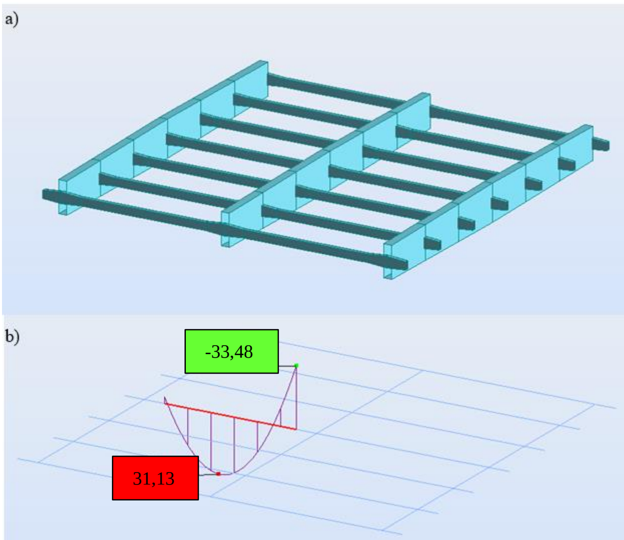

Fig. 5. a) Schematic diagram of the reinforced concrete grid of the roof of the central aisle, b) calculation results – determination of the maximum rib span moment [11]

By comparing the determined load bearing of the structural elements of the side aisles and the central aisle roof with the values of the internal forces, the lead bearing reserves of these elements were determined. This allowed the permissible specific snow loads to be determined; the results of the analyses, in the form of permissible snow loads in kN/m^2, are presented in Table 1. The analysis took into account the dead weight loads of the roof layers, determined from the measured thicknesses.

Table 1. Permissible snow load for individual roof structural elements [11]

Struct. element location	Permissible specific snow load [kN/m^2].		
	slab	rib	rafter
Side aisle	1.58	0.83	1.66
Central aisle	3.21	2.15	0.46

It should be noted that for both the side aisles and the central aisle, the occurring values of permissible snow load were lower than the normative values for the Rzeszów region [14]. In Table 1 they are marked in red. The normative load for the central aisle was 0.72 kN/m^2, for the side aisles taking into account the possibility of snow drifts was 1.28 kN/m^2. This meant that in order to ensure the safety of the reinforced concrete roof support structure of building F20 during the winter period, it was necessary to remove snow from the roof surface of this building. A snow removal diagram was presented in study [13].

After analysing the results of the structural analysis, it was concluded that the load bearing structure of the roof needed to be adapted so that all load bearing elements met the normative assumptions. It was proposed to remove the redundant roof layers overlying the load bearing structure: *suprema* panels, the top layer of concrete (pressure layer) and the bituminous covering layers, which would result in significant relief to the existing reinforced concrete load bearing structure. Once these recommendations have been applied, the allowable loads for the aisles would be as follows:

– for the side aisles: 3.08 kN/m^2,
– for the central isle: 2.38 kN/m^2,

i.e. values more than sufficient to carry the normative snow loads (including those resulting from snow drifts).

A refurbishment to the proposed extent was carried out in 2019 using the current Eurocode standards. The layering over the side aisles is currently as follows:

- 5.5 cm thick floor and ceiling assembly slab,
- vapour barrier made of 0.4 cm thick felt,
- structure for thermal insulation made using TR 55/0.75 mm metal sheets,
- Therma TR26 FM panels thermal insulation,
- PROTAN roofing membrane,

the total specific load of the layers is 1.56 kN/m^2, and after taking into account the design factor for permanent loads according to EC1, the design load on the roof of the side aisles is 2.11 kN/m^2 – compared to 4.54 kN/m^2 for the previous roof layer arrangement.

For the central nave, the layout is as follows:

- 7.5 cm thick floor and ceiling assembly slab,
- vapour barrier made of 0.4 cm thick felt,
- structure for thermal insulation made using TR 55/0.75 mm metal sheets,
- Therma TR26 FM panels thermal insulation,
- PROTAN roofing membrane,

wherein the total specific load of the layers is 2.06 kN/m^2, and after taking into account the design factor for permanent loads according to EC1, the design load on the roof of the central aisle is 2.78 kN/m^2 – compared to 5.26 kN/m^2 for the previous roof layer arrangement.

According to the results of the static-strength analysis of the building's roof load-bearing structure, the permissible specific snow load taking into account the currently occurring dead weight loads and category H of the service load according to EC 1 [14] is:

- side aisle: 2.63 kN/m^2,
- central aisle: 2.30 kN/m$^{2.}$

A photovoltaic system was installed on building F20, on both the side aisles and the centre aisle (Figure 6). The total specific load from the photovoltaic system and its support structure is 0.22 kN/m^2.

It should be noted that for both the side aisles and the central aisle, the snow load values were higher than the normative values for the Rzeszów region [13]. In its current technical condition, given the operational recommendations related to the maintenance of the roof surfaces during the winter period, there is no construction disaster hazard.

The minimum load-bearing reserve for the roof structure of the entire building gives a permissible snow cover thickness of 234 cm of fresh snow or 117 cm of old snow.

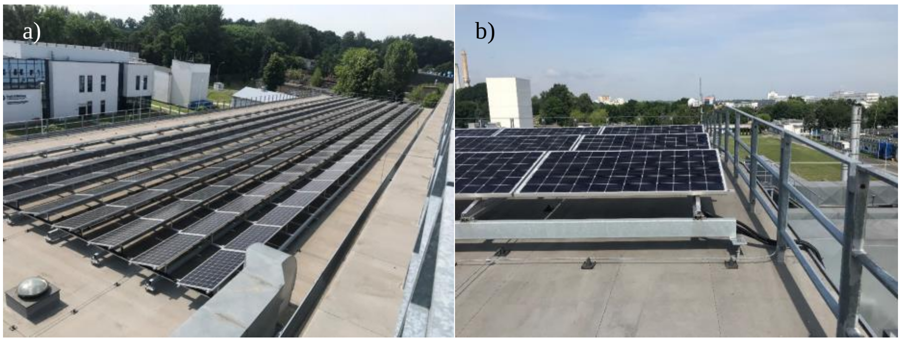

Fig. 6. Roof of building F20 after the modernisation and installation of photovoltaic panels, a) side aisle, b) central aisle

4. DISCUSSION

An analysis of the causes of failures and disasters of buildings subject to snow loads shows that their occurrence is frequent [3]. However, it is directly related to the intensity of precipitation in a given year. Over the years, heuristic methods were used to observe climatic phenomena. Current standards have been drafted on the basis of the aforementioned analyses [4,15]. Analysing how the standards for snow loads on building structures have changed since the beginning of the 20th century, it can be seen that until 1980 the load was successively decreasing [15]. The values of climate impact loads were mostly determined by simple measurements and observations. As views on the safety and reliability of structures have changed, methods for assessing the impact of these loads have evolved. It was only after 1980 that the development of observations and measurements of meteorological phenomena led to the roof snow load being considered as a function of the snow load on the ground. This was a breakthrough in research [4].

Analysing failures and disasters related to snow loads on roofs, the conclusion is that steel structures most often suffer damage. This conclusion is corroborated by the data collected by Żurański et al [4, 15]. They analysed failures and disasters from two winters: 1969/70 and 1978/79. They divided the buildings according to roof structures – steel, timber and reinforced concrete. During the first winter 50% of failures occurred for steel structures. This figure for the second winter was 67%. Analogously for buildings with timber roof constructions – 42 and 30%. Reinforced concrete roof structures proved to be the most reliable, making up 8% and 3% of disasters in 1969/70 and 1978/1979 respectively

[4, 15]. It should be emphasised that the least restrictive normative snow load values for roofs were in force at the time in question.

Analysing the material used for the roof structure, it appears that reinforced concrete roof structures of buildings are proving to be the most reliable under snow loads [4]. Steel structures, on the other hand, are the most susceptible to damage, largely due to the safety provision for the dead weight of the structure. A fairly high percentage of snow load disasters can be attributed to inadequate design or workmanship [4]. There are numerous examples of disasters which have occurred under snow loads of less than the expected weight as set forth in the standards, including [7,16]. There are also known cases where a roof structure collapse was caused by wind loading and ineffective rainwater drainage [17,18].

The existing F20 building in question, characterised by a reinforced concrete roof structure, was designed when the specific snow load was 0.6 kN/m2. The installed insulation comprised a heavy layer of suprema with an additional pressure and levelling layer of concrete. In the case analysed, the term suprema was an exaggeration, as this layer in the test-holes consisted of loose aggregate with pieces of mortar and concrete admixed with sawdust [12] – Figure 7.

Fig. 7. Roof thermal insulation layer, a) sample taken, b) insulation material

As part of the thermal retrofitting, additional layers of insulation were added, albeit lightly. The extension and thermal retrofitting design envisaged local reinforcement of the building's roof structure subject to snow drift loads, although there were still some structural elements that did not meet the load-bearing condition under the normative snow loads assumed for the analysis. It is incomprehensible that, once the existence of additional layers (unforeseen in the expert's report and survey [9,10]) was established, none of the participants of the construction process undertook an analysis of the effect of the additional weight on the load-bearing capacity of the roof structure.

Whilst carrying out roof construction works, including in particular making of openings for the designed skylights, which at the same time constitute openings in the existing roof layers of the side aisles, the site manager should inform the project supervisor, together with an entry in the construction log, that the existing roof layers revealed in these test-holes differ significantly from the solutions adopted in the design documentation. This omission on the part of the Construction Works Contractor constituted a material removable workmanship defect. The additional specific weights of these roof layers were as follows:

- side aisles: 1.94 kN/m^2,
- central aisle: 2.08 kN/m$^{2.}$

The above neglect resulted in the roof structure having to be cleared of snow when the snow cover was below the normative value [11]. The facility manager agreed to a proposal to remove redundant roof layers [11] and carry out thermal retrofitting on a structure subject to a lighter load. This made it possible to achieve a load-bearing value that meets the requirements of today's snow load standard and, in addition, to safely install the photovoltaic panels.

When upgrading roofs of buildings to meet modern thermal insulation requirements, as well as snow load and often loads associated with additional equipment (e.g. air-conditioning units, photovoltaic panels, ventilation, etc.), particular care has to be exercised when determining the current load-bearing capacity. The procedure for verifying it should start with an analysis of the available technical and maintenance documentation for the building in question. It is then necessary to compare the information in the documentation about the structure with its actual state. Geometrical measurements should be taken first, followed by non-destructive testing of the embedded materials (e.g. scanning of reinforcement, sclerometric measurements of concrete strength, etc.). If there are doubts about the structural layout, it is necessary to perform test-holes at structural elements. If there is any doubt about the strength class of the installed materials, an assessment of their mechanical properties should be carried out by means of laboratory tests. The final element of the procedure should be a verifiable static-strength analysis of the structure, based on an accurate drawing of the structure and the parameters of the materials incorporated.

An important element in view of possible additional loads is to check the current layout of the roof layers with an analysis of their thickness and volumetric weight. This also applies to the renovation or reconstruction of inter-storey floor and ceiling assemblies. Often additional levelling layers are added during use and flooring changes, which can lead to overloading of the floor and ceiling assembly [1].

5. CONCLUSIONS

With climate change, the hazard of extreme weather events is increasing. It is therefore extremely important to apply the standard provisions correctly to newly designed buildings. Using a correct computational model when designing a structure is another essential factor of great importance. The execution stage is no less important, and any lapses can lead to a tragedy. During building use, particular attention must be paid to carrying out inspections and checks with the right frequency. Any faults or failures in the structure should be comprehensively analysed, including 'in situ' and laboratory tests and static-strength calculations. In the event of a change in the loads acting on the structure, or a change in the use of the stricture, a detailed check of the condition of the building and the parameters of the materials used is recommended, especially for historic buildings [20] or structures which suffered damage as a result of a construction disaster [21]. This minimises the risk of failure or, ultimately, of construction disasters.

Provisions regarding liability for a disaster are set out, inter alia, in the Criminal Code [19]. According to the aforementioned Act, a person who brings about an event endangering the life or health of many persons or property of great magnitude, taking the form, inter alia, of a collapse of a structure, is liable to imprisonment for a term of one to ten years [19].

Unfortunately, there are cases where, after signs of structural malfunction, the building surveyor allows the building to continue to be used with no restrictions without carrying out a detailed analysis. This is exemplified by the aforementioned disaster in Katowice described in [7], as well as the case of the hall in Święcice described in [17]. In the first case, the expert faced criminally liability. In the second case the expert was not found to be criminally liable – presumably due to the lack of injured parties.

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