Analysis of load bearing structure of selected hall buildings

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Abstract. In this paper the issues associated with correct design of hall buildings has been presented. Large span roof structures require a particularly careful approach to design issues. In this paper examples of two building hall were presented. In the first example, the construction disaster occurred, while in the second of presented buildings due to the changes introduced while construction has been protected against such disaster. The first of the analyzed cause concerns storage hall, whose roof structure has been destroyed due to heavy rainfall. The main cause of this disaster was the malfunction of vacuum roof drainage system compounded by a number of design errors. Mentioned errors were not eliminated, during construction, despite the additional investor supervision, which was independent of parties involved in a construction process. The second case concerns the structure of a sports hall, which was built next to the existing school. Under construction of the hall building, the contractor and the supervision inspector, in conjunction with the designer, introduced a number of modifications of the structure to prevent the disaster. These modifications were a direct result of errors at the design stage, which were eliminated under the construction works. Based on analyzed cases, the scope of diagnostics, which should be performed during the operation of hall buildings with large roof areas, was determined.

1 Introduction

Design errors constitute a relatively small percentage of the causes of building collapses [1]. Nevertheless, these disasters often are tragic in terms of both material damage and the number of victims. An example could be a collapse in the International Katowice Fair Centre, which took place in January 2006 [2]. Due to the design errors, construction errors and errors in the maintenance of the facility, 65 people were killed and over 170 people were injured. Due to the structural reliability and safety of use, buildings with large roof surfaces require special care both at the design and operation stages. In this paper examples of two hall buildings were presented. The first of them is storage hall with a mixed structure, with a preponderance of steel elements. The roof was built as steel structure. The load – bearing structure of the analyzed part of the roof covering was made of a cold-formed Z-section purlin beams and perpendicular plate girders. Supports of the bearing structure of the hall roof were built as a steel and reinforced concrete columns and masonry walls strengthened with reinforced

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concrete skeleton. Due to the reduced stiffness of one of the roof girders, caused by the lack of internal support, the assumptions about the flat roof were not met. Vacuum roof drainage system did not meet the design assumption, which has been compounded by a number of design errors and unsuitable construction of the installation. The second of presented facilities is sport hall of a reinforced concrete structure with glued-laminated timber girders [3]. During the construction works, the following changes of the building permit design have been introduced:

- the change in the assembly method of timber girders with a span of 27.78 m by inserting a steel support fixed to the reinforced concrete column,
- the change in the strength class of steel tie rods and turnbuckles,
- the change of the fixing method of steel tie rods to reinforced concrete columns construction,
- the change of the strength class of wood from which roof girders were made.

Therefore, supervision inspector ordered a survey in terms of their compliance with the approved construction project and the performance of a technical assessment, which determine the correctness of their performance.

2 Description of hall building structure

2.1 Structure of a storage hall

The first of described objects is a single-storey hall building with a steel roof structure. In the Figure 1 a schematic view of the plan and cross section of the hall building has been presented. The part of the covering, that has collapsed, was analyzed in detail. This is the area located between the axes 1, 4, C and E. The gable wall along the axis 1 is built as masonry wall with reinforcing cage. Similarly, a wall along axis 4 between the axes D and E is made. The rigid frame structure of the external wall, in axis 7, is characteristic for gable wall in steel hall building. Steel columns, located at the intersections of structural axes, are internal point supports of the roof support structure. The columns do not occur only at the points of intersection of the D axes with the axes 2 and 3. The lack of these columns, caused by the utility requirements, became a source of strong irregularity in the roof construction system, that contributed by other factors led to a construction collapse. In the plane of the external wall in the E axis at the intersections with the axes 2 ÷ 6, reinforced concrete columns were constructed.

The main load - bearing elements of the roof structure along the axes 2, 3, 5, 6 and partly along the axis 4, between the axes A and D, are plate girders with different cross-sections. Cross – sections of plate girders in axes 2 and 3 in the spans between C and E axes consist of flanges with a cross-section of 300 × 35 mm and a web with a section of 830 × 18 mm. Cross sections of the remaining plate girders consist of flanges with a cross-section of 300 × 16 mm and a web with a section of 668 × 8 mm. Cross-section plate girders are hybrid. The booms are constructed with St4S steel, while the web with 08X steel. Despite the increased cross-section, the girders in axes 2 and 3 have reduced stiffness (EI / L) in the spans between the C and E axes, due to the lack of supporting columns in the D axis. In addition, due to erroneous calculations, the designer of the hall building structure underestimated the total design loads by about 10%. It was demonstrated that the structure of girders in spans with a length of 32.85 m did not meet the requirements of the serviceability limit state, which partly affected the initiation and development of roof failure.

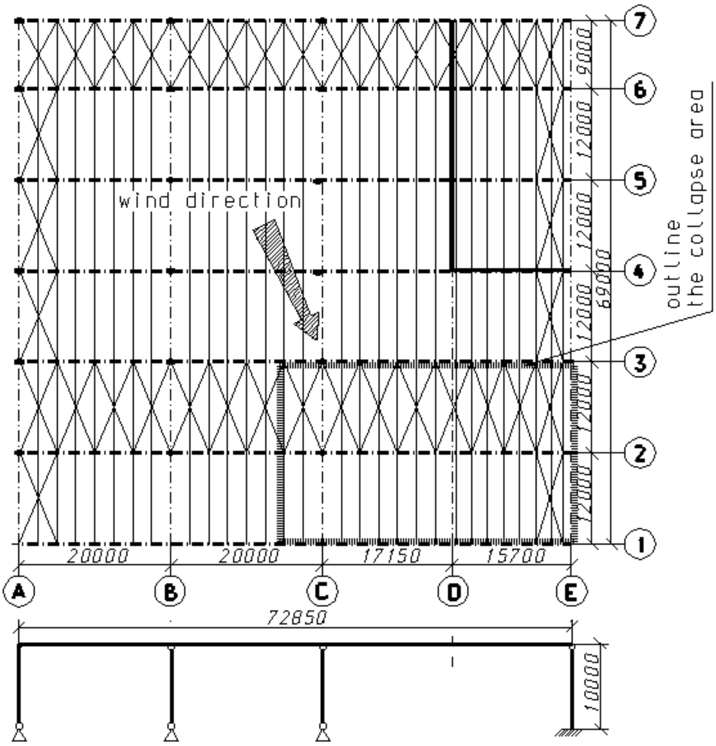

Fig. 1. Scheme of roof structure of hall building

In the central part of the roof, where the failure occurred, under the influence of permanent loads a basin was formed, in which rainwater has been accumulated, due to the disability of the vacuum roof drainage system. The supporting structure of the roof covering is the roof purlins system (parallel to the A, B, C, D and E axis) with Z300 × 95 × 87 × 3 cold-formed Z-section made of St4S steel grade. Spacing of the roof purlins is varied, with a dominant span of 2.5 m. In the roof purlins, within the analyzed area of the roof, all externally located supports should be treated as rigid in terms of vertical displacements. Internal supports that were constructed as girders are flexible supports. Due to the lack of internal columns in the D axis, the support in axes 2 and 3 in the central part of the area contained between the axes C and E, is particularly semi rigid. The results of numerical calculations had shown, that omission of significant deformability of girders, resulted in lowering of the calculated values of stresses in purlin beams by up to 30%. In the global analysis, static schemes of continuous purlin beams with fixed support, were assumed. It was compounded by replacement of steel grade of purlin beams by the contractor. Steel with a yield point of 250 MPa instead of the designed 320 MPa was used. Theoretically, the roof purlins of cold-formed Z-section should be articulate based on plate girders with the possibility of rotation in the vertical plane in a structurally acceptable range [4]. Due to the rigid connection of the girder with roof purlins, the range of their rotation over the supports has been limited to the minimum, which result from the torsional susceptibility of the roof girders, Fig. 2. Significant differences between the deflections of next two spans of the roof purlin had to create torsion plate girder.

Fig. 2. Connection of the roof purlin with the girder (after failure)

Roof braces were designed in the form of an X-type grating constructed of round bars with a diameter of 22 mm made of St3S steel. Due to the lack of pre-tension, in computational terms, it is a N-type grating. The fixing of the ends of bracing tendons at the theoretical grid nodes, was made using the deformable bracket connectors, that aim to support the roof purlins on the girders of the upper booms or at the ring beam of the masonry gable wall. As bracket profiles, C180 hot rolled steel channel shapes were used. The connection of roof purlins and gusset plates with the bracket was made on four M20 bolts of 8.8 classes. The crossing of intersecting bracing tendons was faulty constructed. Due to the location of the tendons, in the middle of the height of the roof purlins' profile, the tendons were divided into two parts and attached to the web of the roof purlin by gusset plate. This connection does not ensure axial transmission of the tensile force from one part of the tendon to the other which causes additional bending of the roof purlins in a horizontal plane, Fig. 3. Furthermore, the deflection of the roof purlin causes an automatic increase of the tensile force in the tie rods and thus the torsional rotation of the girder. Sloping roof braces, instead of stabilize, cause destabilization the main girders, due to their faulty construction [5].

Fig. 3. Damaged bracing elements

2.2 Structure of a sport hall

The first of the assessed elements of the sports hall structure were roof timber girders [3]. According to the building permit design, the structural material of the girders was GL30 glued-laminated timber, while the GL32h class timber was used. Fixing the girder to the reinforced concrete structure of the hall building was designed as eight M16 bolts of 8.8 classes, which pass through the girder and gusset of a thickness of 12 mm and were mounted in reinforced concrete columns. This fixing was additionally changed while constructing steel support. This solution applies to both the mounting in the 10 axis as well as the mounting in the axis no 5, Fig. 4.

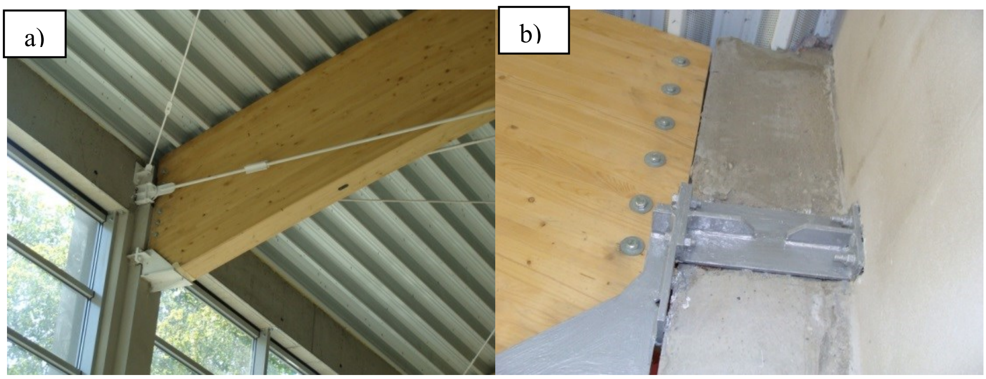

Fig. 4. The method of attaching the timber roof girder a) to the reinforced concrete column in the axis 10, b) to the reinforced concrete column in the axis 5

The layout of fixing of the girders tie rod has been completely changed. Originally in the axis no. 5, the fastening was designed as a butt plate with four HAS-E M16x125/20 dowel bars HIT HY 150, the tie rod fastening on the opposite side was designed in the form of a steel structure mounted in a timber girder within 2852 mm of the axis 10. Mentioned structure was mounted in a timber girder using four M16 grade 8.8 bolts. The whole structure of the tie rod and its fasteners was designed of St3S steel. In fact, the fastening both on the axis 5 and axis 10 sides were made as steel clamps of reinforced concrete columns, where 18G2 steel was used, Fig. 5 [3].

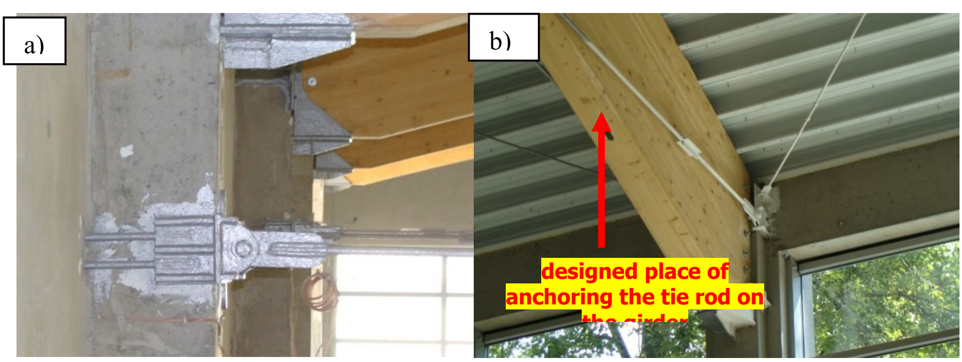

Fig. 5. Fixing the tie rod of laminated timber roof girders, a) in axis 5 b) in axis 10

Another element of which the design solution has been changed were the columns that support the steel beam girder, these elements together form a frame to support the structure over the fire road. The following cross-sections of square column of 35 cm x 35 cm, reinforced with six rods ϕ 32 and two rods ϕ 25, thickness of the cover 25 mm, has been designed. According to the construction site book, the supervision inspector ordered to check the structure of these columns due to their damage that appears. All works in this area were stopped and additional structural support was applied. According to the construction site book, the construction of columns has been overloaded, therefore the designer recommended the dismantling both of the structure, pillars and their foundations [3]. The construction manager received the replacement design documentation two months later, although the designer undertook to deliver it within four days. The main changes in the replacement design documentation have included:

- changes in the depth of column foundation from 1.4 m to 2.42 m below ground level,
- changes in the cross-section of the column in axes A and 10 to a rectangular column of 35 cm x 65 cm reinforced (in the bottom part) with eight rods Ø 25, thickness of the cover - 3,0 cm,
- changes in the cross-section of the column in the axis A between the axes no. 7 and 8 to a square column 40 cm x 40 cm reinforced with eight Ø 25 rods, a thickness of the cover - 3.0 cm.

After the construction of columns in accordance with the replacement design documentation, no damage occurred. The change of the foundation depth of the columns was required due to the presence of the basement walls of the existing building in direct contiguousness with the foundation. The construction of foundations at -1.40 m.p. would threaten the safety of the structure of both the newly built sports hall and the existing building.

Steel girder was designed as a double-span beam with a span of 5.1 m and 4.8 m in the ceiling above the floor in the B axis between the axes no. 6 and 8 with a steel section HEB 240. Supporting of the binding girder on two supports was designed as a steel wall studs. On the third support, the support was designed as a butt fastening to the reinforced concrete binding joist. Originally, they were designed of a gusset plate joined to a steel binding joist using three M16 bolts, grade 5.8, and a reinforced concrete binding joist using HIT HY 150 HAS-E M16x125 / 38 adhesive anchors - in axis no. 8. The support proved to be ineffective, despite moving the partition wall under the binding joist and replacing it with the load-bearing wall (construction designer's entry in the construction site book was classified as non-significant change), the investor's inspector additionally recommended to the structural designer to check the bolted connection and suggested additional support in form of the wall stud. The designer agreed to the suggestion of the investor's supervision inspector by proposing an additional support in the form of a reinforced concrete bolt under the connection, Fig. 6.

Another questionable element that required structural changes as well, was the steel binding joist in the first floor slab in the axes 6 and 7 between the axes A and B. According to the record in the site book, the construction manager had stated excessive (visible to the naked eye) deflection of this member. Therefore, he asked the designer to check this element. The inspector of investment supervision stopped all construction works in this area and also asked the designer to check the structure. Due to all of construction elements damaged during the construction of the object, the inspector of the investor's supervision stopped the acceptance of the shell building state and recommended to verify of the structural design by independent structural designers. A detailed expert opinion based on the verification of project documentation showed the number of irregularities and errors in the design documentation.

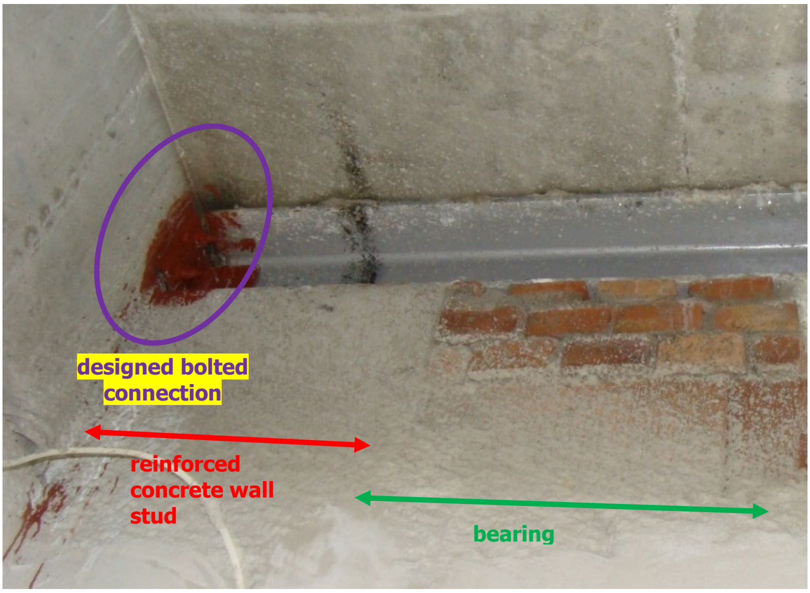

Fig. 6. Additional support of the binding joist in the form of a reinforced concrete wall stud

3 Discussion

3.1 Structure of a storage hall

The collapse of the steel supporting structure of the hall building roof consisted in the collapse of a part of the roof slope after the loss of bearing capacity on the warping of the girder in the axis 2 above the column at the intersection with the C axis, Fig. 1.

The failure of the hall building roof occurred during heavy rainfall, compounded by a strong wind, that caused the flow of the rainwater from the large area of the roof towards the area that has collapsed afterwards, Fig. 1. The disaster was caused by the load associated with the weight of rainwater lying in the lake's basin created due to excessive local deflection of the roof. Assessment of the wind direction, and thus the direction of rainwater flow, variability of wind speed and increase of rainfall intensity during the failure, was possible due to the image recorded by the industrial camera.

The direct cause of the loss of bearing capacity for warping of the roof girder, was the destruction of the most stressed purlin beams by the loss of local and distortion stability and local plasticization of the roof girder walls.

Water deposition was caused by the disability of the vacuum drainage installation, which was caused by incorrect design assumptions and careless accomplishment. The designer of the drainage installation at the stage of initial design had assumed that the water distribution into roof drain, which was regularly spaced on the roof, would be uniformly. The shape of the roof, which was formed as a result of the permanent load associated with the weight of the roof, prevented the execution of this assumption. The water from the surface area of approximately 900 m^2 was drained into a roof drain placed in the centre of the critical area, while in case of the neighbouring roof drains it was a runoff from the surface area of at most 70 m^2. Under these conditions, the vacuum pressure drainage installation could operate only as gravity roof drainage [6]. In addition, the narrowing of pipes due to faulty joining of segments and the use of pipe fittings of a reduced diameter has been found.

In a detailed computational hydraulic analysis, supported by the results of meteorological observations, it was shown that after half-hour rainfall with a maximum intensity of 40 mm/h, on the critical part of the roof was about 16.5 thousand litres of water, that is the load corresponding to the load in a shape similar to a spherical cap with a base radius of a = 9.0 m and a depth of h = 13.0 cm. Mentioned shape of the basin approximates adequately the shape of the roof deformation. The maximum deflection of the roof, which was calculated, amounted to about 18 cm at the moment of failure, which indicate the depth of the basin h = 13.0 cm. It should be noted that the rainfall intensity of 40 mm/h = 111 dm^3 / (s × ha) is only slightly more than 30% of the standard rainfall intensity, which is 300 dm^3/(s × ha).

It was possible to prevent this collapse while roof covering leakproofness test. According to the site manager's account, during the water flooding the roof began to excessively deflect, the overflow openings located around the perimeter of the building at the highest points of the roof, did not work. Therefore, the leakproofness test was stopped and no repair works were undertaken.

3.2 Structure of a sport hall

Considering design construction errors revealed during the construction of the building, which resulted in excessive deformation and damage to some structural elements, the investor ordered to perform additional expert opinion. The purpose of this opinion was to verify the correctness of design solutions. It should be emphasized that the derogations from the approved building permit design, which were introduced during the construction of the building, had protected the sport hall against threat of building collapse.

The designer provided a technical opinion, which contained many errors and inaccuracies, an example of which can be the stresses in the tie rod, calculated at 405.56 MPa, which considerably exceed the limit values in structural steel. The rigid support, that has been implemented by the author of the opinion, on the calculation scheme of the load-bearing structure, also casts doubts. In fact, in the place of this support, there is a ring beam, that support the channel floor slabs. Verification of the supporting structure of the sport hall had confirmed that the damages occurred during construction were the result of significant removable errors in the construction design, which from the point of view of the construction law, did not constitute significant deviations from the approved building permit design.

4 Conclusions

Every engineering structure should ensure safety during its use, for this reason, the correct dimensions of the building and the materials used in construction should be determined at the design stage. Correct building design guarantees the durability of the facility and the transmission of the designed external forces system, i.e. the loads [7].

The estimation of loads at the design stage is not difficult and the difference with the occurring loads after the completion of the object is insignificant, unlike to the bearing capacity of the object, which may differ significantly from the assumed values.

It consists of many factors, such as:

- incorrect model of structure behaviour,
- errors and negligence during construction,
- lack of proper construction supervision,
- use of unsuitable building materials.

The last three of these factors can be eliminated by properly conducted construction supervision, while making of an appropriate model of structure that corresponds to its actual behaviour, can be problematic [7].

In the case of storage hall, the effect of simplifying the model of purlin beams was already visible during the construction of the building. The critical area of the roof of the hall, which after three years of operation has collapsed, even during the roof covering leakproofness test has deflect over the compounded value. The site manager ignored this issue. Ineffective drainage installation and design errors result in a construction disaster.

In the case of the presented sports hall, design errors were also revealed during the construction of the building. Structural changes introduced during the construction did not constitute significant deviations from the approved construction project, these changes were applied by the alternative design documentation and the construction works did not concern any of the relevant changes, from the point of view of the construction law. Introduced corrections did not change the characteristic parameters of the sport hall, such as: cubature, building area, height or length. The planned way of using of the building structure or its part has not changed.

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