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Research paper

Drainage of flat roofs in large–area facilities

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Abstract: The large-area facilities are mostly based on the same materials and construction technologies. However, the larger the roof area, the more difficult it is to ensure the proper functioning of the emergency roof drainage system, which is most often implemented by means of emergency roof overflows. The problem is confirmed by at least several construction disasters caused by sudden and intense rainfall that have occurred in the last 5 years. The paper refers to the method of calculating the design rainfall intensity, as well as the definition of emergency roof drainage available in the literature. The effect of deflection of the structure from individual load cases, including standard loads, as well as from rainwater load in several variants, depending on the assumed snow load zone, is presented on the basis of a sample structure. The calculations show that structures with variable spans are particularly susceptible to the accumulation of rainwater. Due to the complexity of the phenomenon, the author does not provide a clear solution to the problem, but recommends the use of a double pipe emergency roof drainage system in such structures, rather than relying solely on roof overflows.

Keywords: drainage, failure, flat, hall, large–scale, roof

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1. Introduction

Large–scale hall buildings nowadays often reach areas of several hectares or more. They are most often constructed as objects with a usable height of 10 meters, with a structural grid of 12×24 meters and 24×24 meters in the docking zone. The structure of the building consists of a system of prefabricated reinforced concrete or steel columns with a roof structure based on them in the form of trussed steel purlins spaced at 6–meter intervals supported on steel, lattice or prestressed concrete chords. A high–profile trapezoidal metal sheet is usually attached to the top chord of the truss purlins, on which rests thermal insulation made of mineral wool or polyisocyanurate (PIR) panels insulated from above with a welded roofing membrane.

The roof is most often made as a multi–slope with a slight slope to allow water to flow freely down the membrane, i.e. around 2% (or about 1.2°). Roof drainage in a large proportion of these types of buildings is made as a syphonic system of high-density polyethylene (HDPE) welded pipes with roof drains located at the lowest points of the roof slope. This type of drainage works best when rainwater covers the outlets, i.e. reaches a height of 4–5 cm in the roof lowest level. The use of this type of drainage requires the construction of attics around the perimeter of the building, which are most often made as low as possible, i.e. not higher than the roof ridge by more than 15 cm.

'Containment' of rainwater in the roof area (inside the building outline) raises legitimate concerns about how the building's roof will be drained when the primary drainage system fails. The most common method assumed already at the design stage is the use of emergency overflows in the attics of external walls. These are usually rectangular openings located about 5 cm above the roof covering at its lowest points (as seen around the perimeter of the building). Their principle of operation is that when rainwater reaches the level of the emergency overflows (i.e. when the rainwater level continues to rise despite the operation of the drainage system) it will overflow through them. An alternative method, which is far less common, is an independent roof drainage system (gravity or pressure) which, like the basic roof drainage system, works via roof drains located at the lowest points of the roof slope. The difference is that the outlets of the emergency installation are located a few centimeters above the roof slope and start working when the basic installation is overflowing or not functioning properly. These roof drainage systems are widely used in European countries such as Germany, the Netherlands and the Czech Republic. In the Polish (and at least some European) normative system, there are no strict guidelines for the design of the emergency syphonic drainage systems. Recommendations from manufacturers and the provisions of the now withdrawn Polish standard for sewerage systems in general [1], the standard for the design of gravity drainage systems [2] or non–standard sources are most often used [3, 4]. There is also no reference to the interaction of the amount of rainwater with wind gusts or the operation of the structure, which, in the author's opinion, may be of significant importance for large–scale buildings in particular and this will be the subject of subsequent chapters.

2. Rainwater limit

As there is no legally binding Polish standard or guideline for the adoption of a limit for the amount of rain for which a primary and emergency drainage system should be designed. Differences in approach can be found among individual designers. The vast majority of roof drainage systems are designed for a driving rainfall of 300 l/(s·ha), which appears in a withdrawn Polish standard from 1992 [1] and is defined as rainfall with a frequency of 5 years and a duration of 5 minutes. According to the recommendations of DIN 1986–100 [5], which is often referred to for emergency drainage system calculations, the amount of rainfall intensity should be taken as the difference between the rainfall intensity with a frequency of exceedance of 100 years and a duration of 5 minutes and that taken into account during the calculations of the basic drainage system. For about 20 years, the so–called 'IMGW formula' by Bogdanowicz – Stachy [3] has been used in Poland to calculate the amount of representative rainfall. According to the Eq. (2.1) – for instance the region of north–western Poland – rainfall with a frequency of 100 years and a duration of 5 minutes will reach a maximum height of the water column 15.5 mm:

$$P_{(c,t)} = 1.42 \cdot t^{0.33} + (3.92 \cdot \ln(t+1) - 1.662) \cdot (-\ln \frac{1}{c})^{0.584} \tag{2.1}$$

where:

t – duration of steady rain in minutes,

c – frequency of excess rainfall in years.

In view of the Eq. (2.1), the intensity of the 100–year steady rain with a duration of 5 minutes will be as in Eq. (2.2) below:

$$q_{(100,5)} = \frac{P_{(100;5)}}{5\ \text{min}} = 517 \frac{\text{dm}^3}{\text{s} \cdot \text{ha}} \tag{2.2}$$

By carrying out analogous comparative calculations against the intensity of the steady rain given by the standard [1], we obtain water column heights and intensities equal to respectively 9.49 mm and 316 $\text{dm}^3/(\text{s} \cdot \text{ha})$.

The calculated value for the driving rain intensity is close to that contained in the withdrawn Polish standard [1] and it is this value that the author will rely on in the following work.

In view of this – again in accordance with the guidelines of DIN 1986–100 [5] – the difference in rain intensity must be taken for the purpose of calculating the emergency drainage system ass in following Eq. (2.3):

$$q_A = q_{(100;5)} - q_{(5;5)} = 517 - 316 = 201 \frac{\text{dm}^3}{\text{s} \cdot \text{ha}} \tag{2.3}$$

According to this standard – if the building is classified as requiring special protection (there is no precise information what object it is) – then the emergency drainage must be designed for the full $q_{(100;5)}$ intensity. It should therefore be noted that if the building is not in need of special protection, it is assumed that the emergency drainage system is not intended to replace the main roof drainageFig system, but only to drain the amount of water resulting from abnormal rainfall, and that the main roof drainage system itself must always remain functional. It is also important to note that both the outlets of the emergency roof drainage system and the attic emergency overflows do not allow all the water to be drained from the roof, but only the amount that exceeds the level of their fixing (usually about 5 cm). In the event of a complete failure of the primary roof drainage system and once the emergency overflow system has been activated, a layer of water will still remain on the roof with a height dependent on the elevation of the emergency drainage system components. The previously calculated rain height $P_{(100;5)}$ refers to the amount of water that will collect on a perfectly flat surface. Assuming the most common roof geometry with a slight slope of 2‰ in large–scale buildings (Fig. 1), it is possible to determine what the equivalent height of rainwater h will be in the valley of the individual slopes from Eq. (2.4), assuming ridge spread of 24 metres (basin width), infinite stiffness of the roof slopes and no wind.

$$P_{(100;5)} = \frac{1}{2} \cdot 2 \cdot \frac{h}{2‰} \cdot h/24000 \rightarrow h = \sqrt{P_{(100;5)} \cdot 24000 \cdot 2‰} = 86.3\,\text{mm} \tag{2.4}$$

This means that the water level in the roof slope valleys will far exceed the level at which the emergency roof drainage system – whether made as additional pipework or as attic overflows – starts to operate, assuming that the primary roof drainage system is not working.

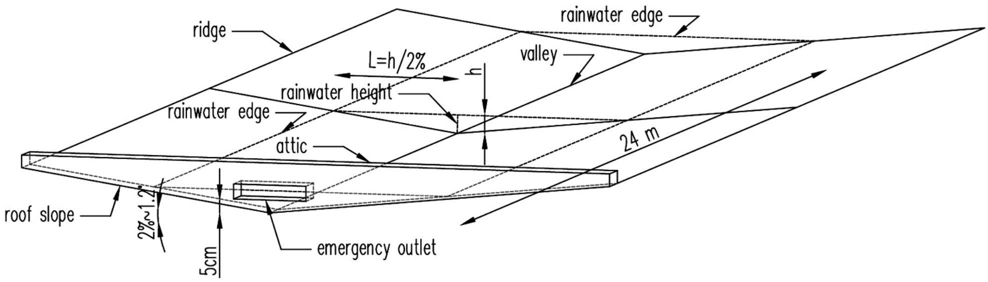

Fig. 1. Geometry of a typical flat roof's last span with rainwater height equal to h

The number of weather extremes is constantly increasing [6]. This is often associated with climate change, which is observed as a rise in temperature [7]. The Blaszczyk model, on the basis of which some people still determine the intensity of precipitation, was developed on the basis of data from 1837 to 1891 and 1914 to 1925. The model currently most frequently used (and yielding higher rainfall intensities than the one mentioned earlier), i.e. Bogdanowicz–Stachy, was based on data from 1960 to 1990. In light of the aforementioned considerations, it is evident that the models in question are based on measurements taken up to a century ago. Given the observed increase in the frequency of extreme weather events, it is possible that these models may not provide an accurate representation of the rainfall intensity assumed for the design of emergency drainage systems.

3. Deflection of structure

The current Polish standards for the design of structures do not specify the height of the water column to be carried by the structure or how to apply these loads. The load from rainwater has a different specificity from the standardised commonly used permanent and live loads [8], from snow [9] or wind [10]. The level of the water table evens out at the top and fills any irregularities or basins below it, while rainwater primarily fills the areas at the lowest level.

According to the applicable standard for steel constructions (because this is the roof construction prevailing in large–scale indoor buildings) [11], the deflection limit for truss girders is $L/250$ (where L is the span of the element). It is important to note here that this deflection limit relates to the member and does not take into account the deflection of the element on which the beam or truss may lay (it is not the vertical displacement of a point on the member, but the deflection of the element measured from the chord). For roof trusses with a span of 24 metres, this gives an allowable deflection value of 96 mm.

According to the standard [12], this deflection value is assigned to a quasi–permanent load combination due to the effect of the deflection on the appearance of the building and user comfort. For the quasi–permanent load combination for the Serviceability Limit State for roofs of buildings less than 1000 m above sea level, only the self–weight of the structure and the permanent loads need to be considered (in December 2004 there was an amendment to the original edition of the standard, which changed the simultaneity factor ψ_2 for snow from 0.20 to 0.00). For the frequent combination of actions, in addition to the self–weight and permanent loads, 20% of the basic snow load must be taken into account, and for the characteristic combination (i.e. used for irreversible limit states) 60% of the wind load and 100% of the snow load for typical design situations. Leaving aside the standard approvals, the vast majority of steelwork designers check the more unfavorable deflection condition for the SLS (above standard) by using the frequent combination (instead of the quasi–constant combination), because it includes at least some of the basic snow load, which in Poland is usually the dominant load case for roofs. It follows, however, that the standard allows a deflection of roof trusses or beams with a span of 24 metres of up to 96 mm from permanent loads and dead weight alone, with a more conservative but frequently used approach, with an additional contribution of only 20% of the snow load.

In order to demonstrate the effect of deflection on rainwater accumulation, calculations were carried out based on the structure of an existing hall in Poland (Fig. 2). Based on the assumed limit and known from the as-built documentation that the elongation of the main steel structural elements (trusses and purlins) is between 80–99%, it was assumed that this hall is located in each of the five snow load zones.

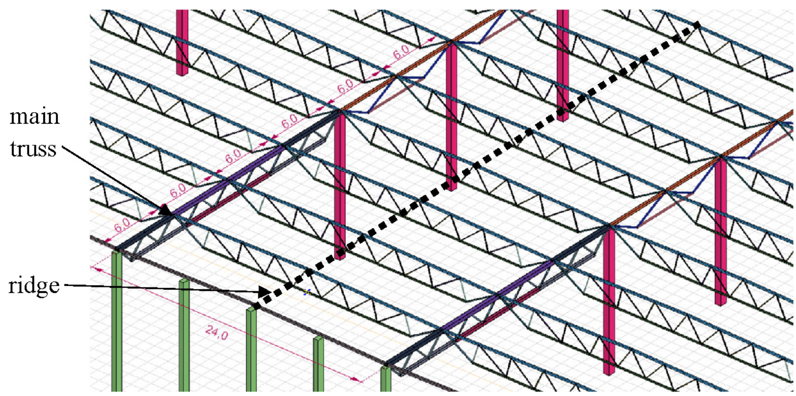

Fig. 2. Simplified geometry of main steel structure of roof implemented in calculation model

The profiles of the main steel members were appropriately enlarged or reduced using the original types of cross-sectional profiles and the adopted rules for the design of steel members. The roof structure was subjected to gravity loads, a constant of 0.53 kPa, external and internal wind pressures for wind load zone 1 (for snow load zones 1, 2, 3 and 4) and snow load zone 3 (for snow load zone 5) and site category II and snow load in 5 areas (in basic value for buildings located below 300 meters above sea level).

In the case of trusses with a span of 24 metres (Fig. 3), it was assumed that the top chord consists of HEB sections arranged with the web in the horizontal plane and that there is a variation in section height along the length of the girder. It was assumed that buckling and lateral torsional buckling could occur between lattice purlins resting on the beam every 6 meters, and that additional buckling limitation in the beam plane was provided by cross bracing. The lower chord was made of HEB sections with the web in the vertical plane and, as with the upper chord, the height of the section used varied along the length of the lower chord. The diagonals were made of closed square hot-rolled sections of varying cross-sections. The lattice girders, with an axial height of approximately 1.5 metres, were supported on reinforced concrete columns every 24 metres.

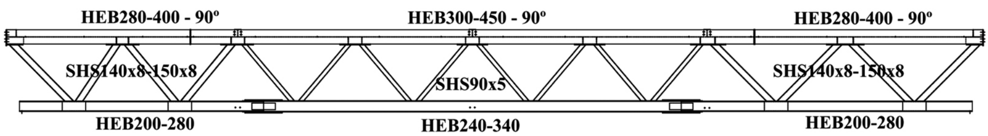

Fig. 3. Profiles used for truss girder with a span of 24 meters

For trusses with a span of 12 meters (Fig. 4), the top chord was assumed to consist of HEA sections arranged with the web in the vertical plane. It was assumed that buckling and lateral torsional buckling could occur between the truss purlins resting on the girder every 6 meters, with cross bracing as an additional constraint for buckling in the plane of the girder. The bottom chord and diagonals are hot rolled square closed sections. The axial height of the girder is approximately 1.5 metres.

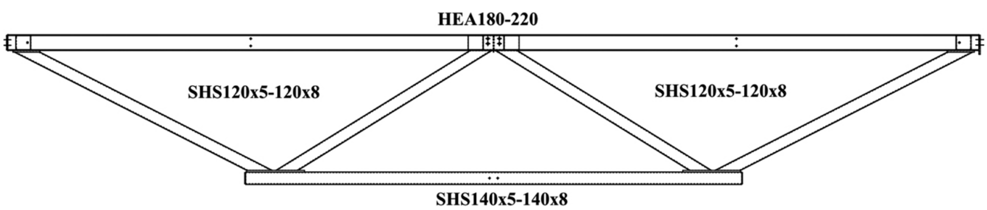

Fig. 4. Profiles used for truss girder with a span of 12 meters

The top and bottom chords of the truss purlins (Fig. 5) were constructed using HEA sections, arranged with the web in the vertical plane. It was assumed that a trapezoidal sheet would provide protection against buckling from the main plane of the truss and against lateral-torsional buckling for the top chord. Moreover, tubular section braces were employed, with bracing occurring at 6-metre intervals. The cross-bracing is composed of closed square hot-rolled profiles with varying cross-sections. The axial height of the truss purlins is approximately 1.6 to 1.8 meters.

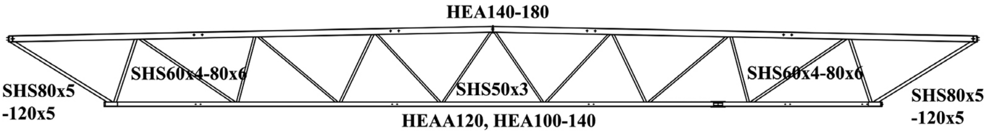

Fig. 5. Profiles used for truss purlin with a span of 24 meters

The calculations related to the design of the steel structure were carried out using the Axis VM program, version 7. Table 1 shows the results of the design of the bottom and top chords of the individual trusses, indicating the percentage use of the Ultimate Limit State (ULS) according to the Eurocode standard for each snow load zone.

Table 1. The main chord profiles of the trusses with the profiles varying according to the snow load

Snow zone 1			
	PD 24 m	PD 12 m	PK 24 m
top chord	HEB280/300/280 (94%)	HEA180 (99%)	HEA140 (88%)
bottom chord	HEB200/240/200 (81%)	SHS 120 × 5 (89%)	HEAA120 (89%)
Snow zone 2			
	PD 24 m	PD 12 m	PK 24 m
top chord	HEB300/320/300 (93%)	HEA200 (85%)	HEA140 (92%)
bottom chord	HEB200/240/200 (95%)	SHS 140 × 5 (83%)	HEA100 (83%)
Snow zone 3			
	PD 24 m	PD 12 m	PK 24 m
top chord	HEB320/340/320 (96%)	HEA200 (92%)	HEA160 (81%)
bottom chord	HEB220/280/220 (87%)	SHS 140 × 6 (80%)	HEA120 (82%)
Snow zone 4			
	PD 24m	PD 12 m	PK 24 m
top chord	HEB360/400/360 (95%)	HEA220 (82%)	HEA160 (94%)
bottom chord	HEB240/300/240 (88%)	SHS 140 × 6 (92%)	HEA140 (75%)
Snow zone 5			
	PD 24 m	PD 12 m	PK 24 m
top chord	HEB400/450/400 (96%)	HEA220 (90%)	HEA180 (89%)
bottom chord	HEB280/340/280 (97%)	SHS 140 × 8 (81%)	HEA140 (87%)

Figure 6 illustrates the correlation between deflections and snow load zones for truss girders and purlins subjected to quasi-constant load combinations, which are representative of permanent loads. The deflections for the purlins were calculated in conjunction with the deflection of the girder at the purlin support. Consequently, these values represent the maximum displacements observed in the structural element. It is noteworthy that, irrespective of the snow load zone, the deflections for 24-metre span are approximately four times greater than those corresponding to 12-metre spanned trusses. The deflections resulting from the application of permanent loads reach a maximum of 50 mm for the longest girders, which is the distance from the lowest point of the emergency overflow to the highest point of the roof slope.

Figure 7 illustrates the correlation between deflections and snow load zones for girders and truss purlins in accordance with the Serviceability Limit State (SLS) characteristic combination. As can be observed, modifying the snow load zone has a negligible impact on the deflection value of each truss. It is noteworthy that deflections for truss girders with a span of 24 meters reach approximately 95 mm, with the largest vertical displacement for a truss purlin exceeding 170 mm.

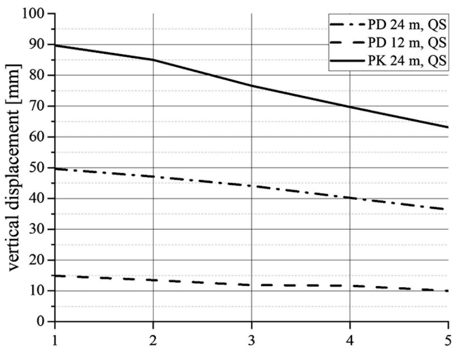

Fig. 6. The deflection of a 24 m span truss girder (PD 24 m), a 12 m span truss girder (PD 12 m) and a purlin (PK 24 m) for a quasi-steady combination (QS)

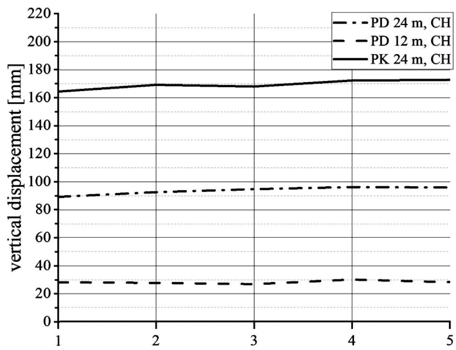

Fig. 7. The deflection of a 24 m span truss girder (PD 24 m), a 12 m span truss girder (PD 12 m) and a purlin (PK 24 m) for a characteristic combination (CH)

Subsequently, calculations were conducted to determine the deflections of the girders and truss purlins in response to rainfall of 50, 100, 150 and 200 mm above the lowest slope level. In accordance with equation 2.7, this equates to rainfall totals of 5.2, 20.8, 46.9 and 83.3 mm/m^2, respectively, for a catchment ridge distance of 24 meters and a slope gradient of 2%.

The results presented in Fig. 8, 9, and 10 demonstrate that the deflections for 24-meter span girders are approximately four times greater than those for 12-meter span trusses, reaching a maximum of 40 mm. The deflections for purlins are observed to be between 30 and 50% greater than those for 24-metre span truss girders. It is noteworthy that the deflections of the aforementioned elements decrease as the snow load increases. The reduction in deflection of a structure designed in the fifth snow load zone compared to one designed in the first snow load zone is up to approximately 30%.

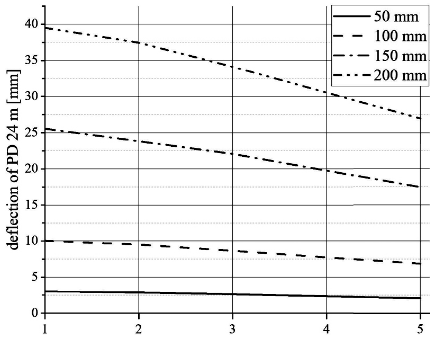

Fig. 8. The deflections of a truss PD 24 m in a function of the height of the water in the basin

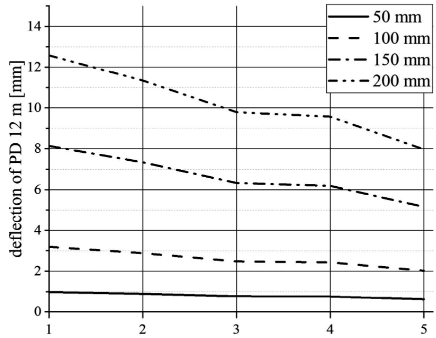

Fig. 9. The deflections of truss PD 12 m in a function of the height of the water in the basi

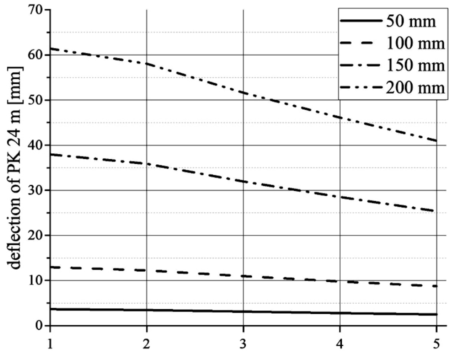

Fig. 10. The deflections of a truss purlin (PK 24 m) in a function of the height of the water in the basin

4. Capacity of emergency overflows

In order to determine the capacity of the emergency overflows, two schemes of their operation were assumed from the hydraulic point of view [13]:

– for rainwater heights not exceeding the level of the upper edge of the emergency overflow, it was classified as a non–submerged overflow with a sharp edge according to the formulas Eq. (4.1) and (4.2):

$$Q = \frac{2}{3} \cdot \mu \cdot b \cdot \sqrt{2 \cdot g} \cdot \left(H + \frac{v_0^2}{2 \cdot g} \right)^{1.5} \tag{4.1}$$

$$\mu = \left(0,6075 + \frac{0,0045}{H} - 0,045 \frac{B-b}{B} \right) \left[1 + 0,55 \left(\frac{H}{H+p} \right)^2 \right] \tag{4.2}$$

where:
μ – flow coefficient according to Bazin's formula with a component for lateral flow attenuation,

b – overflow width; assumed $b = 1.0$ m,
g – acceleration due to gravity; assumed $g = 9.81$ m/s^2,
H – height of water flowing through the overflow,
B – width of the water table in front of the overflow,
p – height of the sill; assumed $p = 0.05$ m,
v_0 – inflow velocity, i.e. the quotient of the discharge and the cross–sectional area in front of the overflow.

– for the height of the rainwater exceeding the level of the upper edge of the emergency overflow, it was classified as a large unlined hole according to the formula Eq. (4.3):

$$Q = \frac{2}{3} \cdot \mu_0 \cdot b \cdot \sqrt{2 \cdot g} \cdot \left[\left(H_1 + \frac{v_0^2}{2 \cdot g} \right)^{1.5} - \left(H_2 + \frac{v_0^2}{2 \cdot g} \right)^{1.5} \right] \tag{4.3}$$

where:
μ_0 – flow coefficient for a non–submerged borehole; conservatively assumed $\mu_0 = 0.6$,
H_1 – height of water counting from the bottom of the overflow,
H_2 – height of water counting from the top of the overflow.

According to the most common industry guidelines [14] – attic emergency overflows are recommended to be between 10–15 cm high located approximately 5 cm above the roof slope level – as shows Fig. 11. It is estimated that for every 25 cm^2 of emergency overflow area, there is a discharge of 1 dm^3/s.

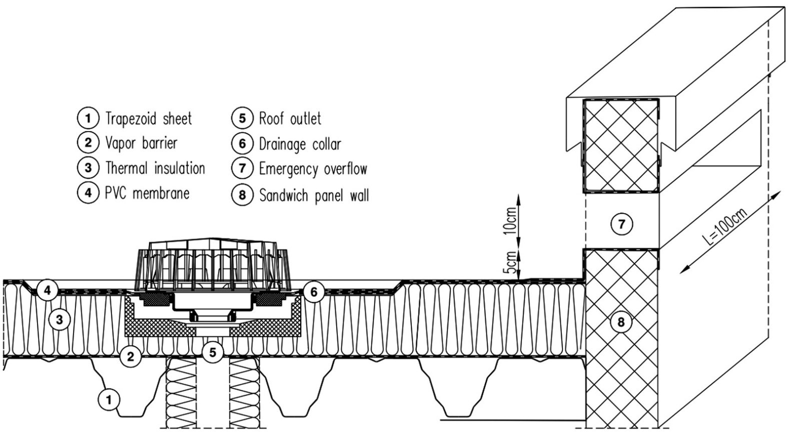

Fig. 11. Geometry of a typical syphonic roof outlet and attic emergency overflow

Based on the above information – for a sample basin area of 24 × 192 m and a rainfall intensity of 316 l/s/ha – it was estimated that emergency overflows with a total capacity of $V = 145.6$ dm^3/s and a surface area of 3640 cm^2 would be necessary. Therefore, 4 emergency overflows with a width of 100 cm, a height of 10 cm and an estimated capacity of 40 dm^3/s each were taken into consideration (usually these overflows are placed symmetrically on both sides of the roof, one per side of the basin with respect to the lowest point of the basin). For the emergency overflow geometry selected above, the dependence of the water table height on the roof slope level h [m] and the flow rate through the attic opening Q [dm^3/s] is shown below (Fig. 12). The dashed line indicates the capacity of the emergency overflow, which results from the estimated selection.

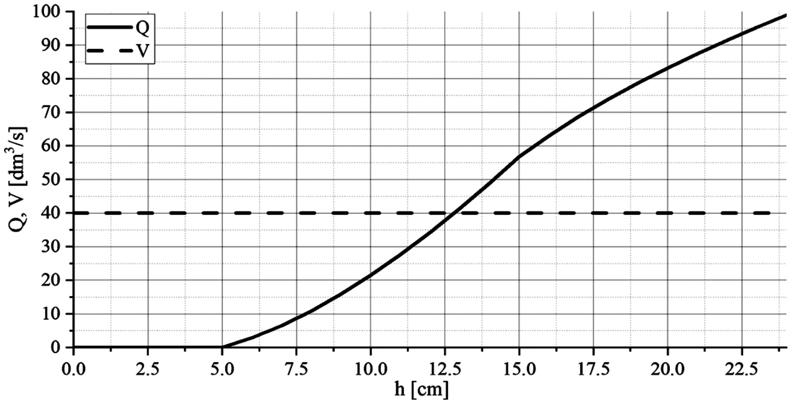

Fig. 12. The capacity of the emergency overflow from Fig. 13 in relation to water level over the roof

It can be seen from the above diagram (Fig. 12) that the estimated capacity of the emergency overflow will not be reached until the water level reaches more than 13 cm above the roof valley level, i.e. when the water level in the emergency overflow exceeds about 80% of its height. This additional load will result in the formation of a rainwater bag and further propagation of deflection before the capacity of the emergency overflows capable of removing normative rainfall water from the roof is reached (assuming the primary drainage system is not in operation).

Given that the deflection of the structure affects both the increase in load and the further propagation of deflection in the case of loading from rainwater, the load cases from the previous section were extended. Further cases were created, resulting from the deflection of the structure under the influence of rainwater and the subsequent filling of the resulting basins with the water (Fig. 13).

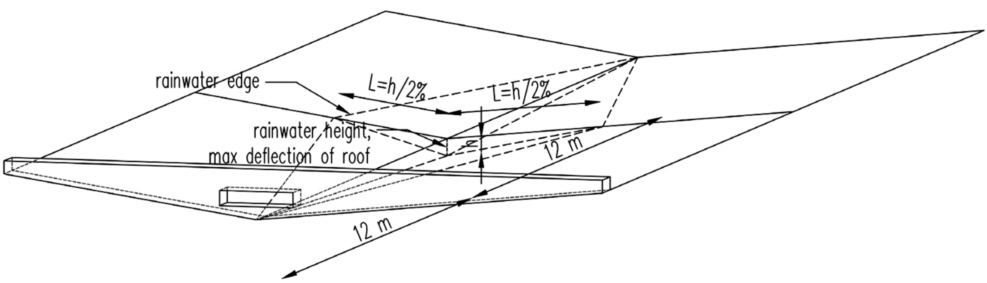

Fig. 13. The geometry of stagnant rainwater filling the deflection created in the roof structure adopted for the dock area calculations

While the loads resulting from the additional volume of water are not significant (the initial load deflection of the structure results in a maximum increase in load from rainwater of 2.5% – Fig. 14, 15 and 16), it is noteworthy that the increase in loads due to this for truss girders with a span of 24 meters is 5 to 10 times greater than that for truss girders with a span of 12 meters. As steel structures are designed in three dimensions and not four, with the duration of the load not taken into account for its effect on the structure, this cannot be directly translated into the volume of rainwater that the structure is able to absorb additionally due to deflection during the load. In the event of rainfall exceeding the capacity of the primary drainage system for a given catchment area, it is possible that emergency attic overflows may not function as intended.

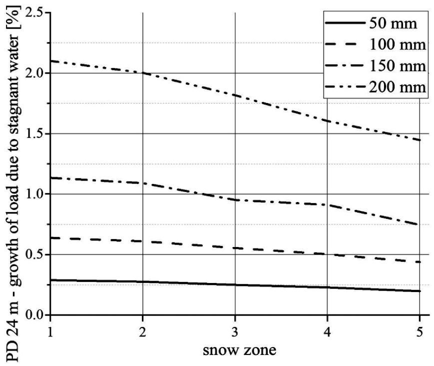

Fig. 14. Growth of water due to filling the deflected area of roof caused by rainwater for truss girder with 24 meter span in first iteration

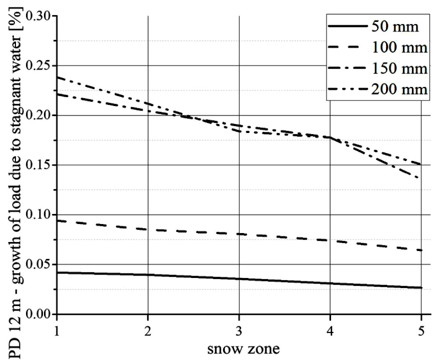

Fig. 15. Growth of water due to filling the deflected area of roof caused by rainwater for truss girder with 12 meter span in first iteration

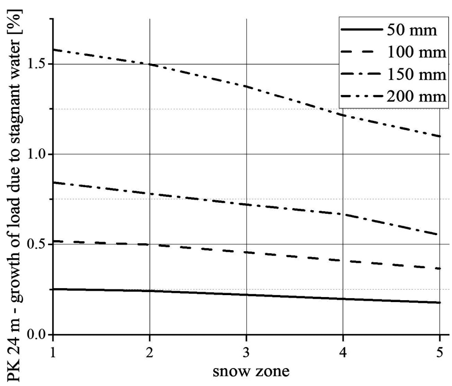

Fig. 16. Growth of water due to filling the deflected area of roof caused by rainwater for truss purlin with 24 meter span in first iteration

The structure in the aisles with a doubled span will yield to such an extent that the water column when resting on the exterior wall pillar will not exceed the height from the slope level to the bottom of the emergency attic overflow. This is despite the fact that the volume of water will cause the roof structure to exceed the load-bearing limit state. In light of the aforementioned considerations, it is this author's opinion that buildings exhibiting a variation in the angle of the nave span should be equipped with an emergency drainage system in the form of a duplicated piping system, with a capacity at least equal to that of the primary roof drainage system. It is imperative that the drains of this installation be placed at the points of greatest deflection of the structure, as this is the only way to ensure its proper functioning during extreme torrential rains.

The calculations in this chapter are for an optimistic situation, which assumes that during heavy rainfall there are no wind gusts capable of piling up rainwater causing an increase in the emergency overflow discharge on one side of the building's roof, and which in the case of a building with a significant surface area can be critical.

5. Conclusions

There are significant deficiencies in the standards relating to the emergency drainage of rainfall from roofs, which is particularly relevant for large halls with low slope roofs. The issue of design in accordance with the standards arises at the stage of determining the emergency rainfall rate for the roof drainage system. Most designers rely on German and Polish standards, which were officially withdrawn decades ago. They also refer to the recommendations of roof drainage system manufacturers. There are no valid Polish standards that provide definitive guidelines on the rainfall intensity for which primary and emergency roof drainage systems should be designed. The lack of clear standards, particularly for emergency drainage systems, can lead to significant differences in the design of these systems.

Current standards for the design of building structures do not provide guidance on the consideration of rainwater loading in the calculation of structural elements. The analyses in this paper show that it is possible to design a structure to the standard where, in situations of extreme rainwater loading and significant structural deflection, emergency roof drainage systems designed as roof drains will be ineffective. This is particularly problematic for large halls with a flat roof and a double structural bay in the docking area. Although it is rare for all the main drains in a catchment area to fail, it is possible, for example as a result of a damaged or blocked drainage pipe serving a catchment area. In order to reduce the differences in deflection between aisles of different spans, the author believes that it is necessary to design the girders and roof purlins so that the structural elements in the areas of the longer span have comparable stiffness to those in the areas of the shorter span (e.g. by using higher steel trusses, reducing their spacing or juxtaposing steel trusses and prestressed concrete girders). Care should also be taken to ensure that the trapezoidal sheeting has a certain reserve capacity at the lowest point of the slope, allowing for a water bag at least the height of the rainwater accumulation. A comprehensive solution to this problem cannot be presented in a single article, as the phenomenon is more complex and requires simultaneous consideration of how the roof drainage system works together with its structure.

In the author's opinion, the most effective method of emergency drainage is to create an emergency system of drains and pipes capable of draining water from the lowest points of the roof slope. In this situation, the deflection of the structure will have a beneficial effect, as it will flood the primary and emergency drains, allowing them to work effectively.

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Odwodnienie dachów płaskich w obiektach wielkopowierzchniowych

Słowa kluczowe: dach, hala, katastrofa, odwodnienie, płaski, wielkopowierzchniowy

Streszczenie:

Obiekty wielkopowierzchniowe w większości bazują na tych samych materiałach i technologiach wznoszenia. Im większa powierzchnia dachu, tym trudniej zapewnić prawidłowe funkcjonowanie systemu awaryjnego odwodnienia dachu, który najczęściej realizowany jest za pomocą awaryjnych przelewów dachowych. Problem ten potwierdza co najmniej kilka katastrof budowlanych spowodowanych nagłymi i intensywnymi opadami deszczu, które miały miejsce w ciągu ostatnich 5 lat. W artykule odniesiono się do dostępnej w literaturze metody obliczania projektowego natężenia deszczu, a także definicji awaryjnego odwodnienia dachu. Zauważono, że istnieją znaczne braki normatywne regulujące kwestie awaryjnego odprowadzania wód opadowych z dachów, co jest szczególnie istotne w przypadku hal wielkopowierzchniowych z dachami o niskim nachyleniu. Kwestia projektowania zgodnego z normami pojawia się już na etapie określania awaryjnego natężenia opadów dla systemu odwodnienia dachu. Projektanci najczęściej opierają się na normach niemieckich, a także polskich, które zostały oficjalnie wycofane kilkadziesiąt lat temu. Dodatkowo powołują się na zalecenia producentów systemów odwodnienia dachów. Nie ma obowiązujących polskich norm, które zawierałyby ostateczne wytyczne dotyczące

intensywności opadów, dla których należy zaprojektować podstawowe i awaryjne systemy odwadniania dachów. Brak jasnych standardów, szczególnie dla systemów odwodnienia awaryjnego, może skutkować znacznymi różnicami w projektowaniu tych systemów dla tych samych warunków. Dodatkowo obecne normy dotyczące projektowania konstrukcji budowlanych nie zawierają jakichkolwiek wytycznych dotyczących uwzględniania obciążenia wodą deszczową podczas obliczeń elementów konstrukcyjnych. Analizy zawarte w niniejszej pracy wskazują, że możliwe jest zaprojektowanie konstrukcji zgodnie z normą, gdzie w sytuacji ekstremalnego obciążenia wodą opadową i znacznego ugięcia konstrukcji awaryjne systemy odwodnienia dachu wykonane jako wpusty attykowe mogą być nieskuteczne. Im bardziej konstrukcja będzie się uginać, tym więcej będzie przyjmować wody opadowej, co z kolei będzie dalej powiększać ugięcie. Jest to szczególnie problematyczne w przypadku wielkopowierzchniowych hal z płaskim dachem i zdublowanym przęsłem konstrukcyjnym w obszarze dokowania. Pomimo, że rzadko zdarza się, aby wszystkie główne wloty odwadniające w danym obszarze zlewni uległy awarii to takie zdarzenia są możliwe, na przykład w wyniku uszkodzenia lub zablokowania rury spustowej obsługującej dany obszar zlewni. Na podstawie przykładowej konstrukcji przedstawiono wpływ ugięć konstrukcji od poszczególnych przypadków obciążeń, w tym obciążeń normowych, a także od obciążenia wodami opadowymi w kilku wariantach, w zależności od przyjętej strefy obciążenia śniegiem. Z przeprowadzonych obliczeń wynika, że konstrukcje o zmiennej rozpiętości są szczególnie podatne na gromadzenie się wody opadowej. Ze względu na złożoność zjawiska, autor nie podaje jednoznacznego rozwiązania problemu. W celu redukcji różnic w ugięciu pomiędzy nawami o różnej rozpiętości należy zdaniem autora tak konstruować dźwigary i płatwie dachowe, aby w polach o powiększonej rozpiętości elementy konstrukcyjne miały porównywalną sztywność do tych w polach o mniejszej rozpiętości (np. poprzez zastosowanie kratownic stalowych o większej wysokości, zmniejszenie ich rozstawu lub zestawienie ze sobą kratownic stalowych i dźwigarów strunobetonowych). Należy również zwrócić uwagę, aby blacha trapezowa w rejonie najniższego miejsca na połaci miała pewną rezerwę nośności, która umożliwia powstanie worka z wodą o wysokości równej co najmniej wysokości piętrzenia wody opadowej z uwzględnieniem ugięcia w środku kratownic. Jest to szczególnie istotne w przypadku uwzględnienia oddziaływania wiatru, który – jak pokazuje doświadczenie z ostatnich katastrof budowlanych – może przetransferować wodę z jednej strony zlewni na drugą. Kompleksowe rozwiązanie tego problemu nie może zostać przedstawione w jednym artykule, bo zjawisko jest bardziej złożone i wymaga jednoczesnego rozważenia współpracy instalacji odwodnienia dach wraz z jego konstrukcją. W opinii autora, najskuteczniejszą metodą odwadniania awaryjnego jest wykonanie awaryjnego systemu wpustów i rur zdolnych do odprowadzania wody z najniższych punktów połaci dachowej. W tej sytuacji ugięcie konstrukcji będzie miało korzystny wpływ, ponieważ spowoduje zalanie wpustów podstawowych i awaryjnych umożliwi ich efektywną pracę.

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