Causes of defects in timber arches of the buildings covering and methods of strengthening

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Abstract. The results of the survey of covering of large-span public buildings located on the territory of the Republic of Belarus, which are in exploitation from 10 to 30 years. The bearing structures of the covering were timber glued arches with spans from 48 m to 60 m. The main defects were both through and deep cracks that appeared in the timber of the arches during exploitation. To restore the performance of timber arches with defects in the form of cracks, methods of strengthening are proposed. The effectiveness of using glued steel rods for strengthening arches with cracks was tested experimentally. For this purpose, two prototypes of semi-arches made of glued timber for a span of 26.9 m were tested: one semi-arch of a solid cross-section $b \times h = 100 \times 670$ mm; the second semi-arch with a crack on the neutral axis, reinforced with glued steel rods. As a result of tests, the ultimate load of a semi-arch with a solid cross-section is 2.4 times greater than the calculated value; and a semi – arch with a crack through the entire length is 1.8 times greater.

1. Introduction
Since the 1970s in the Republic of Belarus, there have been built over 10 long-span objects with the use of timber structures, which are still in operation. These objects include covered markets in the cities Gomel and Brest; equestrian and track and field arenas in Gomel and Minsk; ice arena in the city Novopolotsk; tennis courts and other buildings and structures. The main load-carrying structures of coverings in these buildings and structures are glued laminated timber three-hinged arches of circular shape. Spans of arches were accepted depending on the functional purpose of the designed building or structure. For covering the markets, the arches with a span of 60 m were used, and for covering the sports buildings – from 40 m to 60 m. The first object is the track and field arena in Gomel with a span of 49 m, which was designed and built-in 1976. This object was built as an experimental one with the designed service life for 25 years. This decision was dictated by the lack of experience in designing long-span timber structures, development of technology for their manufacture and erection, as well as the lack of exploitation experience. Subsequently, due to the exploitation practices, this approach was justified. The experience allowed taking into account several identified shortcomings in the design of future facilities.

It is well known that increasing the efficiency of using timber structures is closely related to concepts such as durability and reliability. These two concepts are provided at the stages of design, manufacturing, installation and exploitation stages. All these stages are especially important for long-span glued laminated timber structures, which are characterized by significant cross-sectional dimensions. Since wood is a heterogeneous anisotropic material that weakly resists the shear parallel...
to and stretching across the fibres, the design difficulties arise, primarily due to the transfer of forces in the nodal joints, as well as meeting more stringent requirements to comply with the temperature and humidity conditions throughout the building exploitation. The nodal joints of large-span structures are often subjected to large stresses concentrations [1, 2], which over time lead to the formation of cracks along the fibres and the destruction of the structural element or structure as a whole. Therefore, the results obtained when monitoring the state of long-span timber structures during the exploitation were considered not only as practical verification of the adopted design approaches but also as a source of information on the formulation of tasks, the solution of which is aimed at improving the current design standards and codes. It should be noted that several works [3–17] have been devoted to identifying the causes of defects in timber structures under exploitation. Some research works related to the methods for assessing the strength and elastic characteristics of timber, protecting the timber from biological damage, comprehensive assessment of the technical condition of structures, as well as the development of strengthening methods using innovative technologies. A considerable amount of research is devoted to the development of methods for a strengthening of the timber structures using local strengthening with screwed or glued-in rods [3,13], carbon fibre tapes glued on the wood surface of structural elements [7, 8, 14, 18, 19]. Each of these methods has both advantages and disadvantages. When developing this or that method of strengthening the structure, the exploitation conditions, types of defects, causes of defects, the type of stress-strain state of the strengthened element and its behaviour during further exploitation

Since in existing large-span glued timber structures the most common defects are surface and through cracks, the methods of fracture mechanics should be used when assessing the technical condition of structures, as well as developing strengthening methods [20]. It should be noted that currently there are insufficiently studied problems on the quantitative assessment of the effect of cracks on the bearing capacity of timber structures and, moreover, the development of cracks in time.

2. Methods
The survey of buildings was carried out according to the methodology developed by us. The object of the survey (inspection) was the coverings of buildings made of timber structures. A total of 8 buildings were examined situated in Belarus: track and field arenas in the cities Minsk and Gomel (Figure 1 a) and an urban-type settlement Stayki, an equestrian arena in Gomel, a market in Gomel (Figure 1 b), an ice arena in Novopolotsk, the passenger pavilion “Minsk 2” and the covering of the sports complex in Mogilev city.

Figure 1. General view of coverings for (left-right): the track and field arena with a span of 49 m in Gomel; 60 m market span in Gomel

The main load-carrying structures of the coverings under study were three-level arches made of the glued laminated timber with spans from 48 m to 60 m. The step of the arches was 6 m. Runs and braces were located at the level of the upper edge of the arches. Fastening of girders to the arches was carried out employing supporting tables and nails. The supporting tables were made of galvanized steel. Along with the runs, a single-layer continuous oblique flooring from grooved boards for insulation was laid. The flooring was mounted on nails both to the girders and to the upper faces of the arches. A layer of vapor barrier and girders were laid on the flooring, along which a protective single-
layer flooring was mounted under a soft or steel roof. Stained-glass windows were arranged in the covering along the longitudinal axes of the buildings.

The coverings of the market, the equestrian arena, the track and field arena “Stayki” excepted from the rule, where it was envisaged to arrange stained-glass windows only at the ends of buildings. The arches were supported on reinforced concrete frames – buttresses. The supporting and ridge nodes of the arches were made of glass-type welded metal shoes with open side surfaces, "dressed" on the beveled ends of the semi-arches. The shoes are connected to the semi-arches using five bolts with a diameter of 20 mm. With this solution, the perception of the transverse force in the node is carried out by the direct focus of the side surface of the semi-arches against the side surface of the shoe. Roller hinges were used in nodes. As for the supporting nodes of the arches in the track-and-field arena in Gomel and “Stayki”, they were a metal shoe of T-section, the wall of which was installed in the vertical groove of the semi-arch end with the depth of 160 mm, made along its entire length. The connection of the shoe with the semi-arch is carried out utilizing five pins. The length of the shelf (supporting platform) of the shoe was equal to half the height of the arch cross-section. The perception of the transverse force in the node is carried out employing dowels connecting the wall of the shoe with the timber of the semi-arch. It should be noted that in all examined semi-arches at a distance of 500 and 1000 mm along the upper face from their end and at a distance of 700, 1400 mm along the bottom, steel reinforcing bars are glued across the fibres. The diameter of the reinforcing bars was 20 mm and the length – 650 mm.

In the coverings of the track and field arena in Gomel and “Stayki”, three-hinged glued laminated arches of a pointed shape with a span of 49 m and a lifting arrow in the ridge of 10 m were used as load-bearing structures. The curvature radius of the arches was 40 m. Cross-section of arches was b×h = 200 x 1344 mm.

The main load-carrying structures of the coverings of the passenger pavilion of Minsk-2 airport were three-hinged arches of the variable radius of curvature with a span of 48 m and a 13 m lifting arrow in the ridge. The radius of curvature of the semi-arch in the area 2 m from the support was 11.9 m, in the area from 2 m to 6 m – 21.7 m and in the area from 6 m to the middle of the span – 38 m. The cross-section of these arches was b×h = 200 x 1344 mm. The load-carrying structures of the market covering in Gomel were three-hinged glued laminated wooden arches of circular shape with a span of 60 meters with a lifting arrow height of 13 meters. The step of the arches is 6 meters, section b×h = 280 x 1376 mm. Cross-section of semi-arches was assembled, consisting of bundles. Bundle width was 140 mm. The covering was made in the form of a single-layer oblique flooring from boards 25 mm thick, based on the glued laminated timber girders. The main load-carrying structures of the equestrian arena covering are timber glued three-hinged arches with a span of 42 m and a section b×h = 230x1200 mm. Composite cross-section, assembled from two bundles with a width of 115 mm. The step of the arches is 6 meters. Between themselves, the arches are fastened with struts.

The main load-carrying structures in the covering of the sports complex in Mogilev are glued laminated wooden three-hinged arches of circular shape with a span of 54 m and a camber rise of 10 m. The curvature radius of the semi-arches is 41.45 m. The cross-section of the arches is composite b×h = 280 x 1340 mm. The semi-arches are assembled from two bundles of 140 mm wide, interconnected along the length of the semi-arches employing bolts.

The main load-carrying structures of stained-glass windows arranged in the covering of buildings are transverse frames of glued laminated timber. The vertical frame pillars are fastened to the side surfaces using dowels. A solid single-layer oblique flooring of 25 mm thick grooved pine boards is fixed to the pillars and framework beams.

3. Results and discussion

3.1 Arch Survey Results.

In the process of inspecting timber structures of building covering, it was found that the cross-sections of arches, girders, struts, pillars and framework beams of stained-glass frames correspond to
the design ones. The same applies to designs of nodal joints of covering elements. As for the moisture content of the timber arches, it exceeded the design values and ranged from 11% to 20%. The maximum moisture content of the wooden arches of the covering of the track and field arena in Gomel was 14%, and that of the wooden arches of the passenger pavilion of Minsk-2 airport and the sports complex in Mogilev was 20%. Based on the external appearance of arches and the actual values of the moisture content of the wood, it was found that it was subjected to periodic wetting because of the roof leakage. This was especially noticeable in places of the stained-glass windows. The loss of the roofing imperviousness was caused by the icing in the covering endows in winter, as well as the loss of thermal insulation properties of the insulant. Periodic wetting of the timber arches led to the formation of the surface (blind) and through longitudinal cracks, which are mainly concentrated in the area of the stained-glass window. The maximum depth of blind cracks in the wood of the arches of the track and field arena in Gomel was 90 mm with a length of 2450 mm. In the arches of the airport passenger pavilion and the sports complex in Mogilev, the depth of blind cracks was 100 mm with a length of 3350 mm. Besides, a through the crack with the length of 6000 mm was discovered in one semi-arch of the track and field arena. The crack opening at the time of the survey was 5 mm. It should be noted that through cracks were found in the semi-arches of a sports complex in the city of Mogilev. One of the semi-arches had a through the crack with a length of 4560 mm and crack width of 12 mm. This crack was located in the middle part of the length of the semi-arch at a distance of 265 mm from its upper face. Besides, in one arch, for the most part of its length, a blind longitudinal crack was found with a depth of 130 mm. This crack was located in the middle of the height of the cross-section of the arch. Analyzing the relative frequency $f$ of the occurrence of blind cracks in the wood of the semi-arches, defined as the ratio of the number of identified cracks $n_i$ in a particular section of the semi-arches to the total number $n$ of the examined semi-arches in a particular object, it was found that:

- most blind cracks are concentrated in a section located at the distance of 500 mm from the support;
- in the arches of the track and field arena in Gomel, the maximum number of blind cracks is concentrated at the distance of 250 to 750 mm (Figure 2) from the support, and along with the section height at a distance of 110 and 770 mm from the bottom of the arch;
- with distance from the support of the semi-arch, the number of cracks decreases and begins to increase when approaching the ridge;
- the maximum number of blind cracks is observed in the arches on the south side of the buildings. This is because the timber, located closer to the support nodes, was subjected to greater moisture than the timber, located at a considerable distance from the support. The concentration of cracks in the areas located at the distance of 200 mm from the lower edge of the semi-arches is caused by both wetting and the transfer of vertical load on the semi-arches from the transverse frames of the stained-glass windows.

High frequency of cracks appear in the wood of the semi-arches located on the south side of the building, compared with the semi-arches located on the north side, is explained by differences in exposure conditions. So, the semi-arches located on the south side of the building were subjected to solar heating through stained-glass windows and the direct effects of warm air from heating appliances. All these contribute to a more intensive drying of the surface layers of timber arches and the formation of cracks.

It should be noted that when installing across the fibres, in the support and ridge nodes of the covering arches of the passenger pavilion of the airport Minsk-2 and the sports complex in Mogilev, three pairs of tie rods should only be considered emergency connections. The distance from the end of the semi-arch to the tie rods is 900 mm, 1400 mm and 2000 mm. This solution does not prevent cracking of the wood when humidity changes. Due to the time-dependent deformation of the timber, which is especially evident when compressed across the fibre, the tensile forces in the tie rods will decrease over time. This was discovered during the survey. Most of the tie rods were in a free state. To include the tie rods in the work of perceiving tensile stresses across the fibre arising from changes in the moisture content of wooden arches, it is necessary to ensure their tight connection with the timber.
This can be achieved by glueing the rods into the holes drilled in the wood of a semi-arch. The glueing of the rods should be carried out in the wood, which is least affected by changes in temperature and humidity conditions. The distance from the axis of the hole to the lateral edge of the cross-section of the semi-arch should be at least 30 mm.

Based on the results of the survey, it was found that the causes of cracking in the timber of the arches are violations of the temperature and humidity conditions, insufficient consideration of the characteristics of the wood’s resistance to stress, as well as the presence of stress concentration in the timber of the support and ridge nodes.

To determine the load-carrying capacity of the arches, verification calculations were performed. The values of permanent and variable loads were accepted taking into account the data obtained during the survey. As a result of verification calculations of the arches, it was found that in the arches to which the frames of the stained-glass windows are attached, the load-carrying capacity is lower by 12% of the calculated value. Overloading of arches occurs along the compressed sectional zone located at the distance of 9 m from the support. Stresses caused by the longitudinal force N account for 10% of bending stresses. It must be noted that the calculated resistance value in the indicated section was determined by taking into account the detected defects (cracks). To assess the degree of influence of cracks on the load-carrying capacity of arches, the numerical studies of fragments of arches with a longitudinal crack were performed.

3.2 Arch Survey Results.

In the course of the inspection and verification calculations, it was found that to cover the track and field arena, the design section of the arches is a section located at the distance of 9 m from the support. Normal stresses in this section are 10% of bending stresses. In the stretched section zone at the distance of 200 mm from the lower edge, a through the crack is located. The transverse force is close to zero in magnitude. Therefore, when choosing the design scheme of the arch fragment, the longitudinal and transverse forces were not taken into account. As a design scheme for an arch fragment, a 6 m long timber beam with a crack, loaded with two concentrated forces F (Figure 3 a), was adopted. The crack length was assumed to be 1000 mm. The crack was located in the zone of pure bending at a distance of 200 mm from the lower edge of the beam. The magnitude of the force F was determined from the calculation that the maximum values of normal stresses in the zone of pure bending of the beam are 10 MPa.

When choosing the calculated fragment of the arch to assess the effect of cracks on the load-carrying capacity of the arches of the covering of the passenger pavilion “Minsk-2”, we proceeded...
from the results of the inspection and verification calculations. Based on the data obtained, it was found that the two sections are the calculated design values. In the first section, the calculated value of the longitudinal force was \( Nd_1 = 245.5 \text{ kN} \), and the bending moment \( Md_1 = 385.6 \text{ kN·m} \). In the second section \( Nd_2 = 237 \text{ kN} \) and \( Md_2 = 377.4 \text{ kN·m} \). The transverse force in the indicated sections is close to zero. The crack length was 2000 mm. The crack was located in a compressed zone of the cross-section at the distance of 200 mm from the upper edge of the arch. Based on the obtained data, a beam of rectangular section \( bxh = 197 \text{ mm x 1344 mm} \) 6000 mm long was adopted as the calculated fragment of the arch. On the left, the beam was loaded with a moment \( Md_1 = 385.6 \text{ kN·m} \), and on the right \( Md_2 = 377.4 \text{ kN·m} \). The magnitude of the longitudinal force was taken equal to \( Nd = 245.5 \text{ kN} \). The design diagram of the fragment of the arch is shown in Figure 3 b.

The stress-strain state of the arch fragments (Figure 3) was determined using the program [2, 20] in which the finite element method (FEM) was implemented. The program uses isoparametric finite elements to simulate the singularity of the stress and strain field at the crack tip. Also, a procedure for determining stress intensity factors was implemented.

The analysis of normal stresses \( \sigma_x \) acting in sections 1-1 and 2-2 (Figure 3 a) resulted in establishing the following:
- in section 1-1 (Figure 3 a), the presence of a defect practically does not affect the value of normal stresses \( \sigma_x \);
- in section 2-2 (Figure 3 a), located at the crack tip of normal stresses, \( \sigma_x \) increases by 12%;
- the value of the stress intensity factor \( K_I \) at the crack tip is 0.27 MPa·m\(^{1/2}\).

Analyzing the stress-strain state of the arch fragment (Figure 3b), it can be noted that the presence of a crack in the compressed section zone leads to an increase in normal stresses \( \sigma_x \) by 18% and the appearance of stresses \( \sigma_x = 0.25 \text{ MPa} \) stretching across the timber fibre. The stress intensity factor is \( K_I = 0.28 \text{ MPa·m}^{1/2} \).

Figure 3. Design schemes of the fragments of arches and stress diagrams a) for arches section \( bxh = 200 \times 1100 \text{ mm} \); b) for arches section \( bxh = 197 \times 1344 \text{ mm} \)

To solve the question of whether further propagation of the crack will occur or not, we will compare their size with the characteristic value of the fracture toughness \( K_{Ic,k} \) at normal pull-off. According to [2], for wood with an average wood compactness \( \rho_{mean} = 500 \text{ kg/m}^3 \), the characteristic value of the fracture toughness is \( K_{Ic,k} = 0.29 \text{ MPa·m}^{1/2} \). As a result of comparing the values of \( K_I \) and \( K_{Ic,k} \), it can be argued that cracks will develop. This is confirmed by the results of the survey of the coverings, where cracks over time, as well as changes in temperature and humidity conditions, develop both in their depth and length. Having reached its critical length for the corresponding boundary conditions, the arch will be destroyed. Therefore, arches with through and blind cracks with the depth of more than 1/3 of the width of the cross-section should be reinforced.
3.3 Methods of reinforcing arches.

To strengthen the arches with defects in the form of blind deep and through cracks, steel reinforcing rods glued to the timber can be used. For blind cracks with an opening width of less than 3 mm and a depth of no more than 1/3 of the width of the cross-section of the element, the injection method can be used – injection of glue into the cracks under pressure. With this method, it is recommended to use epoxy adhesives.

When reinforcing the arches with glued-in rods, it should be taken into account that the stress-strain state of timber changes both along the length of the arch and the load application pattern. Therefore, the orientation of the rods concerning the timber grains should be taken so that the glued-in rod maximizes its strength in the weakest directions. According to studies, when the timber is reinforced locally with the glued-in steel rods, the optimal range of their inclination concerning timber fibres is an angle from 30° to 90°. When reinforcing arches, some rods should be perpendicular to the fibres and others – at an angle of 60° to the timber grains (Figure 4 in). The rods installed perpendicular to the wood grains will perceive the action of tensile stresses across the grains, and the inclined ones – shear stresses. Besides, the vertical rods will "intercept" the appearance of tensile stresses across the wood grains, due to the perception of shear forces by the inclined rods.

![Figure 4](image_url)

Figure 4. Schemes of prototype arches a) cross-section semi-arch; b) cracked semi-arch reinforced with glued-in rods 1 – semi-arch, 2 – rods with 12 mm diameter, glued-in perpendicular to the grains; 3 – rods with 12 mm diameter, glued-in at an angle of 60° to the direction of the grains

To experimentally verify the effectiveness of reinforcing arches with defects in the form of through cracks with glued-in rods, experimental studies of the arch models were performed (Figure 4). The parameters of the arch models were taken in such a way that the geometrical characteristics of the sections and the span were half the arches of the covering of the sports complex in Mogilev. The prototypes of the semi-arches were made of glued laminated timber and had the following dimensions: cross-section b×h = 100x670 mm, length to the middle of the arch span – L/2 = 13.45 m, the inner radius of curvature of the semi-arch – R = 20.39 m, lifting arrow in the ridge f=5 m. In total, two versions of the prototypes of semi-arches were manufactured and tested (Figure 4). The first sample was a semi-arch of a single cross-section. The second prototype was a semi-arch with a crack located in the middle of the cross-section. Reinforcing bars with a diameter of 12 mm were glued-in along the length of the semi-arch. The geometric characteristics of this prototype semi-arch are shown in Figure 4 c. Support and ridge nodes in two prototypes were adopted as in the arches of the inspected covering. Testing of the semi-arches was carried out in a horizontal position according to the scheme shown in Figure 5 c. The magnitude of the loads was taken according to the project and the results of the covering survey. The concentrated force F1 at the place of application and size simulated the load from the stained-glass window. Estimated value of F1,d = 72 kN. Concentrated forces F2 simulated a
uniformly distributed load acting on the covering. The calculated value of $F_{2,d} = 36$ kN. The ratio between the values of the forces $F_1$ and $F_2$ was equal to two. When a load was applied to the semi-arch, this ratio was maintained throughout the test. The loading of prototypes of the semi-arches was carried out in steps, with exposure at each step of 10-15 minutes. During the tests, the vertical displacements of the cross-sections of the semi-arches were determined using the D-1 – D-4 deflection meters (Figure 5c). Initially, a semi-arch without a crack was tested, and then a semi-arch with a crack. The calculated value of the load-carrying capacity of the tested semi-arches was determined from the experimental data, taking into account the duration of the tests.

As a result of the tests, it was found that the values of the ultimate breaking load for the semi-arch of the single section (without crack) are $F_{1t1} = 170$ kN and $F_{2t1} = 85$ kN, and for the semi-arch with a crack reinforced by glued-in rods $F_{1t2} = 130$ kN, $F_{2t2} = 65$ kN. The destruction of both semi-arches was brittle in nature and occurred due to chipping of wood in the support node with the formation of a longitudinal crack. A crack formed in the transition zone of the base platform and the level of the abutting end of the semi-arch.

Based on the test results, it can be concluded that the load-carrying capacity of the semi-arches reinforced by glued-in rods is 30% less than the solid-section semi-arches. It should be noted that the destruction of the semi-arch from bending did not occur, i.e. the effectiveness of glued-in rods is obvious. Besides, the ultimate breaking load of a semi-arch of a solid section is 2.36 times its calculated value, and for a semi-arch reinforced by glued-in rods – 1.8 times.

Analyzing the deformability of semi-arches (Figure 5d), it can be noted that it is significantly larger in the semi-arch with a crack in the middle part than in the semi-arches of a solid section. To increase the rigidity of the semi-arches in the specified zone, the pitch of the rods should be reduced.

4. Conclusions
Based on the results of the survey of wooden arches of long-span coverings of public buildings and structures, as well as numerical and experimental studies of their fragments, it was found that:
- the main defects that occur during operation are blind and through cracks, the geometric parameters of which depend on the temperature and humidity conditions of operation, the type

Figure 5. General view and diagrams of deformation “F-U” of the semi-arch prototypes during testing
a) general view of the testing the semi-arches; b) semi-arch test chart; c) diagram of vertical displacement $U_Y$ of the semi-arch ridge; d) deflections of the cross-sections of the semi-arch at $F_1 = 75$ kN and $F_2 = 37.5$ kN. 1 – for the semi-arch without crack; 2 – for the semi-arch with a crack reinforced by glued-in rods
of stress-strain state of the zones of formation of defects and the duration of buildings operation;
- the main causes of defects in the form of cracks in the timber of semi-arches are violations of the temperature and humidity conditions, the presence of stress concentration in the support and ridge nodes of the arches, as well as the action of tensile stresses across the timber fibres resulting from local transmission of compressive forces directed across the grains or at an angle to the grains;
- to assess the technical condition of arches with defects in the form of cracks, the methods of fracture mechanics should be used, that allow taking into account the crack length, boundary conditions and its location in the structure;
- arches with defects in the form of through and blind cracks with the depth of more than 1/3 of the cross-sectional width should be reinforced using glued-in stage reinforcing bars. The glued-in bars in the wood of the arches should be positioned so that they perceive the forces arising in the wood from stretching across the grains, shear along the grains, as well as stretching the wood across the grains that occur in the area of the inclined rods, perceiving shear stresses;
- to ensure the integrity of the cross-section of arches with defects in the form of blind cracks with the depth of not more than 1/3 of the width of the cross-section, the cracks must be injected using epoxy glue.

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