Variant Concept of Elevation of a Steel Grid Tower

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Abstract. The article presents the stages of analysis of an existing steel structure of a 35m-high lattice tower, located in a woodland. The assessment of the condition of the tower's structural elements and connections was made in the aspect of plans for its elevation, in order to install an observation camera warning against fire hazard. The tower up to 30 m was made of five segments with a square section, with decreasing width from 4.86 m to 2.05 m. The last segment, 5m high, was made as a cuboid. Truss walls and horizontal truss membranes were used. Technical platforms were installed inside the shaft. Rolled sections of St3S steel with a yield point f_d = 215 MPa were used as structural elements. The tower was erected on concrete spot footing. Static and strength analyses of the existing structure were carried out, taking into account the actual condition of materials and connections in the aspect of the planned modernization. The influence of wind gusts and the load from ice were taken into account. The user planned to install an observation camera on an additional mast with a height of 12 m. Initially, the concept of an additional construction made of Ø60.3 x 4 mm steel pipe set in a steel sleeve fastened to the bridge plates was adopted. The support tube was stabilized with four steel stay rope with diameter of 5 mm, fixed to the corners of the platform at 35m. As a result of the conducted calculations, it was justified that the adopted concept is inappropriate due to the fact that the limit values of horizontal displacement of the end of the mast, also unacceptable due to excessive vibrations of the image from the observation camera, would have been exceeded. Therefore, another concept was developed, in which a 12 m mast would be made with an aluminium lattice construction. In the horizontal section, the mast would have the shape of an equilateral triangle with a side length of 450 mm (420 mm in the axes). The mast branches would be made of RO35x2 round tubes, and the gratings would be made of rectangular, full profiles with a 20 x 10 mm cross section. The mast would consist of three latticed segments with a length of 4.0 m. The stays would be made of Ø5 mm steel ropes. Mast with sliding support would be stabilized with lashings, mounted on three levels. In order to obtain a proper tilting of the lashings in relation to the vertical axis of the mast, it is planned to use additional horizontal expansion elements made of square profiles RK80x5, located at the level of the highest platform. As a result of the calculations carried out, it was shown that a construction designed as presented would meet the conditions of limit states defined in the current standards of fire safety. The work was summarized by providing user guidelines for ensuring the durability of the modernized structure in the anticipated period of usage.

1. Introduction

A tangible effect of the so-called "global warming" is the increase in the number of fires in forest areas. The extent of damage caused by such fires could be limited by on-going video surveillance. Developing multimedia technologies and methods of wireless image transmission has modernised monitoring methods. Observation cameras are installed on traditional 30-35 m lattice towers built decades ago. Due to the increasing height of trees, the towers require modernisation which involves increasing their height. This involves reinforcement of existing structural elements employing, such as the methods presented in [1].

Figure 1. Overview and structural details of the lookout tower

An example of a 35 m lookout tower furnished with an additional six-meter steel mast is illustrated in Figure 1.

2. Identification of tower construction solutions

A fire lookout tower whose structure is analyzed here was originally built as a steel tower with a spatial lattice structure, standing 35 m tall. The lookout was placed on reinforced concrete foundation footings. The steel framework was connected to the foundation using steel anchors. The tower's cross-section is a square with a 4.86 m side at the base level and 2.05 m at the highest point. The tower's structure comprises of two main elements, at the bottom a 30 m pyramid pointing upwards, and a top cuboid with parallel horizontal legs, 5.0 m tall. The grid wall of the tower's trunk is a cross-bracing with primary and secondary spacers. The primary spacers are set at the level of joints of the diagonals and legs. Horizontal grid diaphragms are settled at the level of primary spacers.

An examination of the tower identified five segments of a varied height ranging from 6.05 to 5.90 m in the bottom portion and two 2.5 m segments in the top section. Inside the trunk of the tower's bottom segment, platforms were placed at the level of grid diaphragms, with technical bridges at the joint of the bottom and top segment, as well as the peak of the top segment. Access to the platforms and bridges is provided by steel ladders. The platforms and bridges are furnished with railings and the ladders come with steel hoops.

The legs and diagonals of the lookout were made from rolled equal angles. The spacers were mainly made from rolled C-sections. The tower was seated on reinforced concrete footings with a base of 2.4 x 2.4 m and 0.6 m height, on which reinforced concrete cores were placed, 0.5 x 0.5 x 1.8 m each. The steel structure is fixed to the footing with four steel M32 anchors.

The measurements done on site indicated that several years previously an additional 6 m pipe mast, stabilized with steel wire rope lashing and turnbuckles, was installed at the top platform. The mast was used as a support for a video camera and satellite dishes. The core of the mast was made from round steel pipe Ø60/4 mm. It was stabilized with four Ø5 mm steel wire rope lashings. The ropes were tightened with turnbuckles. The base of the mast was made in the shape of a cross using two pairs of C65 C-sections fastened to the horizontal spacers with screws. All steel elements were protected against corrosion with a zinc coating. Individual elements were connected to each other with screws. The screws have a metric thread and are M16 or M12 diameter, with an 8.8 class symbol. There are also screws with no markings.

As the trees growing around the tower increase their height continuously, making fire watches more difficult, the decision was made to extend the mast by additional 6 m.

3. Research of current material parameters

The assessment of the steel structure elements' usefulness for the installation of an additional mast was preceded by a study of actual physical parameters of materials which were exploited for decades, [2, 3, 4]. It was also necessary to determine the status of the connections of the steel elements.

Figure 2. Documentation of non-destructive tests of steel and concrete carried out "in situ"

Due to the lack of detailed design documentation, a range of visual examinations and non-destructive tests using specialised ultrasound devices (Figure 2), as well as strength tests (Figure 3), were performed, [5]. Before the measurements were made, the surface of steel elements where the material was measured for thickness and hardness was locally cleaned. Samples were taken and tested in a universal testing machine with a maximum load of 100 kN, [6]. The measurements indicated that the hardness of steel sampled from the steel structure tested experimentally corresponds to St3S standard, and the strength of screws conforms with the 8.8 design class. Some screws were locally identified to correspond to the 4.8 strength class. The condition of the protective coating was assessed as correct, as no concentrations of corrosive processes were observed on a structure operating in an aggressive environment, in the vicinity of industrial plants.

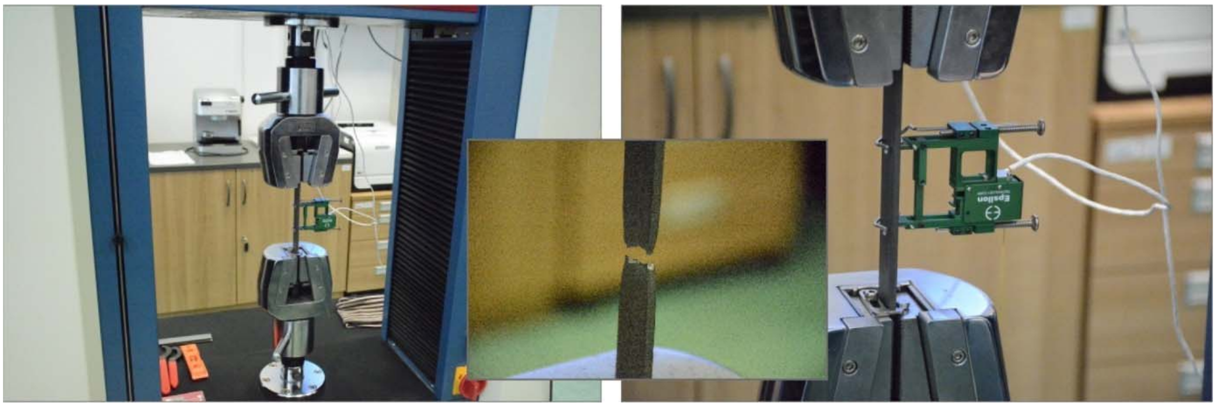

Figure 3. Examples of laboratory strength tests of steel samples

The condition of the foundation footing was assessed as qualifying for modernisation, and the strength of concrete, evaluated using a sclerometer, indicated the C20/25 class.

4. Assumptions of the modernization stage

The 35 m observation tower was erected decades ago. Visual observations of fire threat were conducted from the top platform at specific time intervals. Several years previously, the tower was furnished with a 6-m pipe mast, which was used to install cameras with antennae and devices transmitting image to the emergency management centre. The modernization quickly proved insufficient due to growth of trees. Consequently, the decision was made to dismantle the 6 m mast and replace it with a 12 m pipe mast. Upon completion of the works, after installing the camera at +47 m, it was found that the transmitted image is unreadable due to vibration and significant swing of the mast. Therefore, the decision was made to improve the structure solution which would ensure the required rigidity of the newly designed construction.

The authors verified another concept, consisting in erecting a lattice aluminium structure with a square cross section converging towards the top, 12 m tall, erected on top of the 35-m platform.

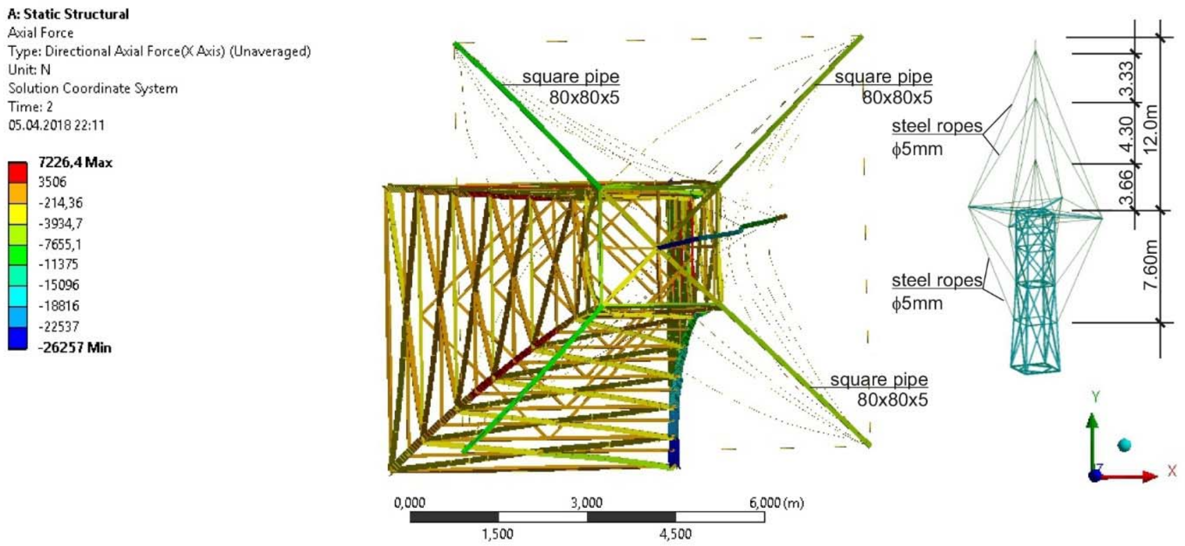

Figure 4. The newly designed structure based on the truss tower

The construction details of the analysed approach are presented in Figure 4.

It was assumed that a 12 m pylon would be installed on the +35 m level. A system of steel wire rope lashing was supposed to stabilise the pylon from excessive deformation and vibration. In order to obtain the necessary inclination of the lashing in relation to the vertical axis of the pylon, additional braces made from 60x4 square pipes, stiffened in the horizontal plane with taut steel ropes, were planned. The calculations assumed that the pylon would be a truss structure, resting on the top platform using hinges. The pylon would be stabilised with three rows of lashing. At each level, four lashings would be set at a 90° angle towards each other in the vertical plane. All of the ropes, the lashing and the ropes stabilising the braces, were designed as Ø5 mm with 2.0 kN tension.

5. Computational analysis of the modernized structure

In relation to the planned modernisation, the structure was subjected to a static and dynamic analysis in order to verify its compliance with standard requirements, [7, 8]. Additionally, control calculations were conducted in Ansys.

5.1. Verification of limit state standard conditions

The first class of reliability has been assumed, because the tower is located on sparsely populated forest area, thanks to which partial coefficients from constant and variable interactions from adverse interactions are γ_G=1,0 i γ_Q=1,2, respectively, which will be important for demonstrating the fulfillment of the limit state capacity. The calculations analysed the events of wind load, [9]. The force resulting from wind pressure is applied at structure junctions. The analysis considered wind pressure on the structure of the tower, as well as its additional furnishings: platforms, railings, ladders. The calculations did not include increased wind load caused by turbulence from objects in the immediate vicinity of the tower or the fact that the bottom section of the tower was under smaller load, as the structure was erected in a forest clearing. The structure was also analysed in terms of ice load, [10, 11].

Ultimate limit states were met by the main structure elements in all of the analysed cases.

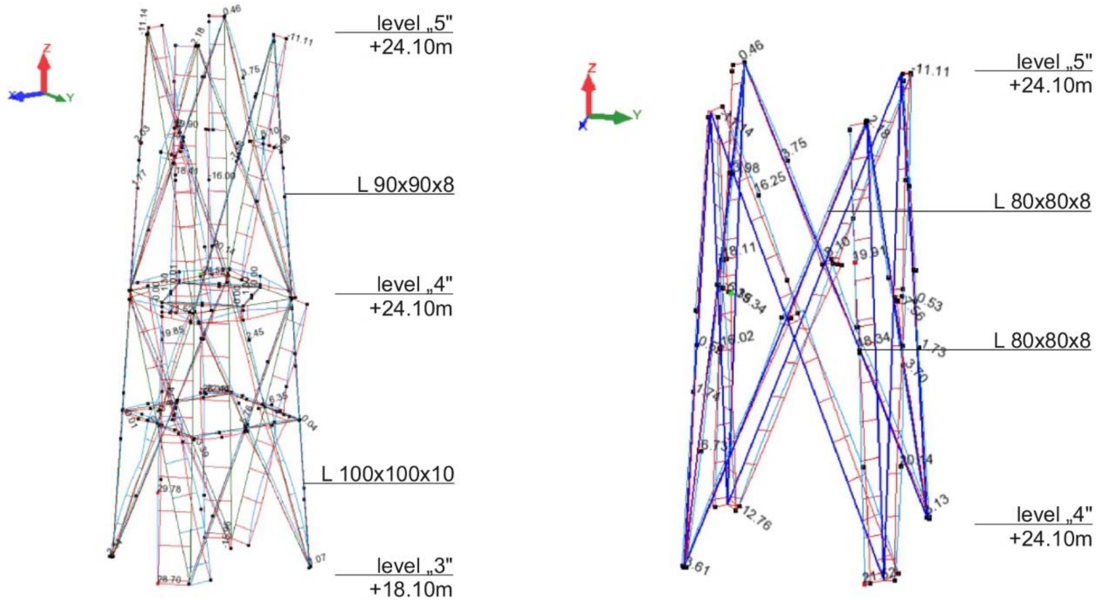

Figure 5. The state of effort of the structural elements of the tower at the mast superstructure stage of 12 m

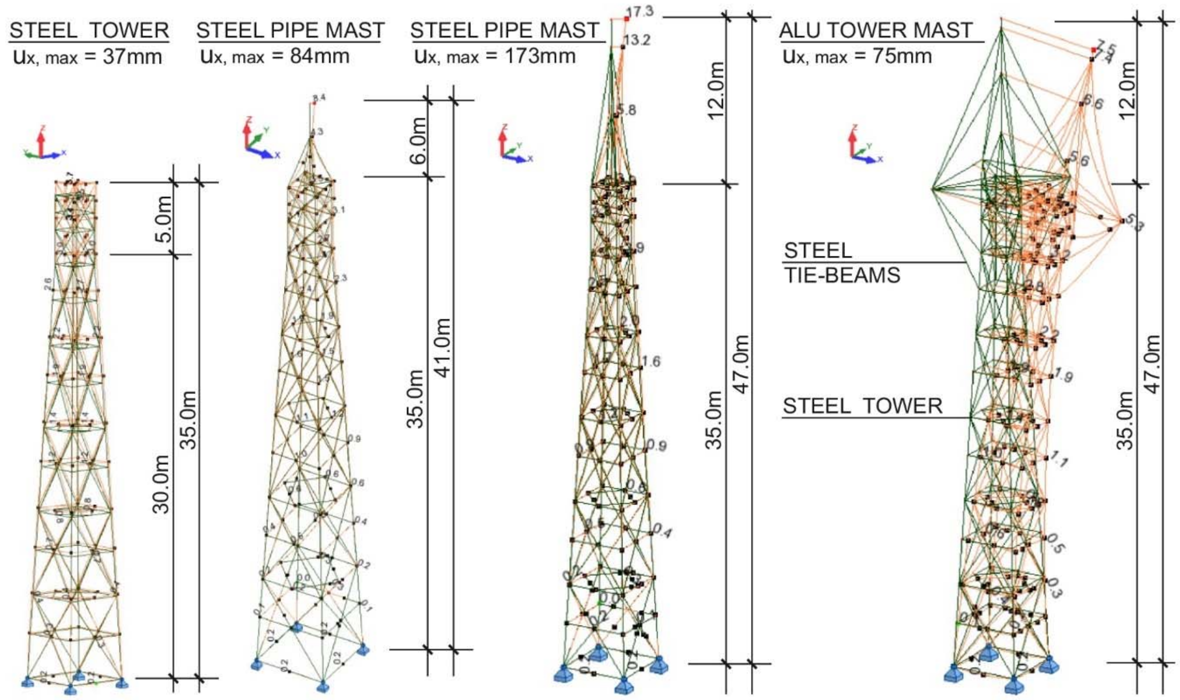

Figure 6. Horizontal deformations during subsequent stages of the modernisation of the observation tower

During the modernisation process, special attention must be paid to the limit states in relation to the tightened truss bars (Figure 5). At the first level, the lattice is made from LR100x8 even angles. The length of the bar is 7.52 m. In the parallel plane to the lattice, the buckling length of the bar is 3.99 m. In the perpendicular plane to the lattice, the buckling length of the bar is 7.52 m, with $\lambda y=246$ slenderness. According to [8], the coefficient of effective slenderness of bars connected to the junction with two screws may be calculated using the following formula: in the axis v-v: $k=0.7+(0.35/\lambda_v)$, in the axis y-y: $k=0.7+(0.40/\lambda_v)$.

The relative slenderness of the lattice bar is 2.65, and upon application of the coefficient of effective slenderness, it equals 2.25. This makes the buckling coefficient χ more favourable, equalling to 0.17, which has a positive effect on the ultimate limit state. This approach proved that the ultimate limit state was not exceeded and was at $N_{sd}/N_{Rd}=0{,}90$. A similar situation occurs in lattice bars at higher levels.

PN-EN 1993-3-1 standard requires consideration of second order effects and dynamic impacts. Second-order effects were taken into account by performing calculations (nonlinear analysis) in the calculation program. The dynamic impacts were taken into account by first calculating the own vibrations in the calculation program and then calculating the respective wind load values. The PN-EN 1993-3-1 standard does not explicitly specify permissible displacements and deformations. It requires checking of displacements or rotations that hinder the use of the structure and proper functioning of antennas and devices. No limitations are explicitly specified. N-B-03204:2002 Steel constructions - Towers and masts - Design and execution in checking the limit state of use take into account in particular the requirements for antennas or other devices installed on the site. Unless agreed otherwise, the following limit values shall be taken: the maximum displacement of the top of the tower or mast may not exceed L/100. Initially, when cameras were mounted on the tower, the guidelines stated that the apical displacement believed cannot exceed $u_x<L/200$, and later $u_x<L/300$. In the analyzed cases, this condition for a 12 m high tubular tower was slightly exceeded (Figure 6). However, during the operation of the object, the image transmitted from observation cameras during strong wind gusts was illegible and unhelpful in terms of the intended functions. As a result of the calculations carried out, a more restrictive condition for horizontal border displacements was formulated, where $u_x \leq L/500$ (Table 1).

Table 1. Horizontal displacements of the top of the different kinds of masts

Type of mast	Construction height [m]	Limit deflection EC L/300 [mm]	Limit deflection EC L/500 [mm]	Calculation deflection [mm]
lattice tower	35	117	70	37
tower + mast 6m	41	137	82	84
tower + mast 12m	47	157	94	173
tower + lattice mast	47	157	94	75

Additional control calculations were conducted in Ansys, a computer simulations platform popular among engineers. The tower was remodelled as a 3D bar structure made from rolled steel sections, whose diameters were determined in “in situ” tests. The limit conditions were modelled as hardening at the ends of the lowest vertical bars. The structure proposed to be built at the top of the platform (35 m) constitutes of four bars on the extension of the platform’s diagonals. In the calculation software, these form a rigid bar with the diagonal of the platform. Also, steel ropes are used for tightening the proposed structure. The ropes were introduced as cables for transferring only the axial force. The structure was loaded with its own load and wind pressure, as this load option was chosen upon earlier verification of the most unfavourable combination of loads, using Robot (Figure 7).

5.2. Verification of the strain and strain stage on the basis of the Ansys program

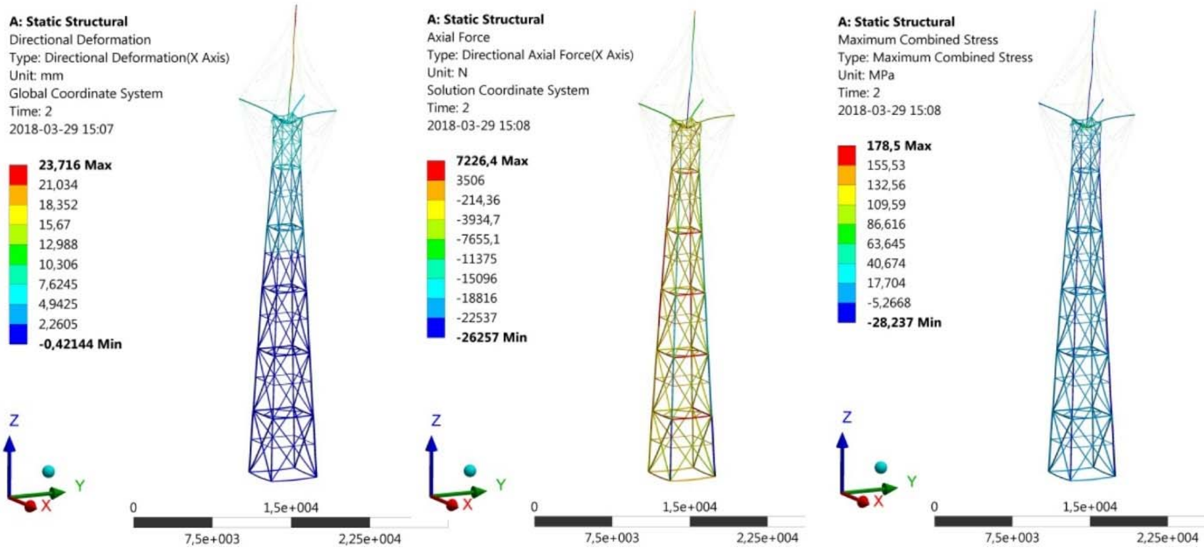

Figure 7. Numerical analysis of the final concept of the superstructure of the observation tower

6. Results and discussions

As a result of the studies of the actual condition of structure materials and the theoretical calculations, the drawbacks of the 12 m pipe mast solutions were justified. The maximum deformation in the horizontal sphere of the peak element of the tower, where the observation camera and transmission equipment were installed, equalled to 173 mm, despite meeting the ultimate limit state conditions by all components of the tower. Due to excessive deformation, the transmitted image was completely useless in terms of fire observation.

Installation of a lattice mast with a system of braces and lashings allowed to limit deflection to 74 mm < L/500 = 47000/500 = 94 mm, meeting all the requirements outlined in national sector specific rules, which are more restrictive than the requirements specified in the European standards [7, 8]. The conducted static and dynamic analysis showed that the construction of the 12 m pylon and the addition of steel rope stiffening elements, the natural vibration of the lookout tower will not change, [12].

The planned construction work comprising dismantling of the existing 6m camera and antenna support and installation of a 12.0 m lattice pylon with lashings at the +35 m platform on the existing fire lookout tower is possible and will not impair the technical condition of the existing structure, will not result in danger or reduce the safety of utilising the facility. The technical condition of the existing fire lookout is satisfactory in terms of the planned modernisation work.

7. Conclusions

The problem of utilising existing lookout towers is important because of their high numbers. Popularising the proposed, implemented and tested solution for strengthening the structure in relation to the necessity to make it taller would eliminate erroneous decisions. Additionally, it must be noted that in order to obtain a stable image it is necessary to limit vertical displacement at the top levels of the structure to L/500, and to carefully verify the method of fastening the flaccid, i.e. highly slender, elements which tighten the tower segments.

Analysis of structures which have been exploited for decades, information obtained within the currently popular BIM technology would prove helpful [13]. The development of the tower structure modernisation project indicated that it is valid to prepare a digital version of physical and functional characteristics of the structure to be used as a source of knowledge and data on the facility and base for decisions, such as the lifetime of the structure.

A separate matter is the problem with finding, through a public tender, for instance, a specialised company conducting modernisation work outside of urban areas, with difficult access for specialised, mobile installation equipment [14].

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