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Research Paper

Numerical Simulation of Deflection and Stability of the Wooden Transmission Tower of Gliwice Radiostation

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The wooden transmission tower of the Gliwice radiostation is the highest wooden construction around the world. Considering that it is in operation over 80 years it is necessary to properly maintain it to extend its residual life. In order to analyse the most critical elements of the tower as well as to plan the strategy of its maintenance, including repairs and replacements of wooden elements, it is necessary to analyse its deflection and stress distributions under possible loading scenarios. In this paper, the authors present an overview of history and construction of the tower as well as the results of static and buckling numerical simulations to evaluate a condition and a margin of safety, and to find the critical structural elements of this construction. The 3D numerical analysis is performed for the wooden transmission tower of the Gliwice radiostation for the first time. The results of simulations presented in this study indicate acceptable condition of the tower, in particular the resulting stresses under the static loading or buckling are at the level of 18% of the critical stresses according to the assumed geometrical model. However, further studies are necessary to refine numerical model and consider secondary elements of the tower, wood imperfections and other factors that have significant influence on its integrity and safety.

Key words: wooden transmission tower; Gliwice radiostation; wooden constructions; finite element method; buckling analysis.

1. Introduction

The historic wooden tower of the Gliwice radiostation is an object with a special wooden construction and unique history. It owes its difficult history primarily to the tragic events that took place on August 31, 1939, known as the "Gliwice provocation". These events permanently inscribed the tower in the history cards of both Poland and Germany. Noteworthy is also the fact that the tower survived over 80 years, during which the so-called winds of history were unfavorable. Despite the war and the passage of the front in 1945, the tower was not damaged. The tower also did not experience extreme climatic influences, such as hurricane winds or whirlwinds. A similar type of aerial wooden radio towers were built in Germany at that time, including, among others, Berlin (165 m high – used between 1933–1948), Ismaning (156 m – 1932–1983), Muhlacker (190 m – 1933–1945), in Szczecin (150 m – 1936–1945) and in Żurawin near Wrocław (140 m – 1936–1990).

The unique engineering craftsmanship of wooden construction, the precision of the selection of elements and shaping the nodes that provides lightness of the construction, as well as consistent renovation and maintenance activities throughout its lifetime made this tower, the highest in the world with a purely wooden structure, worth to admire. It is currently the only survived wooden tower from several similar ones, built before World War II for the needs of telecommunication. It is widely recognized as the highest, historic wooden structure in the world. Due to its age and unique technical solutions, as well as historical events from 1939. known under the name "Gliwice provocation", the tower is under conservation protection and was included in 1964 in the register of monuments of Republic of Poland. This object is currently used as a support structure for antennas, radio transmitters and cellular telephony. Currently, several tens of antennas with different masses are installed on the tower. The general view of the tower is shown in Fig. 1, and Fig. 2 shows the geometry of one of the tower's surfaces.

The history of radiostation and transmission tower objects cover many important events [1]. In August 31, 1939, in Gliwice radiostation historical events known as "Gliwice provocation" take place. Later, in 1939–1945 the tower was probably used as a civilian radio transmitter. In 1945–1946 the radio station is taken over by the Polish Radio. In the next years the tower serves as an aerial mast for Radio Katowice (1946–1952), and then in 1953–1956 the radiostation takes on a reserve role for the Polish Radio program broadcast from the station in Ruda Śląska. Starting from 1956 the tower ceases to act as a supporting structure for the transmitting antenna, but it is used to research new types of antennas.

In 1964 by the decision of the Provincial Conservator of Monuments in Katowice, the complex of buildings of the former Gliwice radiostation together with the tower is included into the register of monuments due to its historical value. In 1974 the radiostation objects are taken over by the Central Radiocommunication Laboratory; the production character of the changed profile is preserved.

Fig. 1. General view of the tower (photo: J. Brol 2017).

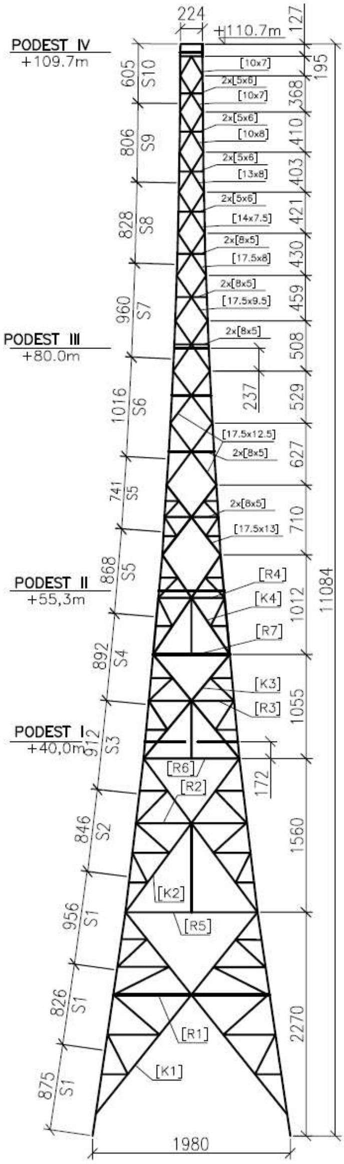


Fig. 2. Geometry of the tower [2].

Starting from 1972 and until 2002 the objects of the radiostation are owned by the Research and Development Center of Telekomunikacja Polska S.A., the tower serves as a support structure for many telecommunication and radio antennas.

In 2002 Gliwice city buys from the Research and Development Center of Telekomunikacja Polska S.A. antenna tower with accompanying technical buildings and residential buildings. In 2003 in accompanying facilities, a museum commemorates events related to the history of the tower is established. In 2005 the Radio and Media History Museum is being created at the radiostation territory, and in 2009 the three-hectare estate was transformed into an open-air park with fountains, alleys, benches and elegant vegetation, and as part of nationwide events commemorating the 70th anniversary of the outbreak of World War II, a tower illumination system was presented.

Considering the necessity of preservation and proper maintenance of the wooden construction of the Gliwice radiostation transmission tower it is decided to perform numerical calculations using finite element method in order to localize the most critical structural elements as well as to evaluate an overall structural condition of the tower subjected to its normal loading and environmental impact.

2. Wooden transmission tower of Gliwice radiostation

2.1. Characteristics of construction

The tower has a spatial lattice structure with square heights with variable cross-section. The axial spacing of columns at the base is 19.80 m. The tower consists of four profiled grids with common edges. The gratings form a spatial structure, the edges of which are parabolic, so that the surfaces of each of the gratings are also parabolic. The object has four platforms located at heights: 40.0 m, 55.30 m, 80.00 m and 109.70 m. From the level of the tower foundation to the level of the third platform, the lattice has a double-cross construction, with bolts passing through the intersections of crossbars and through points contact between diagonals and corner posts. Between the third and fourth level, the gratings are characterized by a double-cross structure with bolts passing through cross-brace intersections. The type of wireframe used, so-called double trellis with lacings, well braces a high tower structure. In the tower, there are additional stiffeners in the form of secondary lattice support posts and flat and spatial secondary gratings, reducing the buckling lengths of the struts.

The tower was made as a detachable construction of larch wood, the origin of which was not established. Corner poles have a cross-section that changes with the height of the object. Up to the height of the third platform, the pillars are four-leaf, and the above-one-branch. The rods of the tower are connected in its length by means of inserts and wooden covers. All elements of the tower are connected to each other by means of bolts and brass pins and rings. Each corner of the tower poles is anchored in the concrete foundation foot with four M60 steel bolts (Fig. 3).

As mentioned earlier, the pillars at the base have a four-branch cross-section composed of 200×200 mm bars with a 100 mm spacing (Fig. 4). Together with the height, the column cross-section is gradually reduced. At the level of 79.1 m,

Fig. 3. Support joint
(photo: J. Brol 1998).

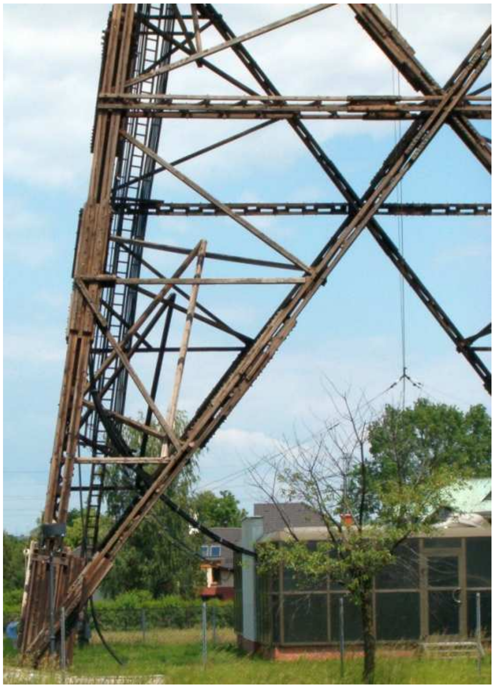


Fig. 4. Bottom of the corner leg
(photo: J. Brol 2002).

the shape and cross-section of the pillars changes. Four-sided poles are replaced with one-branch poles with a cross-section of 250×250 mm, which is also reduced with height. The last change in the cross-section of columns takes place at a height of 104.1 m. Column rods from this height to the level of 110.7 m have a cross-section of 120×120 mm. The cross-sections of columns are shown in Fig. 5. The symbols S1, S2, ..., etc. refer to the dimensions of particular elements of the tower (see Fig. 2).

The individual sections of the column (Fig. 6) are connected with each other by means of caps and inserts by means of rings, screws and brass bolts (originally, in the upper part of the tower, screws made of oak wood were used).

Also the cross-sections of the elements in the plane of the tower lattice are varied and change at the height of the tower. Up to the height of the second landing (55.4 m), the diagonals and bolts appearing in the planes of the gratings are made as four-leaf (Figs. 7 and 8), while above it has one branch. Figure 6 presents the connections between four-leaf transoms and crossbars with corner posts, and Fig. 9 shows interconnections of cross-braces, bolts, hangers and spatial braces. Spatial concentrations occur up to the height of the second landing, their cross-section is very diverse. All concentration rods at the intersections are

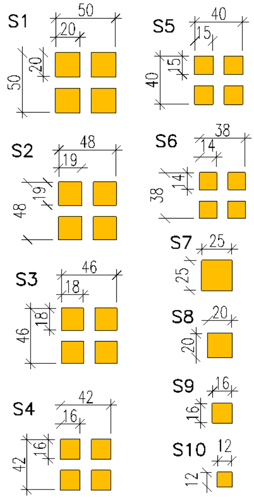


FIG. 5. Legs cross-sections [2].

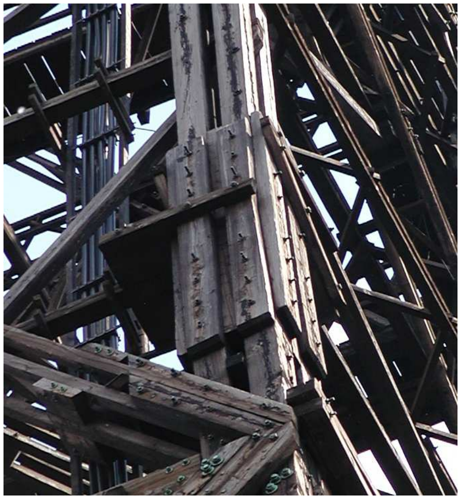
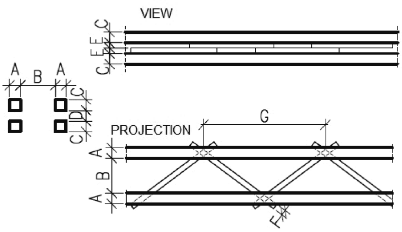
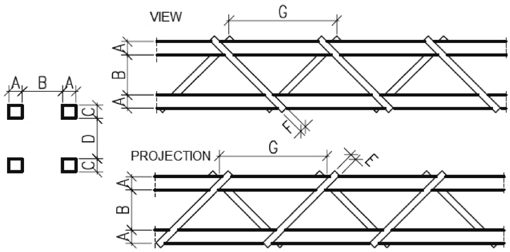


FIG. 6. A joint of the legs (photo: J. Brol 2002).

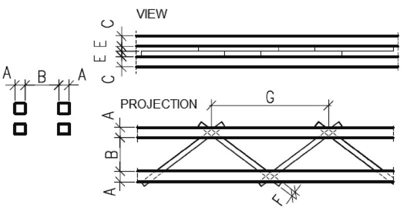


Cross-sect.	A	B	C	D	E	F	G
R1	9	50	9	16	8	6	88
R2	8	47	8	12	6	5	122
R3	7	41	7	12	6	5	116
R4	6	28	6	10	5	5	130



Cross-sect.	A	B	C	D	E	F	G
R5	12	26	12	26	6	6	132
R6	10	26	10	26	6	6	110
R7	10	22	10	22	5	5	150

FIG. 7. Cross-sections of the four-bars purlins [2].



Cross-sect.	A	B	C	D	E	F	G
K1	15	20	9	10	5	5	115
K2	8	28	8	10	5	5	150
K3	7	28	7	10	5	5	154
K4	6	28	6	10	5	5	123

FIG. 8. Cross-sections of the four-bars diagonals [2].

FIG. 9. View of the joint of diagonals, purlins and the spatial bracing (photo: A. Malczyk 1998) [2].

ensured continuity through the use of caps and inserts. Brass bolts and pins were used in all connections. The symbols R1, R2, ..., etc., and K1, K2, ..., etc. refer to the dimensions of particular elements of the tower (see Fig. 2).

2.2. Characteristics of materials

In 1998, in order to determine the compressive strength of the wood, bending strength and modulus of elasticity along the fibers, wood samples were examined from the collected elements of the tower structure (from 1935). In [2] the results of wood testing following the standards appeared before 2000 were presented. Using the notation of "K" type of these Polish Standards the wood from the tower from 1935 was classified as a wood of class K39. In strength tests, the results depend on the size of the samples. On small samples without defects, higher strength values are achieved than on larger samples, where wood defects cannot be avoided. According to [7], testing of strength characteristics of wood should be carried out on full-size elements. It is virtually impossible to determine the characteristics of wood for existing structures. Therefore, the study of strength characteristics of wood on small samples without defects according to the previously applicable standard [8], which is very often still used especially to determine the strength parameters of existing structures, was also carried out. According to these standards, bending test specimens should have the following dimensions: $20 \times 20 \times 300$ mm. The obtained results were converted into full-size elements using the formulas according to [7], taking into account the size of the samples and the dissimilarity of the test scheme. The procedure is described in more detail in [4]. The results of wood bending strength tests are presented in Table 1, and the average modulus of elasticity in bending was 15403 MPa. Here, the standard deviation (Std. Dev.) is presented as a parameter s.

Table 1. Strength of the larch wood obtained during experimental studies and conversion.

Type of wood	Average strength f_{ave} [MPa]	Std. dev. s [MPa]	f_{05} [MPa]	Corr. coeff. k_s [–]	Corr. coeff. k_v [–]	Corr. coeff. k_l [–]	Corr. coeff. k_h [–]	Charact. strength $f_{m,k}$ [MPa]
Larch wood 1935	97.68	6.85	86.45	–	1.0	1.32	1.50	43.6
Larch wood 1935	97.68	6.85	86.45	0.77	1.0	1.32	1.50	33.6

Based on the obtained results from the performed tests, due to the small number of specimens, it was limited only to the estimation of the current class of wood. Omitting the k_s coefficient (correction coefficient referring to the number and size of tested specimens), the tower wood, according to [6], based on $E_{0,\text{mean}} = 15.4$ GPa and $f_{m,k} = 43.6$ MPa, could be classified as class C40. However, taking into account the coefficient k_s, which for the small number specimen is 0.77, the wood would have to be classified as C30. Therefore, in order to hold the true mechanical properties of the wood for calculations, the wood class C40 was assumed for further computational analysis.

3. Numerical simulations

3.1. Definition of geometric model

The complex construction of the tower implies the application of 3D CAD modelling. Since the 1D beam model is an essential simplification, the subsequent calculations can be inaccurate and diverge far from reality. The geometric model of the tower was developed in the Autodesk® Inventor® Professional 2017 CAD commercial software as a 3D model. The geometric model, initially dedicated for numerical calculation purposes, was simplified in order to remove elements, which have no significant influence on the resulting stress distributions and displacements. Following this, the most of the secondary truss frameworks, which lower the tower buckling, and also vertical and horizontal lattices were not considered in the model. As a result, around 250 unique elements were created in the CAD software, from which the tower was composed. The geometric properties of these elements were prepared according to the technical documentation of the tower as well as based on the photographs taken in the period of realization of this study in 2017. The applied connections were of rigid type, and the fixed connections between purlins and diagonals were not defined, since in real conditions these connections were of sliding type in a truss plane [12]. The general view with particular detailed views on the most important elements and intersections are presented in Fig. 10.

Pillar arc geometry was developed based on the surveying measurements and further approximation of its curvature [1, 2]. After performing comparative studies based on the technical drawings, it was transferred to 3D sketch in the Autodesk® Inventor® environment. The corner pillars geometry can be constructed by applying an offset on the two 2D trusses. Since their axes are characterised by some curvature, the respective elements on the opposite plane have different sizes.

The geometry and the construction of the tower elements vary, depending on their position. The lower is the location of the tower element, the higher are the internal stresses. From the ground up to the third platform, a truss is constructed with purlins passing through the diagonals and crossing them (see Fig. 11).

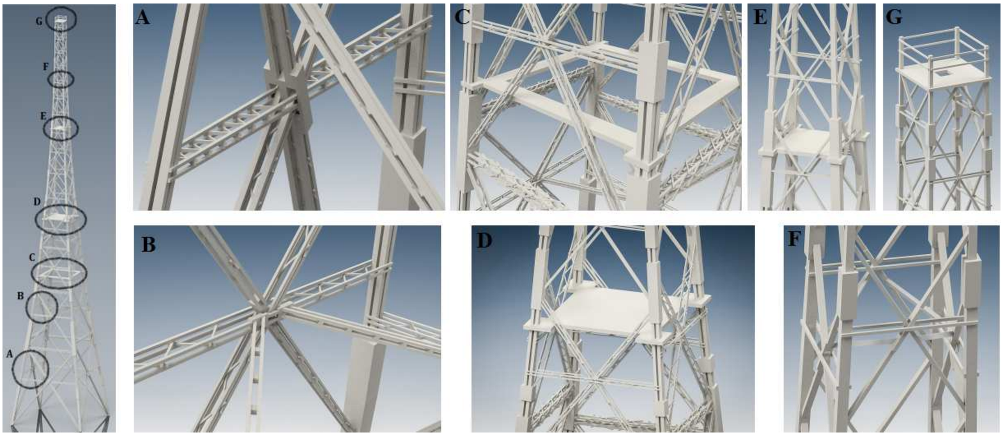


Fig. 10. CAD model of the tower: general view and particular views on the most important elements.

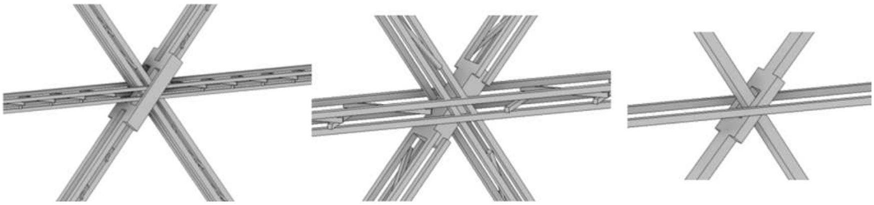


Fig. 11. Examples of junctions of two diagonals and one purlin.

In these segments, the purlins are also located at the contact locations between the diagonals and the corner pillars. They cannot be in the contact with each other in the joints, so no additional transverse forces act on beams. Above the third platform, the purlins are located only at the points, where two diagonals crosses. These elements are also much simpler (see Fig. 11). Such type of latticing is typical for tall towers. Up to the second platform in the junction of the diagonals, the wooden bars are placed, which help hanging up the purlins (three areas with this type of hangs can be distinguished). The example of such a purlin is presented in Fig. 12.

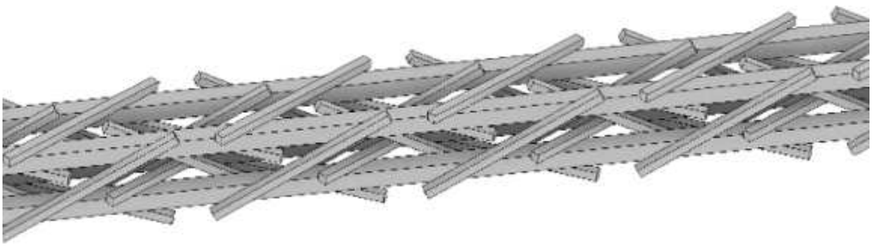


Fig. 12. Example of modelled purlin.

In such joints all the elements can freely move without interacting with each other. This prevents the occurrence of the tangential forces. All of the connectors, like brass bolts and screws used in the tower construction were substituted by appropriate contact constraints in the numerical model [13].

3.2. Definition of finite element model

The numerical analyses were performed in the ANSYS® 17.1 finite element commercial software. The CAD model was exported to ANSYS® and subjected to mesh discretization using linear tetrahedral 3D elements with its total number of almost two millions [12]. The views of the discretized model is presented in Fig. 13.

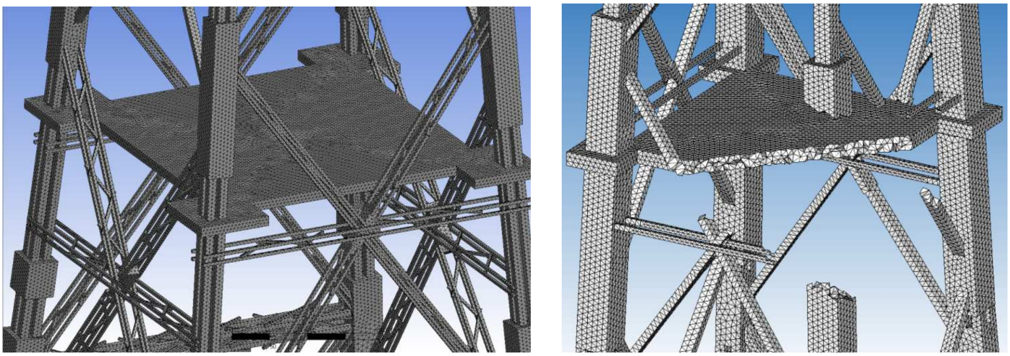


Fig. 13. View of the discretized model of tower with a cross-section.

Over the years, the construction elements of the tower had to be repaired many times or replaced with new ones. In 1969 the broken Larch wood elements were replaced by cheaper parts made of pine wood. However, because of the fast wood-rotting, they had to be replaced again with new Larch wood in 1997 [1, 2].

The calculation of the complex structures would consume too much computation time, if the anisotropy was taken into account. Therefore, as mentioned before, wood is considered as an orthotropic material. In reality the strength of wood varies significantly, once the moisture content in the wooden fibres is changed. At the maximum moisture content of beers ($w = 30\%$) the strengths decreases 50% for compression, and 40% for bending [5].

Bending and compression strengths differs also depending on the volumetric density of the material. It is assumed that reduction of the density from 600 to 400 kg/m^3 equals to the strength reduction over more than 1.5 times. Wood as a building material is segregated into strength classes (according to the standard [6]), class C for soft and D for hard wood, respectively. The number following the letter refers to the characteristic bending strength, given in MPa, at a moisture content of $w = 12\%$. The properties used for the definition of material properties of the numerical model, according to the assumptions made on Subsec. 2.2. for the wood class C40, are stored in Tables 2 and 3.

Table 2. Strength of the wood class C40 [6].

Strength type [MPa]	Compression	Tension	Bending	Shear
Parallel to grain	26.0	24.0	40	3.8
Perpendicular to grain	2.9	0.4		–

Table 3. Mechanical properties of the wood class C40 [6].

<table>
<tr><th colspan="3">Modulus of elasticity [MPa]</th><th rowspan="2">Average density [kg/m^3]</th></tr>
<tr><th>Average</th><th>5-percentile</th><th>Average shear</th></tr>
<tr><td>$E_{0,mean} = 14000$</td><td rowspan="2">$E_{0.05} = 9400$</td><td rowspan="2">$G_{\text{mean}} = 880$</td><td rowspan="2">$\rho_{\text{mean}} = 500$</td></tr>
<tr><td>$E_{90,\text{mean}} = 470$</td></tr>
</table>

Various types of loading, act on the tower and their occurrence significantly influencing the strength and the mechanical behaviour of the structure. Their disregarding can be lethal for lifespan of the construction. Hereafter, the most important types of loading acting on the tower are distinguished:

- Dead load of the tower and its equipment – vertical loadings are caused by self-mass of the structure, the mass of equipment (platforms, antennas, ladder, wires etc.). Most important is to calculate the mass of a tower core, because it has the highest value among them all.
- Icing – the occurrence of ice increases the area exposed to the wind and significantly enlarge the mass of the tower. The distribution and the size of icing depends on the local conditions. This type of loading is often uniformly distributed over the tower. The standard [10] holds a procedure to compute it. In this study, the object of the interest is located in second zone of climate influence.
- Wind loading – this type of loading is one of the most important factors. It is necessary to compute the wind resistance of the tower, because the pressure generated against the structure can be very dangerous. Wind load is proportional to the exposure area and a distance from the ground. Due to the fatigue, the most dangerous are cyclic and stochastic, turbulent loads. Wind loading is usually determined using the procedure from the standard [11]. The Gliwice radiostation is localised in 1st wind zone. Since, it is a very unpredictable type of loading, different horizontal wind directions should be taken into account [1].

The own mass of the tower was determined by ANSYS®, using geometry and applied Larch density with the moisture of 12% (according to [6]). To compensate the mass of missing bars, the mass of the model was increased by 30%. Furthermore, to simulate icing, the mass was increased by additional 50%. The mass of aerials in the total amount of nine was taken from [1] and added in the form of concentrated forces.

Wind load acting on the tower was determined using the procedure from the standard [11]. The Gliwice radiostation is located in the 1st wind zone in open space. Calculations were performed for two scenarios: with a wind acting perpendicularly to the lattice and at angle of 45° to the lattice.

3.3. Analysis of deformation and stress under static loads

The static analyses were performed considering the above-presented geometric and numerical models for two considered scenarios: action of wind in parallel to the lattice, and with an inclination in the horizontal plane of 45°. The resulting displacement maps are presented in Fig. 14.

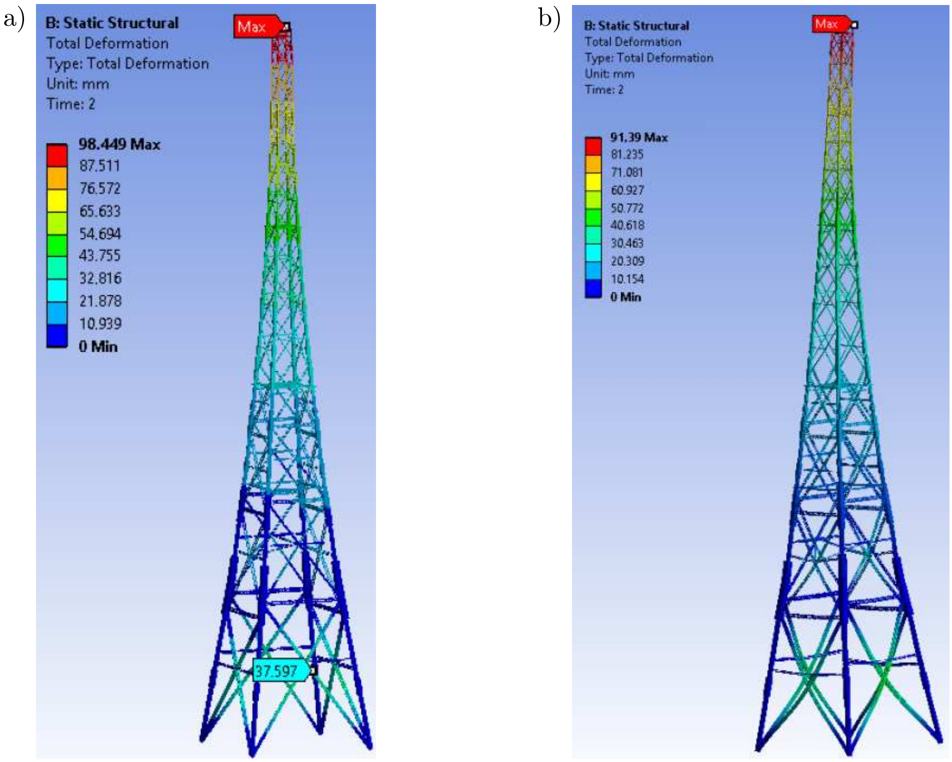


FIG. 14. Total displacement for the tower model with considered loading scenarios: a) parallel loading, b) loading at the angle 45°.

Maximal deformation is observed mainly on the top of the tower. Setting the limits is difficult due to the lack of any standards concerning a displacement of wooden towers. Because of that reason for analysis purposes the standard [9] was used. Due to this standard maximum tower tilting is equal 1% of the structure height (in this case 1.1 m), but according to the heterogeneity of the wood, changing of wood properties with change in moisture content and loses in joints the safe zone is probably half smaller. Vertical compression of the structure when the wind load is omitted is equal to 43 mm. A total deformation of the tower loaded with the wind acting at the angle of 45° to the lattice is equal to 91 mm and in parallel case to 98.5 mm. Lower displacement in the first case is caused by smaller value of wind load. Therefore, the obtained displacement is of 18% of the assumed maximal value.

Comparing to the results obtained for wireframe model of the tower by Adamczyk [1], where the maximal displacement values equal to 217 mm, the obtained results of displacement are over 50% lower with respect to this value. Such a difference in results is probably caused by considering more stiffening elements in the geometric model presented in this study as well as investigation of mechanical behaviour of the tower based on 3D geometric and numerical models. These results can be considered as upper and lower estimates of the true displacement values. According to the assumed criterion, the tower is safe with respect to calculated displacement.

Additionally, the stress distribution (calculated following the Mises-Huber-Hencky hypothesis) was obtained for the considered model. The results for two scenarios were presented in Fig. 15.

The highest equivalent stress values can be observed in the main tower pillars, particularly in the locations, where they meet with the diagonal bars. The massive mass of the bottom diagonals also causes accumulation of the stress near the joints, even when only dead load is applied. Probably, if the bottom secondary truss frameworks were considered in the model, then stresses would distribute more evenly. In the case of the wind loading perpendicularly to the tower diagonal, the loading stress is 25% smaller of characteristic values of the wind load. However, due to the fact, that equivalent stress accumulates in a single pillar, the factor of the stress gain is higher than in the second case when the wind acts parallel to the lattice. Even after applying 2.5 times higher wind loads, the equivalent stress values do not exceed the Larch yield strength (see Table 2), but it is important to have in mind that this calculation does not take into consideration local wood defects.

Analysing results for the shear stress (Fig. 16) one can observe the maximal values that are quite close to the assumed yield shear stress value (see Table 2). The highest shear stress values are observed in the locations, where the truss framework connects with the main tower pillars. In reality, these areas are much more robust, and shear stress values are presumably much smaller.

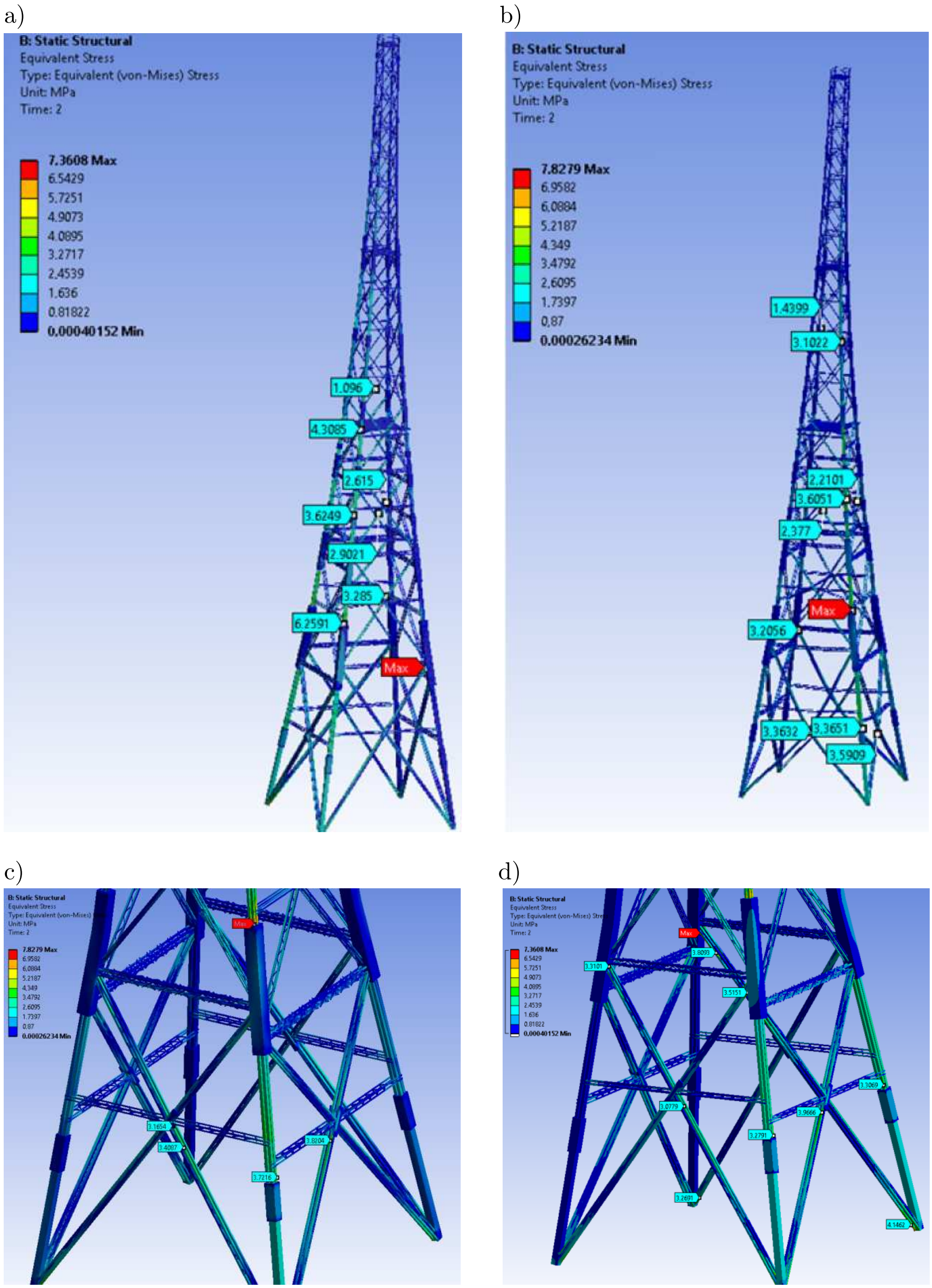


FIG. 15. Equivalent stress for the tower model with considered loading scenarios: a) parallel loading, b) loading at the angle 45°, with zoomed views (c), (d).

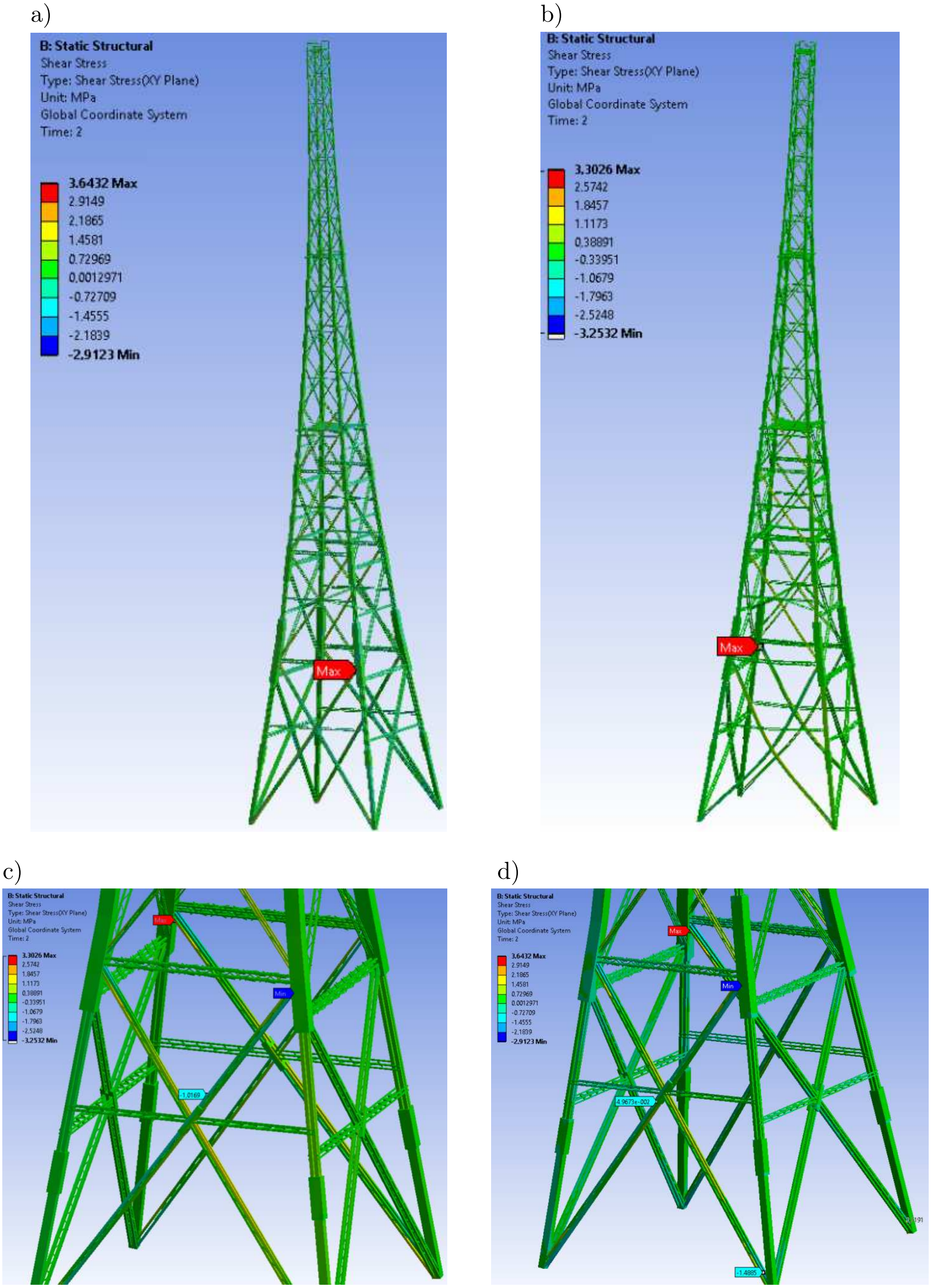


Fig. 16. Shear stress for the tower model with considered loading scenarios: a) parallel loading, b) loading at the angle 45°, with zoomed views (c), (d).

3.4. Buckling analysis

Buckling analysis is performed to examine if an assumed set of loads will cause buckling, and to investigate the buckling modes. After this examination,

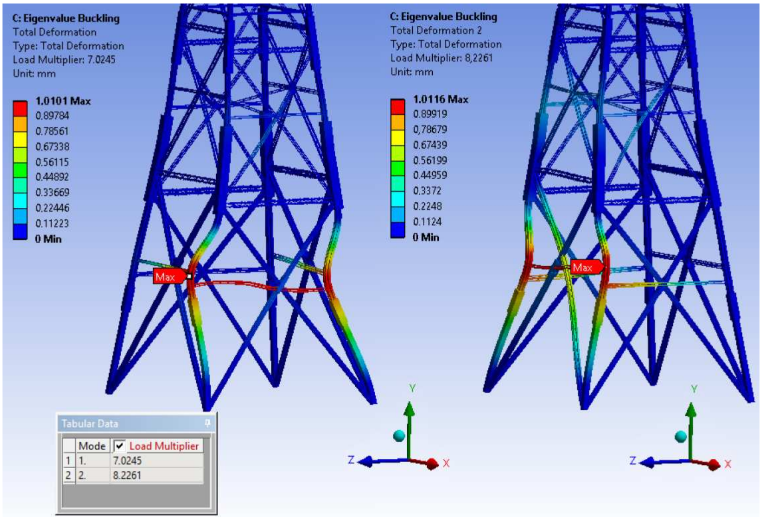


FIG. 17. First two buckling modes for the scenario of the wind acting in angle of 45° to the lattice.

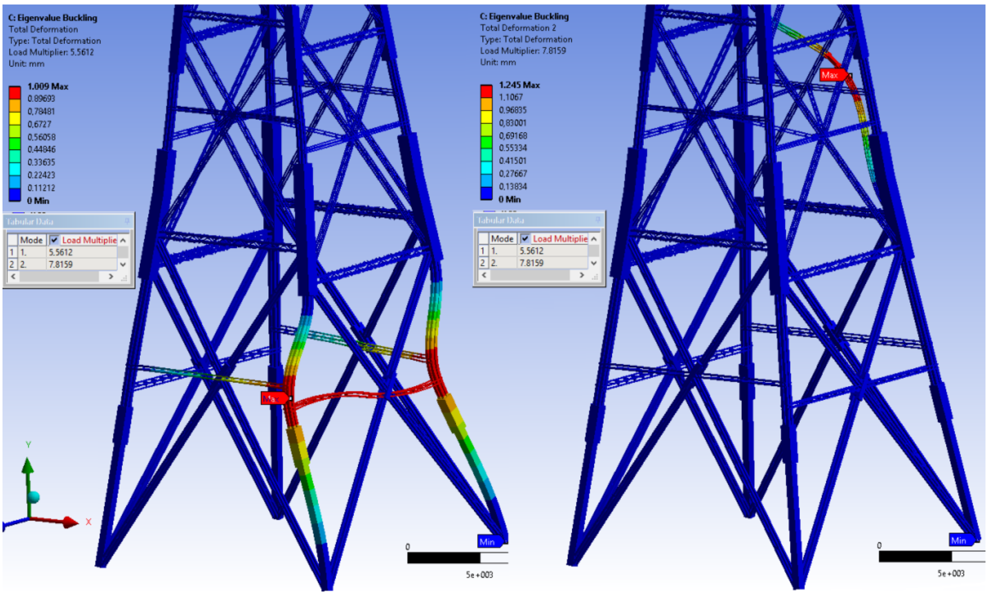


FIG. 18. First two buckling modes for the scenario of the wind acting in parallel to the lattice.

additional design supports and stiffeners is performed in order to prevent this phenomenon. Only forces which may cause compression, affect the buckling.

To calculate eigenvalue buckling, the load was applied in the form of a prestress from the static simulation. The result of this study is a load multiplier, which indicates when the model will buckle. Theoretical load that causes buckling is evaluated by multiplying all of the applied loads on the model by the buckling load multiplier. It is important to have in mind that real parts due to the imperfections, may buckle in much smaller loads. Since this result is an eigenvalue solution, the displacement results will explain the buckling mode shape, but the quantity of the displacements is meaningless. Furthermore, there are no stress or strain results from a critical buckling analysis. The tower is resistant to the buckling when the case with the wind in angle of 45° to the lattice is considered. The load needed to buckle the tower is 7 times higher than that assumed in the analysis. Two modes of buckling are similar to each other, and the most susceptible is the pillar with the highest values of equivalent forces (in static simulation). The results of buckling for this case are presented in Fig. 17. One should note that the simplification of the tower model, i.e. omitting secondary elements in this model, may significantly influence on the buckling modes. However, since the omitting this secondary elements results in decrease of stiffness, one can assume that the critical buckling load in this case is even higher than one obtained in the numerical analysis.

The second simulation, reveals that first mode of buckling is the most likely to appear in studied cases. Load multiplier is smaller, probably because applied loads are a little bit higher. Additionally, slightly higher displacement is observed for this case. Because the wind load is acting in parallel to the tower, the second mode is different, and it reveals possible buckling in one of the diagonals (see Fig. 18).

4. Discussion and conclusions

In the extended periods of time after World War II the maintenance of tower was poor or none. Also due to the proximity of chemical plants and heavy industry, the Tower is exposed to polluted atmosphere (among others acid rains). Because of that it is highly recommended to clean and after that correctly impregnate all of the towers members. Chemicals should primarily prevent from the expansion of decay and reduce water absorptivity. Nevertheless, the results of the performed numerical calculations, with taking into consideration cases with overestimated dead loads, indicated significant strength reserve. Essentially, in the performed calculations several significant factors were omitted for the sake of simplicity and shortening the calculation time. These factors include primarily heterogeneity of wood, existence of extended cracks and possible internal damage, and several simplifications in the tower's geometry, which are planned to be investigated in further studies.

Simulation tests in ANSYS® revealed the highest strains in the pillars, bottom diagonals and purlins, and also in connections between them and pillars. The performed numerical studies allow for evaluation of a criticality of stresses acting at typical loading and environmental conditions. The results of these studies clearly show that the construction of the tower is far below the critical stresses, and its operation is safe. This conclusion can be done even considering the simplification of geometrical model (excluding of secondary elements from the analysis) and material model (not taking into consideration wood imperfections). However, in order to increase the accuracy of these predictions consideration of these factors in further studies is planned.

From the practical point of view, it is recommended to reinforce these beams with materials made of carbon fibre with reinforced polymer (CFRP) to enhance the compressive and shear properties of the Larch wood. CFRP sheets applied at the surface of bar prevents from crack opening, confines local rupture and bridges local defects in the timber. This method is particularly useful with the bars which for various reasons, are difficult to replace in situ. FRP methods are widely used for reinforcement of concrete structures, but this technology more and more frequently appears in wood maintenance repairs. At present, there are many techniques of reinforcing a wood member using different layouts of the FRP elements, and potentially each may lead to the different results. For this reason, selection of the reinforcement design and material should be supported by adequate research in order to avoid ineffective interventions [3]. Such reinforcement is an easily applicable solution typical in maintenance of old wooden constructions, which may prevent or limit the consequences of structural degradation of wooden elements of the tower subjected to various mechanical and environmental factors, in particular the separation of the lower-area cross-sections from the solid element due to cracking, which increases the slenderness of this solid element significantly.

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