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Research paper

The impact of leg spacing of telecommunication towers on their bending stiffness, total weight, and dynamic properties

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Abstract: The paper presents analyses aimed at determining how the spacing between legs of a steel lattice telecommunication tower affects displacements of its top, its natural frequency, and its self-weight. The critical characteristic of the results is that they were obtained for structures optimized with respect to the load-carrying capacity of their individual components. It was assumed that levels of effort should be kept between 85% and 95%. As far as engineering practice is concerned, the key conclusion is that a larger distance between the legs has a positive impact on the self-weight of the structure. It was demonstrated that a larger leg spacing is related to a smaller self-weight of the tower and thus a smaller quantity of material required. The proposed research and calculation method makes it possible to conclude that providing a larger distance between the legs while optimizing structural members with respect to their effort (preferably by means of an automated process) results in both higher bending stiffness and lower structure self-weight required.

Keywords: dynamic, lattice structures, stability, standards, telecommunication tower, wind load

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1. Introduction

Tower structures of multiple forms and serving numerous purposes have been present in public spaces for a number of centuries [1]. Such objects include mainly office and residential buildings (skyscrapers, high-rise buildings) and civil structures (viewing, radio, transmission, or telecommunication towers). This paper deals with tower support structures for telecommunications purposes.

Telecommunication towers are the passive part of a digital mobile telephony network. They serve as structures supporting sophisticated telecommunications equipment such as microwave antennas, sector antennas, or remote radio units with cabling. The quantity of such structures in public spaces is increasing in line with the rising demand for unlimited high-speed wireless data transmission services, e.g. the 5G network and broadband Internet, and for an improved signal quality in general telephone communication. The level of expectations regarding modern telecommunication services is directly reflected in passive infrastructure design and construction requirements. Telecommunication towers built in 1990s, for instance, are currently used for purposes for which they were not explicitly designed. This is most notably demonstrated by the increasing use of such structures for mounting solar panels or small wind turbines [2]. As a matter of fact, the authors of paper [3] are right in pointing out that "the telecommunications sector is a fundamental pillar of today's society". Given the above mentioned aspects, an obvious conclusion is that there has been a significant progress in the scientific research and development concerning the design and construction of telecommunication tower structures. It also must be acknowledged that the conservative method of designing tower structures has definitely ceased to be viable. Solutions that are needed nowadays should make it possible to achieve the best weight-to-stiffness ratio, as is proposed for example in [4].

The climate change is another new and unique challenge that affects not only the design of tall structures but also the current human living conditions in general. In this context, gusty or even hurricane winds are another factor resulting in higher requirements concerning how self-supported towers are analyzed [5]. Increasing wind action values combined with a constant pressure on reducing costs of building such objects prompt the need to use more sophisticated methods of determining wind actions in scientific studies [6]. Wind tunnel tests are used to answer questions such as how ancillaries mounted on an object impact the aerodynamic drag [7, 8]. In routine engineering practice, however, it is not possible to carry out the wind tunnel tests due to high costs and the long time needed to analyze the results. Instead, codes and standards are commonly used. When working with standards, one often finds that each standard might recommend or provide a slightly different approach to calculations. For example, when estimating wind actions we will obtain different load values using U.S., Australian, or European standards. Such discrepancies can result in obtaining different axial forces acting on tower legs, which is demonstrated in [9].

Technical means of reinforcing tower structures are another important aspect of maintaining technical infrastructure [10]. This applies to existing objects that have been used as support structures for a dozen-plus years or even for a few decades. Telecommunication towers need to be reinforced usually not due to problems related to aging, corrosion, or a general failure condition. In fact, they have to be reinforced mainly when their load increases after mounting new telecommunications equipment (for instance, when one structure is used by two mobile network operators) or when wind action standard provisions are amended. The research work on this topic focuses on new concepts of reinforcements of angle sections [11, 12] or round solid bars [13].

In addition to all quite obvious conditions concerning the load-carrying capacity, support structures used in the telecommunications industry should also meet one specific requirement: the adequate bending capacity. For telecommunications facilities, the displacement capacity of tower tops, depending on the equipment installed, can be limited to as little as 0.5°. In short, this is necessary to provide telecommunication services of appropriate quality (e.g. uninterrupted phone calls) regardless of weather conditions. Therefore, a key task is to determine the bending stiffness of telecommunication towers, especially when selecting a structure for a given location and a certain antenna arrangement.

This paper mainly presents an attempt to develop a numerical tool for easy and automated optimization of a tower structure with respect to its bending stiffness and self-weight. The analysis was provided for one specific type of tower (with accessories including an access and cable ladder, working platforms, and structures to support antennas) and simplified for finding the optimal distance between tower legs. The following results concerning the impact of the leg-to-leg distance and the quantity of telecommunications equipment were obtained using mainly (but not limited to) the Dynamo parametric design software, and the FEM Autodesk Robot engineering suite.

2. Experimental background

The key study used in the analyses presented in this paper is the experimental research on full-scale tower structures. The tests involved a forced loading of a structure assumed to simulate a horizontal wind load. One key result obtained from the research was the horizontal displacement of the tops of the structures (designated below as ux). Details on the tests, assumptions, and results can be found in [14, 15]. What follows is the information considered most important in the context of the final results and analyses.

Two tower structures of the same height with different bracing systems and, most importantly, with different leg-to-leg distances (3.40 m – tower no. 1; 4.54 m – tower no. 2) are shown in Fig. 1 and Fig. 2.

The differences between these values are notable: for tower no. 1 and the load of 60 kN, the displacement of the tower tops is about 90 cm. For structure no. 2 with the same load level, the deformation is about 38 cm, i.e. more than two times smaller. The reasons for such differences include a larger leg-to-leg distance of tower no. 2 and using a different bracing system. For tower no. 2, the legs are supported symmetrically from both sides, while for structure no. 1 only from one side. Failure modes of the towers can be found at: https://www.youtube.com/watch?v=XfaMDHFowzQ and https://www.youtube.com/watch?v=7SbSVfdZ42M. Considering the above, tower type no. 2 was selected for further numerical analyses.

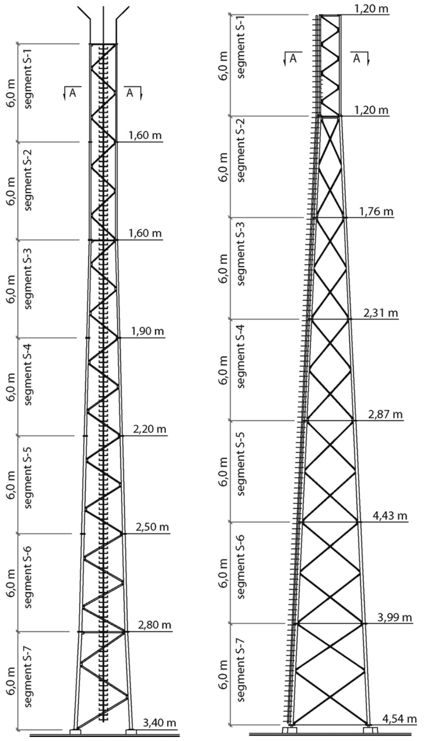

Fig. 1. Tower structures with different bracing systems

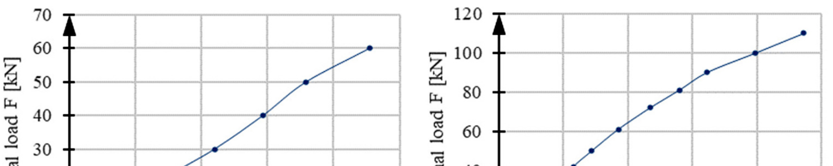

Fig. 2. Displacements of the tops of the tower structures subjected to full-scale tests: type 1 (left) and type 2 (right)

3. Tower structure

The analysed object was a lattice, spatial structure of a triangular cross-section (equilateral triangle) and a height of 62.0 m. The body consisted of 11 segments. The first 10 bottom segments (up to the 56th meter) have constant taper and form a truncated pyramid of side of

1.2 m at the top. The leg spacing was the variable in this case and varied between 3.6 and 8.1 m. The eleventh, top section forms a prism of the base side of 1.2 m. The scheme of the structure is shown in Fig. 3.

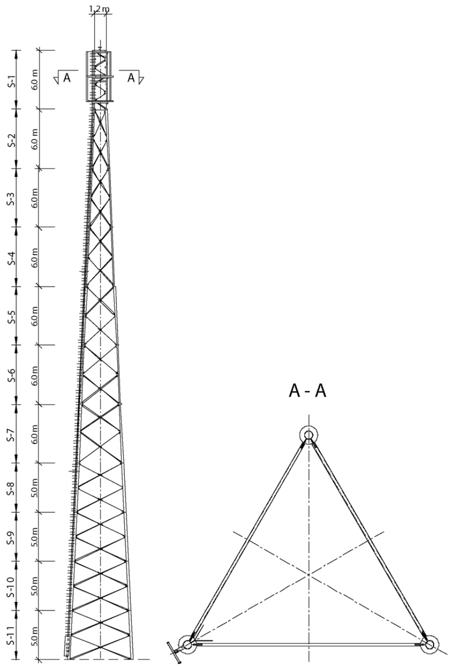

Fig. 3. The analyzed tower scheme

The legs of the tower as well as the elements of the bracing were made with cold-formed circular hollow sections. All structural elements have been made with the structural steel S 235. The bracing system for the walls in segments S-2 to S-11 is of type X, while the one for the

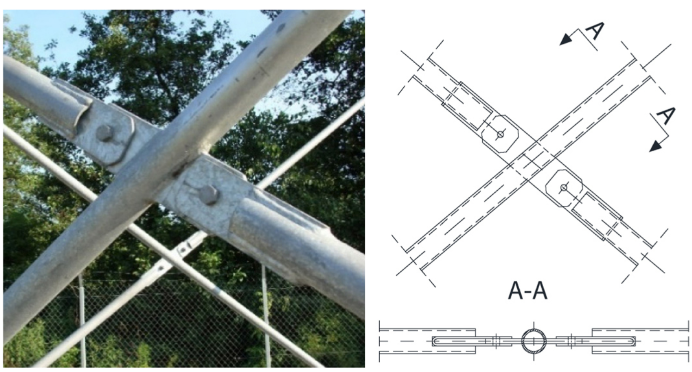

Fig. 4. Connection between bracing members

top segment of the tower is of single type. The cross braces are arranged axially so that one member, a circular hollow section, is continuous, while the other one crosses it via a gusset plate that runs through the center of the first member (Fig. 4).

Their attachment to the tower legs was realized as fork connections using a single bolt (a double shear connection), cf. Fig. 5.

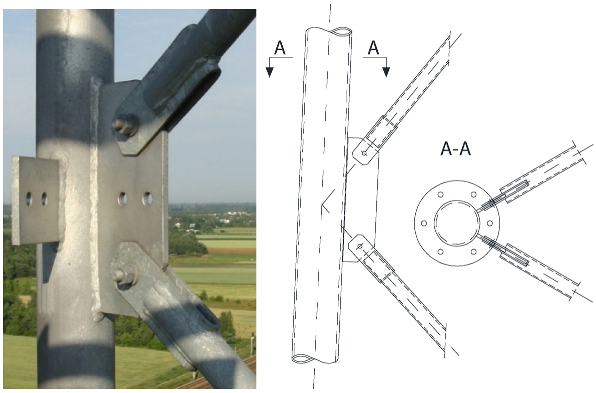

Fig. 5. Connection between bracing members

The legs at the ends of individual segments were joined using connecting flanges welded at their ends and a suitable number of bolts (6 bolts per connection), cf. Fig. 6.

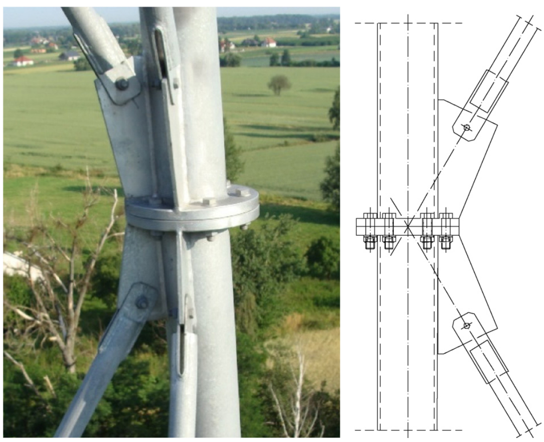

Fig. 6. Connection between tower legs

An access ladder is mounted on the outside of one of the tower legs, and a cable ladder is provided in the form of a slotted and Y-shaped flat mounted on the inside.

On the top of the structure (segment S-1) there are service platforms and support structures to which telecommunications equipment (sector antennas, microwave antennas, and remote radio units) is attached. In each case the telecommunications equipment, the support structures, and the service platforms are the same (in terms of their geometry, arrangement, and height above the ground level), and thus the loads that act on them, implemented in the calculation model as concentrated forces, are the same in each of the leg spacing cases considered. This is important due to the fact that in the case of a cantilevered structure, such as a lattice tower, actions exerted close to its top have the largest impact on cross-section forces occurring in its structural components, which is why these actions affect the results of the analysis to a large extent. Ancillaries mounted on the tower are listed in Table 1.

Table 1. Summary of telecommunications equipment on the tower

Height above the ground level	**Telecommunications equipment**	**Quantity**
61.1 m	Microwave antenna Ø0.6 m	1 item
	Radio units	5 items
60.3 m	Microwave antenna Ø0.6 m	1 item
59.0 m	Panel antenna	9 items
	Radio units	9 items
58.4 m	Radio units	3 items
58.0 m	Radio units	2 items

Service platforms and cross-sections of the upper part of the structure are shown in Fig. 7 and Fig. 8.

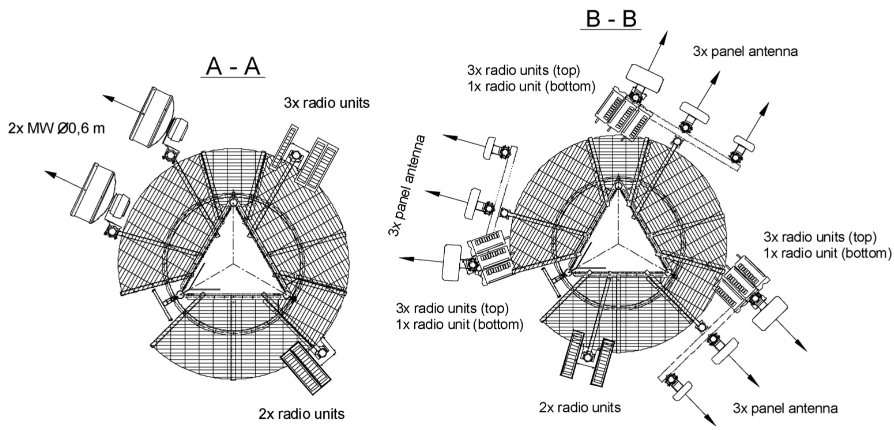

Fig. 7. The configuration of telecommunication devices and supporting structures in the analysis (cross-sections)

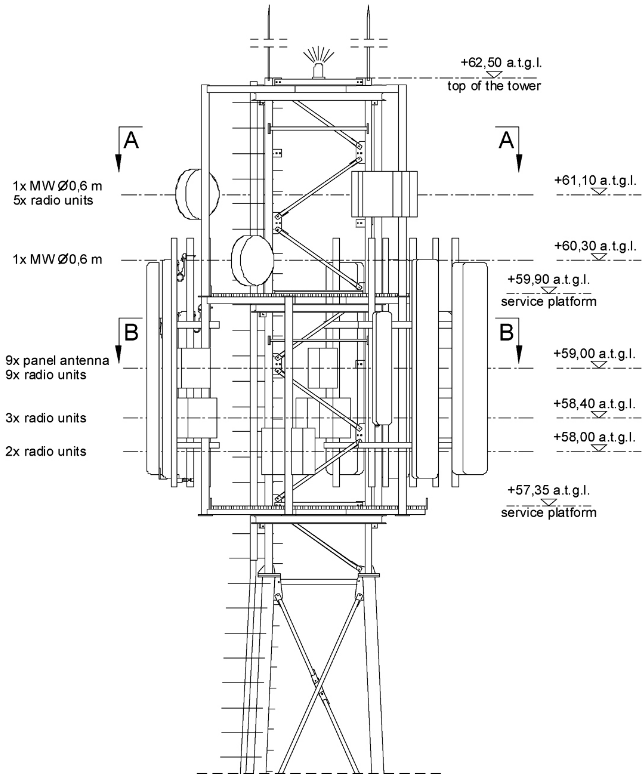

Fig. 8. The configuration of telecommunication devices and supporting structures in the analysis (front view)

4. Analysis

The wind action on the tower body and on equipment and support structures was calculated according to [16, 17], the buckling capacity was defined according to [18].

The main aim of the study was to analyze how the leg spacing at the tower base impacts wind loading, internal forces within structural members, and the natural frequency of the tower. To this end, a geometric model of the structure was first developed using Autodesk's Dynamo Sandbox software.

This software is used for parametric design [19] and is particularly suitable for complex structures whose geometry depends on multiple parameters. Working with the software involves creating custom algorithms and scripts that are linked with each other within a network of connections used to automate the design process and modify values of parameters defined as variables. A parameter can be the length of a bar or the number of bars. When a model has been built, other models with some different parameters can be quickly generated based on the original one.

Towers with leg-to-leg distances between 1.2 m and 8.4 m in steps of 0.3 m were built. Dynamo Sandbox is compatible with other Autodesk software solutions, so the geometric models could be exported to the Robot Structural Analysis Professional software and subjected to static and dynamic analysis.

A total of 25 models were produced, but only towers with leg spacing between 3.6 m and 8.1 m were finally analyzed. For each model, cross-sections were initially chosen, the wind load was calculated, and the effort of the legs was checked. As the leg spacing at the base and cross-sections of legs of the whole structure changed in each case, the wind load acting on the tower was calculated on a case-by-case basis. Values of the equivalent gust wind load acting on the tower (for simplicity designated as F) for wind direction 0° by the tower segment are shown in Fig. 9 and values of this load for wind direction 60° by the tower segment are shown in Fig. 10.

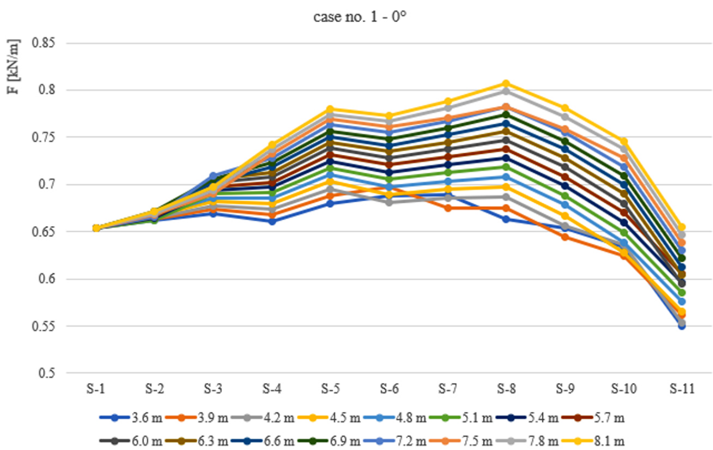

Fig. 9. Wind action for wind direction 0° by the tower segment

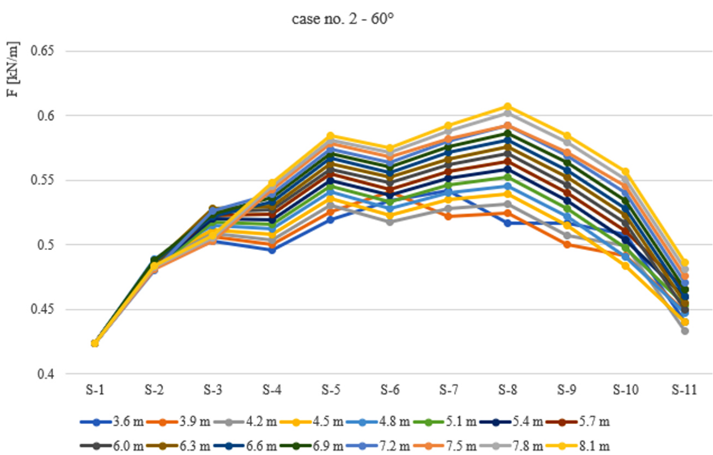

Fig. 10. Wind action for wind direction 60° by the tower segment

For a vast majority of the leg-to-leg distances at the tower base that were considered in the analysis it can be seen that the wind action on individual segments is increasing from S-1 (the tower top) to S-8, excluding S-6 (and S-4 for some spacing values), and decreasing from S-8 to S-11 (the tower base). The load also increased with an increase in the leg spacing at the base. Deviations from this trend can be found particularly with leg spacing of 3.6 m and 3.9 m. They occur when a change in the leg spacing at the tower base required a change in the cross-section height of legs in a given segment. The gust wind load acting on the tower for wind direction 0° was applied at each segment for two tower legs and for wind direction 60° for three tower legs (cf. Fig. 11 showing a model with leg spacing of 6.0 m). In the figure below, tower segment S-5 is marked. The loads are 0.74 kN/m for wind direction 0° and 0.56 kN/m for wind direction 60°.

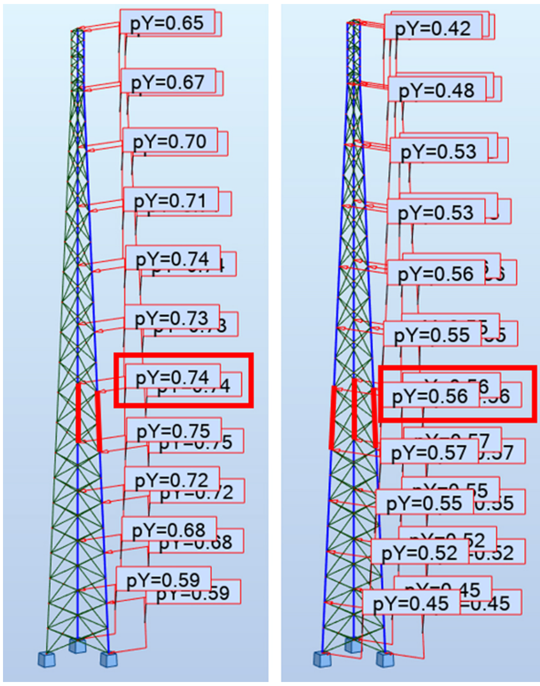

Fig. 11. The gust wind load acting on segments of tower legs for wind direction 0° (left) and 60° (right)

As already mentioned, after the leg spacing at the tower base was changed, leg cross-sections were initially chosen, wind load was calculated, and the effort of the cross-sections was calculated. The minimum and maximum effort of the legs was assumed to be 85% and 95% respectively. Table 2 lists cross-sections chosen for legs of each segment for the values of leg spacing at the tower base between 5.1 m and 6.6 m.

The optimization of the spacing between the legs, and thus their cross-sections, directly changes the self-weight of the structure. The summary of the tower self-weight for the models analyzed is shown in Fig. 12.

Table 2. Cross-sections for the considered values of leg spacing at the

	5.1 m	5.4 m	5.7 m	6.0 m	6.3 m	6.6 m
S-1	RO 88.9 × 4.5	RO 88.9 × 4.5	RO 88.9 × 4.5	RO 88.9 × 4.5	RO 88.9 × 4.5	RO 88.9 × 4.5
S-2	RO 108.0 × 4.5	RO 108.0 × 4.5	RO 108.0 × 4.5	RO 108.0 × 4.5	RO 108.0 × 4.5	RO 108.0 × 4.5
S-3	RO 139.7 × 5.0	RO 139.7 × 4.5	RO 139.7 × 4.5	RO 139.7 × 4.5	RO 139.7 × 4.5	RO 133.0 × 4.5
S-4	RO 139.7 × 7.1	RO 139.7 × 6.3	RO 139.7 × 6.3	RO 139.7 × 6.3	RO 139.7 × 5.6	RO 139.7 × 5.6
S-5	RO 168.3 × 6.3	RO 168.3 × 6.3	RO 168.3 × 5.6	RO 168.3 × 5.6	RO 168.3 × 5.6	RO 168.3 × 5.0
S-6	RO 168.3 × 8.0	RO 168.3 × 7.1	RO 168.3 × 7.1	RO 168.3 × 7.1	RO 168.3 × 6.3	RO 168.3 × 6.0
S-7	RO 193.7 × 7.1	RO 193.7 × 7.1	RO 193.7 × 6.3	RO 193.7 × 6.3	RO 193.7 × 6.0	RO 193.7 × 5.6
S-8	RO 219.1 × 6.3	RO 219.1 × 6.3	RO 219.1 × 6.3	RO 219.1 × 5.6	RO 219.1 × 5.6	RO 219.1 × 5.6
S-9	RO 219.1 × 7.1	RO 219.1 × 7.1	RO 219.1 × 7.1	RO 219.1 × 6.3	RO 219.1 × 6.0	RO 219.1 × 5.6
S-10	RO 219.1 × 8.0	RO 219.1 × 8.0	RO 219.1 × 7.1	RO 219.1 × 7.1	RO 219.1 × 7.1	RO 219.1 × 6.3
S-11	RO 244.5 × 7.1	RO 244.5 × 7.1	RO 244.5 × 7.1	RO 219.1 × 8.0	RO 219.1 × 7.1	RO 219.1 × 7.1

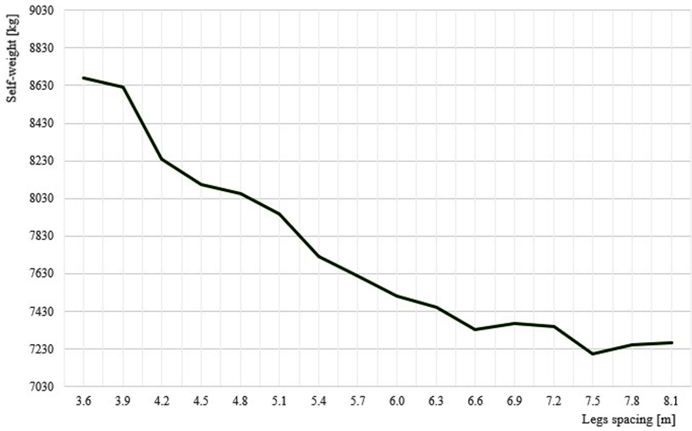

Fig. 12. Self-weight of the structure

5. Results

This Section summarizes the results of both the initial analysis of all 25 models and the final analysis of the models with the leg spacing between 3.6 m and 8.1 m.

5.1. Initial analysis

The initial analysis consisted in providing calculations for 25 models for two directions of wind action and for the leg spacing between 1.2 m and 8.4 m in steps of 0.3 m. For each value of leg spacing the structures were not redimensioned and their cross-sections were not changed.

Table 3 lists the leg cross-sections taken in each model considered and the displacements of the tower top as a function of the leg spacing at the tower base are shown in Fig. 13.

Table 3. Leg cross-sections

Segment	Cross-sections	Segment	Cross-sections
S-1	RO 88.9 × 4.5	S-7	RO 168.3 × 6.3
S-2	RO 139.7 × 4.5	S-8	RO 193.7 × 5.6
S-3	RO 139.7 × 4.5	S-9	RO 193.7 × 6.0
S-4	RO 139.7 × 5.0	S-10	RO 193.7 × 8.8
S-5	RO 168.3 × 5.6	S-11	RO 193.7 × 8.8
S-6	RO 168.3 × 5.6		

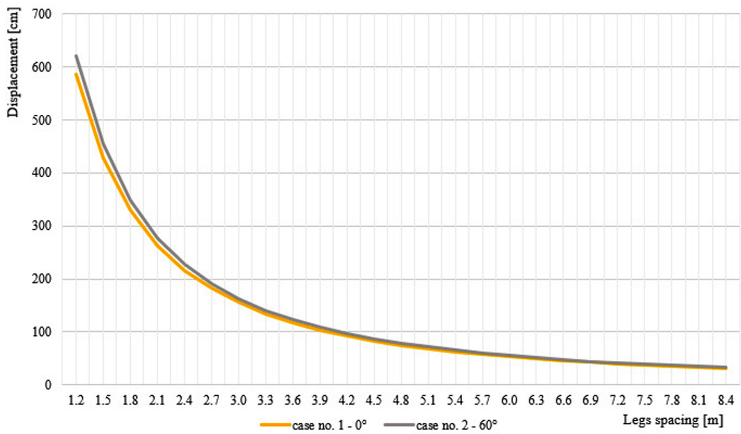

Fig. 13. Displacements of the tower top as a function of the leg spacing

It can be seen that although the leg spacing changes linearly, the curve of the tower top displacements is not linear. The displacements are decreasing as the leg spacing at the tower base increases. These changes are particularly significant for the spacing between 1.2 m and 3.0 m. Another characteristic is that the displacements differ only slightly for different wind directions.

5.2. Final analysis

The final analysis was limited to the leg spacing between 3.6 m and 8.1 m. Contrary to the initial analysis, for each distance between the legs their cross-sections were chosen so that their effort is between 85% and 95%. The displacements of the tower top as a function of the leg spacing at the tower base are shown in Fig. 14.

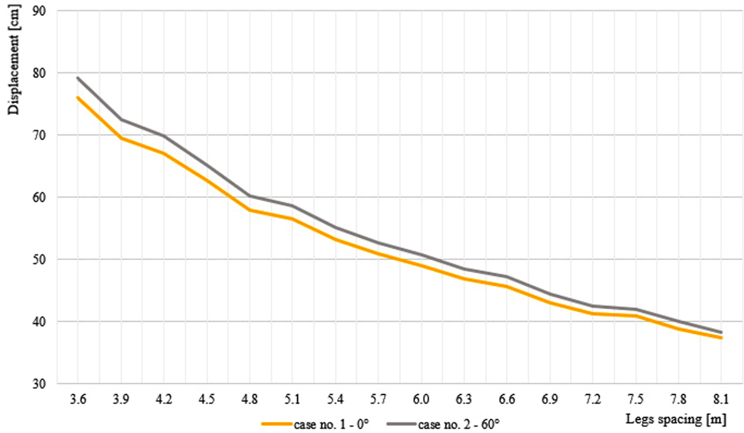

Fig. 14. Displacements of the tower top as a function of the leg spacing

As in the initial analysis, it can be seen that the curve of the tower top displacements is not linear and they tend to decrease. In this part of the study, the dynamic analysis of the tower was also carried out and values of the first natural frequency of the structure were determined; they are shown in Fig. 15.

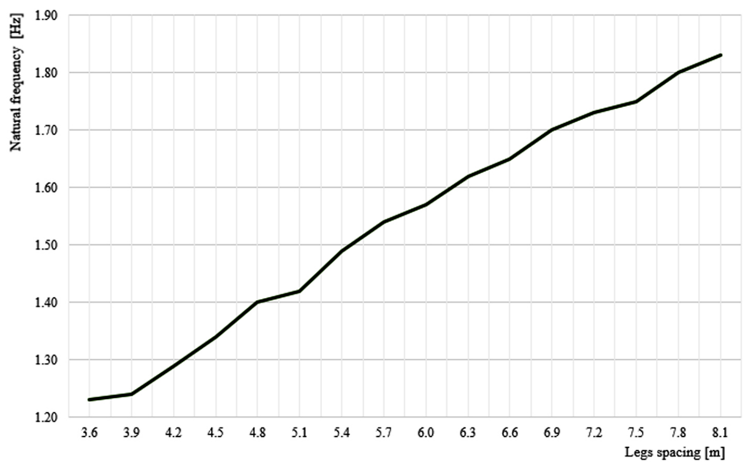

Fig. 15. First natural frequencies of the structures

These values are increasing as the leg spacing at the tower base in-creases. The curve is nearly linear. The increase in the natural frequency is closely related to the change in the stiffness and mass of the structure. The impact of the natural frequency on the wind action was ignored as it was negligibly small.

6. Conclusions

The goal of the analyses and results presented in the paper was to determine how the spacing between the legs of a steel lattice telecommunication tower (of a certain type and of a given height) affects three basic design parameters:

- The displacement of the top of the tower (Serviceability Limit State – SLS) resulting from bending stiffness of the structure right
- The natural frequency
- The self-weight of the structure.

The most important characteristic of the above results is that they were obtained for structures optimized with respect to the load-carrying capacity of its individual components (Ultimate Limit State – ULS). In this case the optimization parameter was the effort of each member and it was assumed to be between 85% and 95%. It can be therefore stated that all the values provided in the paper were determined for structures that were designed correctly and have some margin of load-carrying capacity, different for each member but falling into this range.

Basic conclusions are the following:

- The wind action on the structure: it can be seen that a larger leg spacing at the tower base results in an increased wind action (by about 15–20%), mainly due to the use of longer and wider bracing members directly enlarging the windward area.
- At the same time, the higher wind load (for larger leg spacing) does not result in larger displacements of the tops; on the contrary, the displacements tend to be smaller (see Fig. 12).
- If we consider a simplified condition of stiffness of a tower structure defined as limiting horizontal displacements of the top to 1/100 of the total height of the structure, we can see that this condition is satisfied for the leg spacing of 4.8 m and more.
- As far as engineering practice is concerned, the most crucial conclusion is that a larger distance between the legs has a positive impact on the self-weight of the structure (the larger leg spacing, the smaller self-weight and the smaller quantity of the material required).
- As the leg spacing increases, the self-weight drops (despite longer bracing members). Therefore, if we increase the spacing from 4.8 m to 6.3 m, for instance, the global stiffness of the structure will be higher (i.e. horizontal displacements will drop by about 12 cm), while the self-weight will drop by about 600 kg.
- It can be seen that an increase of capacity in terms of the Serviceability Limit State does not result in a higher use of material; as a matter of fact, it does the opposite.
- In addition, if the self-weight is smaller and the leg spacing is larger, the natural frequency increases (see Fig. 15), which should also be considered a positive consequence.

Therefore, it can be generally concluded that providing a larger distance between the tower legs while optimizing structural members with respect to their effort results in both higher stiffness and lower structure self-weight. Certainly, it is not possible to apply this conclusion to all types of telecommunication towers or other ranges of wind action, but the proposed research method provides promising results and can be used also in other cases.

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Wpływ rozstawu krawężników wież telekomunikacyjnych na ich sztywność na zginanie, ciężar całkowity i własności dynamiczne

Słowa kluczowe: dynamika, struktury kratownicowe, trwałość, normy projektowe, wieże telekomunikacyjne, obciążenie wiatrem

Streszczenie:

W artykule zaprezentowano analizy polegające na poszukiwaniu wpływu rozstawu krawężników kratowej, stalowej wieży telekomunikacyjnej na przemieszczenia wierzchołka konstrukcji, częstotliwość jej drgań własnych oraz ciężar własny obiektu. Krytyczną cechą uzyskanych wyników jest ta, że zostały one uzyskane dla konstrukcji zoptymalizowanych pod względem nośności jej poszczególnych elementów. Przyjęto, że stopień wytężenia powinien mieścić się w zakresie 85–95%. Kluczowym wnioskiem z punktu widzenia praktyki inżynierskiej jest ten, że zwiększony rozstaw krawężników wpływa pozytywnie na ciężar własny konstrukcji. Udowodniono relację: większy rozstaw – mniejszy ciężar własny wieży – mniejsze zużycie materiału. Zaproponowane metoda badawczo-obliczeniowa pozwala na stwierdzenie, że zwiększenie rozstawu krawężników przy jednoczesnej (najlepiej zautomatyzowanej) optymalizacji elementów konstrukcyjnych pod względem ich wytężenia, zarówno zwiększa sztywność na zginanie jak i zmniejsza wymagany ciężar własny konstrukcji.

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