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Analysis of guyed lattice masts considering various types of mast shaft geometrical imperfections

Analiza masztów o kratowych trzonach z różnymi imperfekcjami geometrycznymi

Abstract

The paper covers an analysis of guyed masts as per the second order theory. The influence of various types of mast shaft imperfections was analysed based on the example of a certain 200 m-high lattice guyed mast structure with 3 guy levels. The computations were performed taking into account the mast shaft with sway imperfections, imperfections associated with an offset of structure nodes and bow imperfections. As there are no guidelines concerning imperfections in the current European Standard EN 1993-3-1, the permissible assembly deviation values were taken as the reference point. Based on the obtained results of the analysis some final conclusions and comments have been formulated that may be useful in the design of mast structures.

Keywords: guyed masts, sway imperfections, bow imperfections, carrying capacity condition utilisation

Streszczenie

Praca dotyczy analizy masztów z odciągami w ujęciu teorii II rzędu. Na przykładzie pewnej konstrukcji kratowego masztu wysokości 200 m z 3. poziomami odciągów badano wpływ różnych form imperfekcji trzonu masztu na obliczenia statyczno-wytrzymałościowe masztu. W obliczeniach uwzględniono trzon masztu z imperfekcjami przechyłowymi, imperfekcjami związanymi z wzajemnym przesunięciem węzłów konstrukcji oraz imperfekcjami łukowymi. Z braku wytycznych normowych w aktualnej europejskiej normie EN 1993-3-1odnośnie do imperfekcji za punkt odniesienia przyjęto wartości imperfekcji równe dopuszczalnym odchyłkom montażowym. Na podstawie uzyskanych wyników analizy sformułowano pewne wnioski końcowe i uwagi, które mogą być przydatne w projektowaniu konstrukcji masztów.

Słowa kluczowe: maszty z odciągami, imperfekcje przechyłowe, imperfekcje łukowe, wytężenie

1. Introduction

In accordance with current standard guidelines [5], guyed masts using elastic global analysis as per second order theory should be computed. The internal forces in the structure elements must, therefore, be related to the final, deformed configuration of the structure. Perfect mast structures are idealised. In real member structures there are always some imperfections. The most frequently occurring imperfections in steel member structures include transverse and eccentric loads, initial bar curvature and various structural defects, eg. own stresses [12]. In structures with imperfections, according to the load increase, displacements increase in a strictly defined way, dictated by imperfective factors. Therefore, it is important to adopt proper imperfection form. Unfortunately, in the current standard regulations there are no guidelines pertaining to the form of adoption of the initial imperfections and their magnitude in guyed mast computations. There is also no information that such imperfections can be omitted from computations. The community of scientists and designers interpret this lack of guidelines pertaining to imperfections differently. According to some people, keeping the tolerances recommended in Annex F to the standard [5] fully justifies the analysis of the mast shaft with ideal geometry. According to others, the lattice mast shaft should be treated as a uniform built-up column, for which, as per [4], bow imperfections between the fixing levels featuring the maximum amplitude value equal to $L/500$ (L – span length of mast shaft). Assumption of the mast shaft imperfections as for a uniform build-up column appears, in the author's opinion, to be a simplification. Bow imperfections with maximum amplitude $L/500$ pertain to a member with hinged ends because this assumption corresponds to its buckling form. In guyed masts, due to the geometric non-linearity of the guys, the stiffness values of elastic supports in guy fixing points are characterised by wide variation and depend on the current mast configuration. These supports differ, therefore, from typical steel structure supports, and so the buckling form of the mast shaft is also different than in typical steel structures [8].

In this study, based on the example of a specific mast structure, a comparative analysis of the "perfect" type mast shaft and the "imperfect" type mast shaft was performed. However, due to the lack of standard guidelines, the permissible assembly deviations values, specified in Annex F of the standard [5], were taken as the reference point. The most important of the listed assembly tolerances include:

A) The final position of the centre line along the entire mast height should fall within a vertical cone with the radius of $H/1,500$ at the top of mast (H – mast height);

B) After erection, the tolerance on the alignment of 3 consecutive guy connections on the shaft is limited to $(L_1 + L_2)/2,000$ (L_1, L_2 – lengths of the two consecutive spans of the shaft)(according to [2] – deviation from the vertical of any two structure points should not exceed 0.05%);

C) Maximum initial deflection of the mast shaft between two guy levels should be $L/1,000$ (L – distance between the guy levels).

The main objectives of the analysis are an attempt to address the question of which imperfections affect the increase of the internal forces in the mast elements and how the consideration of imperfections affects the effort of the mast shaft member and the effort of the guys.

2. Mast structure description

The subject of the analysis is a 200 m high lattice guyed mast (Fig. 1). The mast shaft was designed as S355 steel in form of trihedral space truss with side width a = 2.5 m. As legs, member Ø 168.3/14.2 mm pipes were used, and as mast face lacing – pipes Ø 76.1/4 mm (bolted connections between bracing members and leg members). The mast guys were fixed at heights of 65.0 m, 125.0 m and 175.0 m. For the guys, spiral strand steel ropes 1 x 61, Ø 32 mm, rope grade R_r = 1,570 MPa and a minimum breaking force of 823.0 kN were used. The assumed modulus of elasticity of ropes E was equal to 160 GPa. The assumed values of initial guy forces at all guy levels are the same and amount to 75 kN (according to [5] – they do not exceed 10% of the rope breaking force).

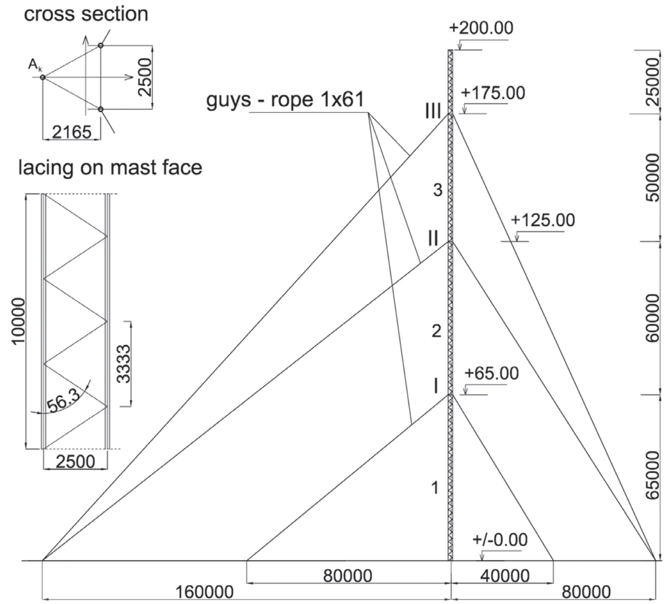

Fig. 1. Mast structure diagram

3. Actions and analysis area

The assumption was made that the mast would be located in the 2nd wind load zone for the territory of Poland as per [3], in the second category arena. The value of the base wind pressure for this zone is 420 Pa. The mast structure has been qualified to the second reliability class for which the partial factors for permanent loads are equal to 1.1 and for variable loads 1.4. Schemes of the mast wind load were determined in accordance with [5] (Fig. 2).

The wind load acting on mast structure consists of the mean load acting on the entire height of the structure and a number of the patch loads acting only on some fragments of the structure. The structure's own weight (excluding its outfit) and wind loading for the two most unfavourable directions of wind action $W1$ and $W2$ were considered in the analysis (Fig. 2).

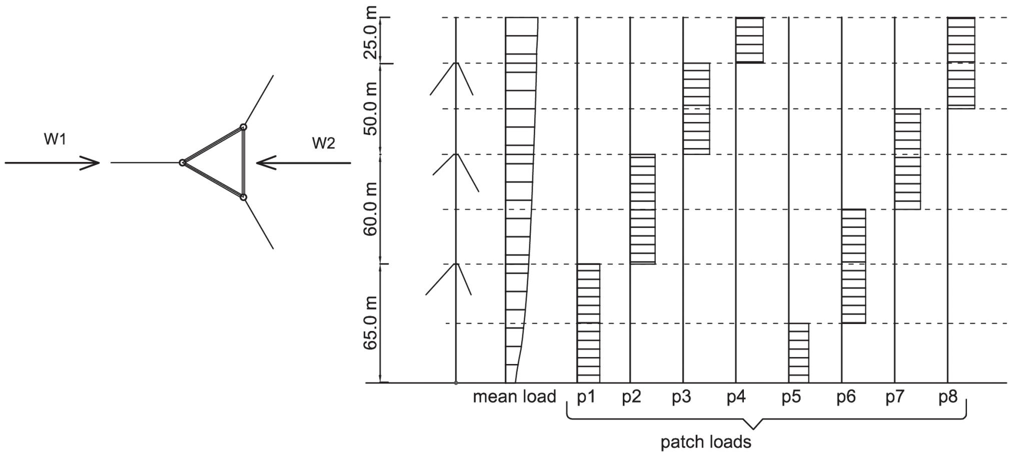

Fig. 2. Wind load diagrams and wind directions

The computations were performed for the "perfect" type mast shaft and the structure with imperfections whose forms and values were taken into account based on the assembly tolerance values described in Annex F of the standard [5]:

A) The maximum deflection from vertical of the mast shaft top by the value $R = H/1{,}500 = 133$ mm (Fig. 3);

B) After erection the tolerance on the alignment of 3 consecutive guy connections on the shaft should not exceed the value $(L_1 + L_2)/2{,}000 = (65{,}000 + 60{,}000)/2{,}000 = 62.5$ mm value, and according to [2], deviation from the vertical of any two structure points should not exceed 0.05%, i.e., at the first level of guy connections, the deviation of the mast shaft from the vertical the value of $65{,}000 \cdot 0.05\% = 32.5$ mm, at the second level $60{,}000 \cdot 0.05\% = 30$ mm, and at the third level $50{,}000 \cdot 0.05\% = 65$ mm (Fig. 4);

C) Maximum initial deflection of the mast shaft between two guy levels should not exceed $L/1{,}000$. The proper form of bow imperfections in the analysed guyed mast example was adopted in accordance with the determined first buckling form of the mast shaft with the assumption of the maximum buckling value equal to $L/1{,}000 = 65{,}000/1{,}000 = 65$ mm (Fig. 5) [7, 8]. The values of displacements of the other nodes were determined proportionally, in accordance with the displacement eigenvector coordinates based on the mast shaft stability analysis.

The static computations from the second order theory perspective, with consideration of the non-linearity of the guys, were performed using the *Mast1* software described in [11]. Finally, mast computations for 126 various load combinations were conducted. The values of the internal forces of the mast structure elements – both for the perfect structure and the imperfect structure – were computed, in accordance with [5], as the total load effect:

$$S_{TM} = S_M \pm S_P \qquad (1)$$

where:

S_M – the mean wind load effect,
S_P – the total patch load effect.

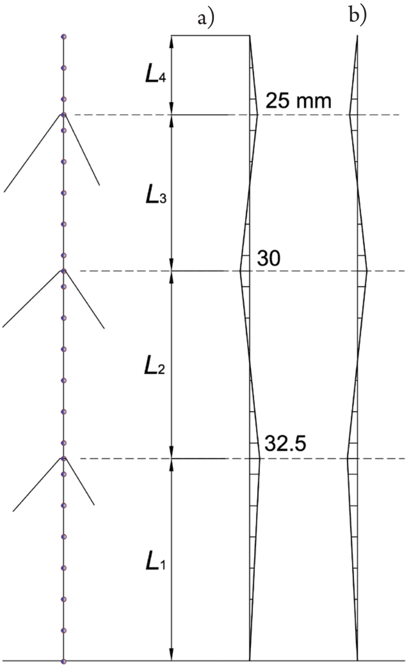

Fig. 3. Adopted mast shaft imperfections – type A): a) in one direction, b) in the opposite direction

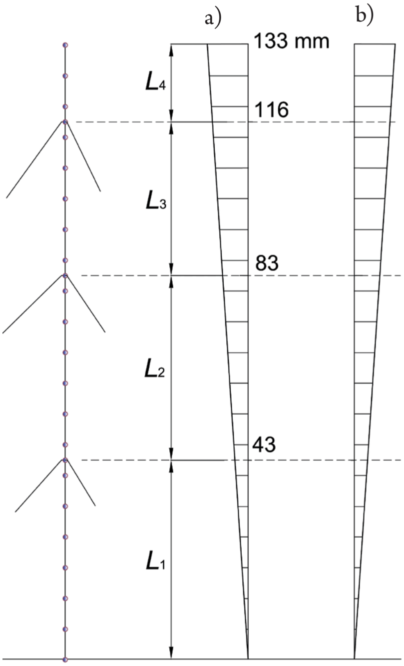

Fig. 4. Adopted mast shaft imperfections – type B): a) in one direction, b) in the opposite direction

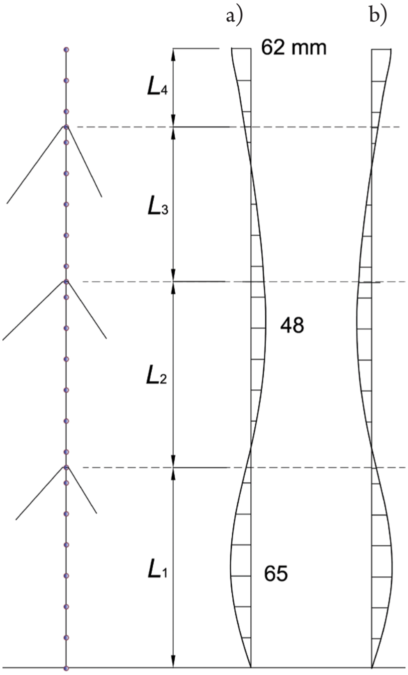

Fig. 5. Adopted mast shaft imperfections – type C): a) in one direction, b) in the opposite direction

The total patch load effect, S_p, is calculated from the formula:

$$S_p = \sqrt{\sum_{i=1}^{N} S_{PLi}{}^2}\,, \tag{2}$$

in which S_{PLi} – the effect of i-th wind patch load, N – the total number of patch load schemes required in the calculations.

Due to the fact that the values of the internal forces of the mast elements don't depend directly on the load acting on the mast, but are the result of laborious calculations, the substitute model of the mast shaft (beam-column model) can be successfully used in the analysis. The authors of papers [9, 10, 12, 13] have shown a good convergence of mast structure computations with the assumption of the beam-column model and the frame-truss mast shaft model with a continuous members of legs, modelled as 3-D beam elements and hinged connected to the legs with bracing members, modelled as 3-D truss elements.

4. The analysis results

Selected results of the static-strength computations of mast structures elements are presented in tabular form. Since the computations were made taking the substitute beam--column model of the mast shaft, the normal forces in the leg elements have been determined as $N_{Ed} = N_1 + N_2$, where $N_1 = N/3$ and $N_2 = M/h$ (in the case of a positive value of bending moment) or $N_2 = M/2h$ (in case of a negative value of bending moment), N, M – design values of normal forces and bending moments in the mast shaft members, h – the height of the triangular section of the mast shaft ($h = a\sqrt{3}/2$, a – axial spacing between legs members).

The maximum value of the shear force for the perfect type mast shaft structures was V_{Ed} = 49.3 kN. The value increased by less than 6% only in the case of including the bow imperfections in computations, as shown in Fig. 5. In other cases of imperfections, the differences didn't exceed 1%.

Table 1. The maximum values of normal forces in the leg member of the mast shaft N_{Ed} [kN] ($W1$, $W2$ – wind direction; a), b) – according to Figures 3, 4, 5)

Location	**Perfect type of structure**	**Imperfections A**	**Imperfections B**	**Imperfections C**
Level 0	–379.4 ($W1$)	–379.5 ($W1$a)	–379.4 ($W1$a)	–379.4 ($W1$a)
Span 1	–605.7 ($W1$)	–606.1 ($W1$b)	–608.5 ($W1$a)	**–634.9** ($W1$b)
Support I	–507.6 ($W2$)	–508.2 ($W2$a)	–513.1 ($W2$a)	–509.0 ($W2$a)
Span 2	–442.5 ($W1$)	–442.9 ($W1$b)	–445.1 ($W1$b)	–455.6 ($W1$a)

Support II	–498.6 (*W*2)	–498.9 (*W*2a)	–504.8 (*W*2b)	–507.5 (*W*2a)
Span 3	–391.9 (*W*2)	–392.1 (*W*2b)	–396.1 (*W*2b)	–397.1 (*W*2a)
Support III	–356.4 (*W*2)	–356.5 (*W*2a)	–334.5 (*W*2b)	–357.7 (*W*2a)
Cantilever	–383.8 (*W*2)	–384.1 (*W*2a)	–384.2 (*W*2a)	–384.4 (*W*2a)

According to [5], in the computations as per the second order theory, checking the mast shaft strength comes down to checking the load capacity condition for a single leg member taking into account its buckling between the bracing member nodes:

$$N_{Ed} / N_{b,Rd} \leq 1. \tag{3}$$

The design buckling resistance of the compression leg member shall be defined from the formula:

$$N_{b,Rd} = \chi \cdot A_1 \cdot f_y / \gamma_{M1}, \tag{4}$$

where:

$A_1 = 68.7\ \text{cm}^2$ – leg member cross-section area,
$\chi = 0.798$ – reduction factor corresponding with the leg member buckling length $L_1 = 333$ cm,
$\gamma_{M1} = 1.0$ as per the Polish National Annex, therefore $N_{b,Rd} = 1{,}945.0$ kN.

Percentage utilisation of the end condition of the analysed mast in spans and on supports – the guy fixing points, are presented in Table 2.

The computed maximum values of the mast guys at particular levels and the percentage ultimate limit state utilisation of the guys are presented in table 3.

As the reader can see, consideration in the computations of the sway imperfections (Fig. 3) and imperfections associated with an offset of structures nodes (Fig. 4) has practically no effect on the change of internal forces in the mast structure elements. In the case of computations of the mast with bow imperfections (Fig. 5) the increase of the leg member effort applies only to the first span and amounts to approx. 2%, due to the increase of the computed bending moments in the span by almost 12%.

Table 2. Percentage of leg member carrying capacity condition utilisation

Location	**Perfect type of structure**	**Imper-fections A**	**Imper-fections B**	**Imper-fections C**
Level 0	20	20	20	20
Span 1	31	31	31	**33**
Support I	26	26	26	26

Span 2	23	23	23	23
Support II	26	26	26	26
Span 3	20	20	20	20
Support III	18	18	17	18
Cantilever	20	20	20	20

Table 3. The maximum force values in the guys F_{Ed} [kN] and the percentage ultimate limit state utilisation

Guy level	Perfect type of structure		Imperfections A		Imperfections B		Imperfections C	
	F_{Ed}	F_{Ed}/F_{Rd}	F_{Ed}	F_{Ed}/F_{Rd}	F_{Ed}	F_{Ed}/F_{Rd}	F_{Ed}	F_{Ed}/F_{Rd}
I	181.5	33	181.6	33	182.3	33	182.9	33
II	180.5	33	180.7	33	181.0	33	181.6	33
III	219.8	40	220.1	40	220.0	40	220.2	40

5. Final conclusions

The influence of various types of mast shaft imperfections on the values of internal forces and the effort of mast structure elements has been analysed in the paper on the example of a particular guyed mast structure. Due to the lack of any standard guidelines as to the form of imperfections and their values, the mast shaft has been analysed subjected to sway imperfections, with the maximum value on the top of the mast equal to $H/1{,}500$, imperfections associated with non-axial positioning of the mast structure nodes, shifted against each other by the maximum $(L_1 + L_2)/2{,}000$, and bow imperfections, with maximum amplitude $L/1{,}000$. In the latter case, the appropriate curvilinear mast shaft shape was found based on the stability analysis of the mast shaft in the prestress condition and supported at the guy fixing points, where the stiffness of the elastic supports based on the values of initial guy forces have been determined. In the guyed mast analysed, as in the case of the mast structures described in [8], the zero-value points of horizontal displacements are located beyond the guy fixing points (Fig. 5). It has been shown that the sway imperfections and imperfections associated with an offset of structure nodes have practically no effect on values of internal forces and the effort of the mast structure elements. A slight influence on the increase of the span bending moments was noted only in the case of bow imperfections which had an effect on a slight increase of the effort of the mast shaft legs. Similar results were observed by the author in the case of analysis of other mast structures, characterised by a greater mast shaft slenderness [6, 7]. The increase of the effort of the mast span legs in any cases did not exceed 10%.

The relatively small differences observed in the results of computations of the mast structure with bow imperfections compared with computations of the "perfect" mast structure, originate from the specific properties of mast structures characterised by large (even several metre) nodal displacements of the mast shaft caused by external loads. The situation is quite different in typical steel structures where nodal displacements caused by load are of the same order as the assumed initial imperfection values.

Due to the extremely laborious computations of mast structures considering mast shaft geometrical imperfections and the very slight effects of these imperfections, one could, in the author's view, omit the imperfections in the analysis, and with a careful approach, in calculating the design buckling resistance of the compression leg members of the mast, in view of model uncertainty, reduce the buckling resistance by 10%, which will result in the design of mast structures with an appropriate level of safety.

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