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TELESCOPIC JOINTS IN STEEL TUBE TOWERS

Modeling problems of new generation of steel shell towers supporting overhead power transmission lines and having telescopic joints are presented in this paper. The towers of such structure, designed according to the European standard EN 50341 [8], ensure high reliability even when subjected to high technological and climate loads. In this paper elements of differentiated reliability requirements are verified, and special attention is paid to the values of variable loads coefficients for different reliability classes of the considered structure. Using computer modeling tools and linear, elastic shell theory, the modeling error of telescopic joints in a sample tower supporting overhead power transmission line rated at 110 kV is estimated.

Keywords: columns, shells, power lines, bearing capacity, reliability, telescopic joint

1. Introduction

1.1. Types of towers supporting overhead power transmission lines

Overhead power transmission lines, rated for the voltage of 110 kV and up to 400 kV, are usually supported by transmission towers having lattice structure. The precise shape of the supporting structure depends on the parameters of the supported line, including the spacing of the phase conductors and grounding wires, economic considerations and the requirements of environmental protection. The lattice supporting structures are often characterized by large dimensions, and especially built up horizontal size, due to the required stiffness of the whole structure.

In the urbanized areas, as well as in the areas with difficult access, such as near shore and on steep slopes the steel tube supporting towers characterized by compact contour, much smaller than the contour of lattice towers, seem to be a much better solution. Regarding the purpose and specific application the towers of both types may be divided into the following groups: tangent (P),

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angle (N), tangent strain (O), angle strain (ON), dead end (K), branching (R) and crossover (S). The functions of all tower types are described in detail, among others, in monograph [3].

The steel shell towers offered in Poland cover typical structures, designed to support overhead power transmission lines rated for the voltage of 110 kV (cf. the catalogue [2]). An assortment of towers supporting one or two circuits and belonging to various series is listed in this catalogue. Depending on the foreseen application, the following two types have been designed: tangent (P) and strong (M2), (M4), (M6), (M9), (K) and (KG). With the catalogue heights of 15.00 m to 25.00 m and span lengths of 250.00 m to 400.00 m in the 1st wind and icing climate zone these towers satisfy the reliability requirements of the Eurocode PN-EN 1990 [4], including the specification of load and bearing capacity coefficients listed in the code PN-EN 50341-2-22:2016 [8]. The fitting elements of these towers, exceeding the commercially available sheet metal lengths, are assembled on site with telescopic joints (cf. fig. 1).

Fig. 1. Assembly of the steel shell tower trunk

The steel shell towers supporting overhead power transmission lines, in spite of modern structural solutions and warranted durability, should not be treated as competition of the lattice towers, but rather as an important supplement of the commercial offer available in Poland.

1.2. Reliability elements of steel supporting structures

Elements of reliability for the steel structures supporting overhead power transmission lines, including the partial components of: loads γ_F and bearing capacity γ_M present in the version of limit state method adopted in Eurocodes may be found in several codes. Especially the general rules specified in Eurocode PN-EN 1990 and the following parts of Eurocode 3: PN-EN 1993-1-1 [5], PN-EN 1993-1-6 [6], PN-EN 1993-3-1 [7] (lattice supporting structures) and PN-EN 1993-3-2 (shell supporting structures) are to be reckoned with. In addition the European standard PN-EN 50341-2-22:2016 [8] deals with the operating conditions of overhead power transmission lines, specifying the reliability components corrected with respect to the general requirements, and oriented exclusively on this group of steel structures. Analysis of the requirements listed in the codes enumerated above indicates that the system of load and bearing capacity coefficients based on Eurocode 3 and European standard PN-EN 50341-2-22:2016 is incoherent. This is documented in Table 1 and Table 2.

Table 1. Partial coefficients for permanent G and variable: wind W and ice I actions

Result of Actions	**Reliability Class RC**	**Permanent Actions *G***	**Variable Actions**	
			Wind *W*	**Icing *I***
(1)	(2)	(3)	(4)	(5)
coefficients γ_F according to PN-EN 1993-3/1 and 1993-3/2				
disadvantageous	3	1.2	1.6	
	2	**1.1**	**1.4**	
	1	1.0	1.2	
advantageous	1, 2 i 3	1.0	0	
coefficients γ_F according to PN-EN 50341-2-22:2016				
disadvantageous	3	1.0	1.4	1.5
	2	1.0	1.2	1.25
	1	1.0	1.0	1.0
advantageous	1,2 i 3	1.0	0	0
coefficients γ_F according to own research				
disadvantageous	3	1.1	1.5	
	2	**1.1**	**1.4**	
	1	1.1	1.3	
advantageous	1,2 i 3	1.0	0	

The partial coefficients of limit state method according to the Eurocode PN-EN 1990 are expressed as product of two components covering the random influences (coefficients γ_f for loads and γ_m for bearing capacity) and modeling errors (coefficients γ_{Sd} for loads and γ_{Rd} for bearing capacity):

$$\gamma_F = \gamma_f \gamma_{Sd}, \quad \gamma_M = \gamma_m \gamma_{Sd} \tag{1}$$

Initial specifications regarding the range of values the load coefficients may assume may be found in PN-EN 1993-3/1 and 1993-3/2 for the reliability class RC 2, these values are listed in bold in the Table 1. The values assumed in the abovementioned codes may be derived from the specifications listed in PN-EN

1990 [4], Table A1.2(B), when smaller modeling errors γ_{Sd} of constant and variable loads acting on supporting structures of overhead power transmission lines, with respect to other building structures, are accounted for. Such interpretation is doubtful in the case when load coefficients are specified for RC 2 class structures according to the code PN-EN 50341-2-22:2016 (reduction of coefficients: γ_G from 1.15 to 1.00 and γ_Q form 1.50 to 1.20÷1.25 is not justified).

Differentiation of reliability in both standard approaches raises serious doubts. The value of load coefficient may not be assumed arbitrarily, but has to be justified statistically, with proper analytical formulae (cf. [1]). In the basic case the reliability condition for each structure describes the relationship, in which computational values of bearing capacity R_d and load effects E_d are compared:

$$R_\mathrm{d} = \overline{R} - \beta_\mathrm{R}\mu_\mathrm{R} \geq E_\mathrm{d} = \overline{E} + \beta_\mathrm{E}\mu_\mathrm{E} \tag{2}$$

Specifications of partial coefficients attributed to the bearing capacity γ_M for reliability classes other than RC 2 may be derived by application of correctional factor K_R, having the form:

$$\overline{R} - 0.8\,\beta_\mathrm{RC2}\mu_\mathrm{R} = K_\mathrm{R}(\,\overline{R} - 0.8\,\beta_\mathrm{RC}\mu_\mathrm{R}\,) \tag{3}$$

to the left hand side of the formula (2), and thus

$$K_\mathrm{R} = \frac{1 - 0.8\beta_\mathrm{RC2}\mathrm{v}_\mathrm{R}}{1 - 0.8\beta_\mathrm{RC}\mathrm{v}_\mathrm{R}} \tag{4}$$

where $\mathrm{v}_\mathrm{R} = \mu_\mathrm{R}/\overline{R}$ – variable random material strength coefficient.

Analogous explanation may be made for variable loads Q (characterized by the average value of $\overline{Q}$ and standard deviation μ_Q), considered at the right hand side of formula (2), by introduction of the correctional factor K_Fi:

$$K_\mathrm{Fi}\,(\,\overline{Q} + 0.7\,\beta_\mathrm{RC2}\mu_\mathrm{Q}\,) = \overline{Q} + 0.7\,\beta_\mathrm{RC}\mu_\mathrm{Q} \tag{5}$$

$$K_\mathrm{Fi} = \frac{1 + 0.7\beta_\mathrm{RC}\mathrm{v}_\mathrm{Q}}{1 + 0.7\beta_\mathrm{RC2}\mathrm{v}_\mathrm{Q}} \tag{6}$$

where $\mathrm{v}_\mathrm{Q} = \mu_\mathrm{Q}/\overline{Q}$ – random load Q variability coefficient.

For the structures belonging to RC 3 class, designed for the sample reference period of T = 50 years, the reliability coefficients according to the Table B2 of the code [4] are equal to: β_RC2 = 3.8 and β_RC = 4.3, respectively; thus the formulas (4) and (6) for bearing capacity and load reduction coefficients depend only on corresponding variability coefficients:

$$K_R = \frac{1-0.8\cdot 3{,}8\,v_R}{1-0.8\cdot 4{,}3\,v_R} = \frac{1-3.04\,v_R}{1-3.44\,v_R}\,, \tag{7}$$

$$K_{Fi} = \frac{1+0.7\cdot 4.3\,v_Q}{1+0.7\cdot 3.8\,v_Q} = \frac{1+3.01\,v_Q}{1+2.66\,v_Q}\,. \tag{8}$$

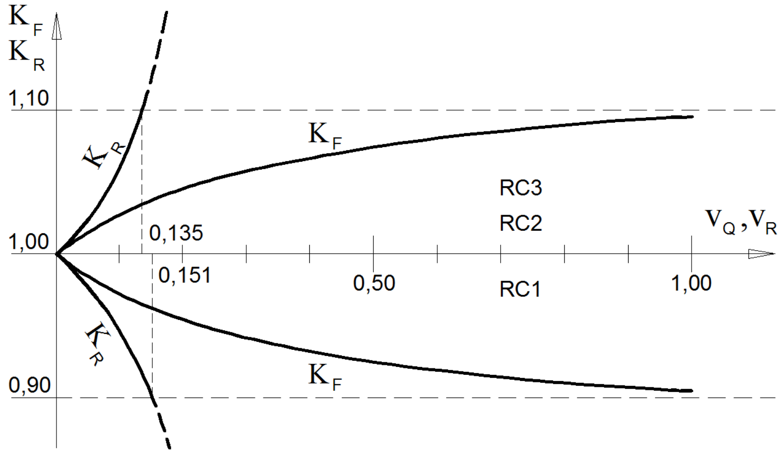

Fig. 2. Graphs of reduction coefficients K_R and K_{Fi} according to the formulas (7)÷(10), cf. [1]

For the RC 1 class, the reliability coefficient, according to the Table B2, is equal to β_{RC} = 3.3; thus the formulae (4) and (6) for reduction coefficients have the following form:

$$K_R = \frac{1-0.8\cdot 3.8\,v_R}{1-0.8\cdot 3.3\,v_R} = \frac{1-3.04\,v_R}{1-2.64\,v_R} \tag{9}$$

$$K_{Fi} = \frac{1+0.7\cdot 3.3\,v_Q}{1+0.7\cdot 3.8\,v_Q} = \frac{1+2.31\,v_Q}{1+2.66\,v_Q} \tag{10}$$

The graphs of reduction coefficients as a function of material strength variability coefficient v_R and variable load variability coefficient v_Q are depicted in fig. 2. In view of the above results, the differentiation of the reliability requirements set for building structures according to the recommendations of the code PN-EN 1990 is fully justified and safe (K_R = 1 for RC 2 class and correction of the load coefficients γ_Q for classes RC 1 and RC 2 by the correction factors K_{FI} having the values listed in Table 2).

Supporting structures of the power transmission lines are designed to be made using unified range of the rolled products: angle irons or plates, which are characterized by small negative thickness tolerances. Safe estimation of constant loads G is a result of designing for nominal dimensions, this justifies the simplifications in formulas (8) and (9): $v_G = v_Q = 0$, as well as $K_{FG} = 1.0$. Icing and wind are characterized by substantial variability, thus a correction of the load coefficients for variable loads according to the Table 2 is justified.

Table 2. Values of the K_{FI} coefficients for actions according to PN-EN 1990 [4]

Correction coefficient	**Reliability class**		
	RC1	RC2	RC3
(1)	(2)	(3)	(4)
K_{Fi}	0.9	1.0	1.1

Analyzing the data presented in the Table 1 one may observe, that the ratio of wind and icing load coefficients γ_Q for various reliability classes is not preserved, both for recommendations contained in the code PN-EN 50341-2-22:2016 as well as codes PN-EN 1993-3/1 and 1993-3/2. Graphs depicted in fig. 2 show the limiting values $0.9 \leq K_{Fi} \leq 1.1$. Our own specifications, rounded down to 0.1 are in perfect agreement with analytical results.

1.3. Numerical analysis of bearing capacity for shells with telescopic joints

Shell model is usually assumed to analyze the bearing capacity of steel towers with telescopic joints.

Static linear (LA) analysis is the simplest method of analysis available, with simple linearly elastic material model and contact phenomena accounted for. The interaction between two independent, but adjoining shells is ensured by the tools available in the FEM computational environment. Contact is defined on the common boundary of the shells. The displacements in all three directions of the orthogonal coordinate system are restricted at the bottom edge of the joint. The nodes located at the top edge of the upper shell assembly are usually connected with rigid elements. This approach does not perfectly correspond to the real behavior of the structure, but if proper care is taken when the length of the modeled section is determined the errors induced may be limited. The same applies to the fixing of the lower shell assembly.

The numerical simulations performed allow for easy determination of the correct tower section length, at which it is possible to obtain reliable results of calculations. Quadrilateral shell elements are selected for analysis, as these elements are very convenient in modeling the regular geometry of a shell having polygonal cross-section and shall correctly model the performance of the shell subjected to loads. Two finite elements of this type, namely CQUAD4 and CQUAD8 are available in NASTRAN solver. These are isoparametric elements, taking into account the interaction of membrane and bending stiffnesses, but differing in the number of nodes in each element. According to experience, with the same number of nodes in the whole structure the CQUAD8 elements yield better quality results in static analysis.

The contact between touching surfaces in the linear analysis is accomplished via contact surfaces, and in the nonlinear analysis – via linear gap elements. Static and dynamic friction coefficients may be defined. Finite elements of the CGAP type generate three possible states on the boundary between two touching surfaces:

- relative sliding of the surfaces, when the friction coefficient is equal to zero,
- static pressure between the surfaces, when the tangent force between the surfaces is lower than the maximum static friction force,
- relative sliding of the surfaces, when the tangent force between the contacting surfaces exceeds the maximum value of the static friction F_{max}:

$$F_{max}= \mu F_N, \tag{11}$$

where: μ – static friction coefficient,
F_N – force normal to the contacting surfaces.

Friction coefficient for steel surfaces subjected to hot dip galvanization is assumed to be in the range of 0.1÷0.3, depending on whether the structure is loaded primarily in a static or a dynamic manner. The loads acting on the supporting structures of overhead power transmission lines are predominantly of static character, thus the upper bound of the range may be assumed. This increases the bearing capacity of the telescopic assembly joint.

2. An example of telescopic joint modeling

2.1. Assumptions for numerical calculations

The numerical analysis results are presented for a telescopic joint in a sample strong tube tower designed as a part of a 110 kV overhead power transmission line Munina-Lubaczów construction project to the designed 110/15 kV transformer station Korczowa. The analyzed section is a two circuit one, with span lengths of 240 to 360 m. Single spans, or occasionally pairs of adjacent spans constitute a strain section.

The tube tower of the type Orc M2+32, designated M2/47, designed for the considered line functions as tangent strain and angle strain tower. The routing angle at the locations of this tower is between 160 and 180 degrees. Total height of the tower is equal to 57 meters. The structure comprises of 6 tapered sections having hexadecagonal cross-section. The taper is equal to 21 mm per one meter of section height. The diameter of circle inscribed into the contour is equal to 2.20 m at the support and 1.00 m at the top of the tower. The tower is fitted with six working cross beams and two grounding ones. The conducting and grounding wires are suspended in a strain mode at all 8 supporting points. The steel-aluminum wires AFL 6–240 mm^2 are used as conductors, while steel-aluminum wires AFL 1–750 mm^2 are used as grounding wires.

The following design assumptions have been made regarding the supporting structure:

1. The tension in the conductor wires at +10°C is equal to 8.00 kN and 16.32 kN in the spans adjacent to the considered tower, while the tension in the grounding wires is equal to 2.94 kN and 6.93 kN, respectively.
2. The primary stress, i.e. the stress existing in the wires at the ambient temperature of -5°C, when the standard icing occurs. In the conductors this stress is equal to σ = 55 MN/m^2 and σ = 100 MN/m^2, while in the grounding wires it is equal to σ = 100 MN/m^2 and σ = 180 MN/m^2.

The telescopic connection of the two middle tower sections is analyzed numerically. Both segments are made of 14 mm thick steel plate. Segment lengths are equal to 11.4 m and 12.0 m, respectively, and the sleeve length in the joint is equal to 2.70 m, (cf. fig. 3). Both segments are made of S355J2 steel.

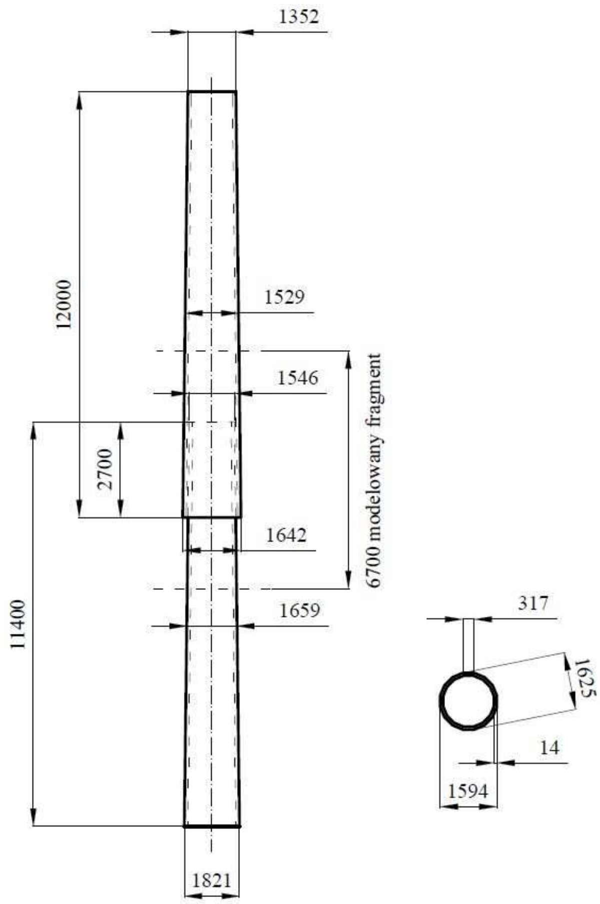

Fig. 3. Basic dimensions of the analyzed assembly sections in the telescopic joint

2.2. Numerical model of the joint

Numerical model of the tower segment has been built, having the length of 6.70 m, comprising of a section 2.70 m long, where the two connecting shell sections overlap, and two additional segments having the length of 2.00 m each, located above and below the overlapping zone. A fragment of the shell model of the joint is depicted in fig. 4.

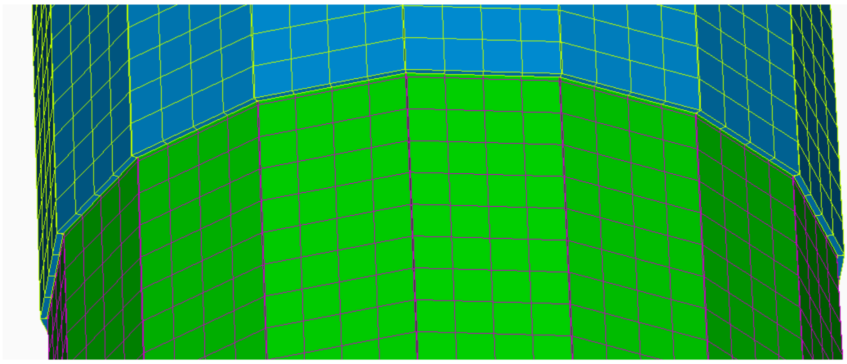

Fig. 4. Fragment of the shell model of the joint with 8x8 cm finite element mesh

The model has been loaded with a set of sectional forces: M_{ED} = 3,166.0 kNm, N_{ED} = 190 kN and V_{ED} = 157 kN, which were computed during the global statical analysis for authoritative combination of loads acting on the tower. The numerically nonlinear analysis including the contact between elements required several simplifying assumptions, which do affect the final precision of results, especially when the model undergoes large deformations. In particular the load is applied to the shell in single step, and the interaction mode of the contacting elements is set at the beginning of the analysis. Each pair of finite elements may at any moment during the analysis be in one of three mutually exclusive states: separation (no contact), compression (tangent forces lower than the maximum static friction force) or slip (tangent forces exceed the maximum static friction force). The linear analysis leads to the results, which are strongly dependent on the assumed finite element mesh size, but the increase in mesh density usually leads to the results, which are in better agreement with the results of nonlinear analysis.

The influence of finite element mesh size on the final results expressed as the equivalent stresses in the joint area is depicted in fig. 5. In fig. 5a), for the 8 × 8 cm element size, the maximum equivalent stress value is σ = 275 MN/m^2 and occurs in isolated points at the bottom edge of the upper section, while in fig. 5b) for the 4 × 4 cm element size the maximum equivalent stress value is σ = 241 MN/m^2.

According to the analyses performed by the authors, the finite element mesh composed of 16 × 16 cm elements yields the extreme equivalent stresses $\sigma = 175$ MN/m^2, thus the obtained results are ambiguous. Taking the results depicted in fig. 5a) as binding, one may state that the limit state condition, taking into account the plastic bearing capacity factor $\gamma_M = 1.1$ is satisfied, as: $\sigma = 241$ MN/m^2 < 355/1,1 = 323 MPa.

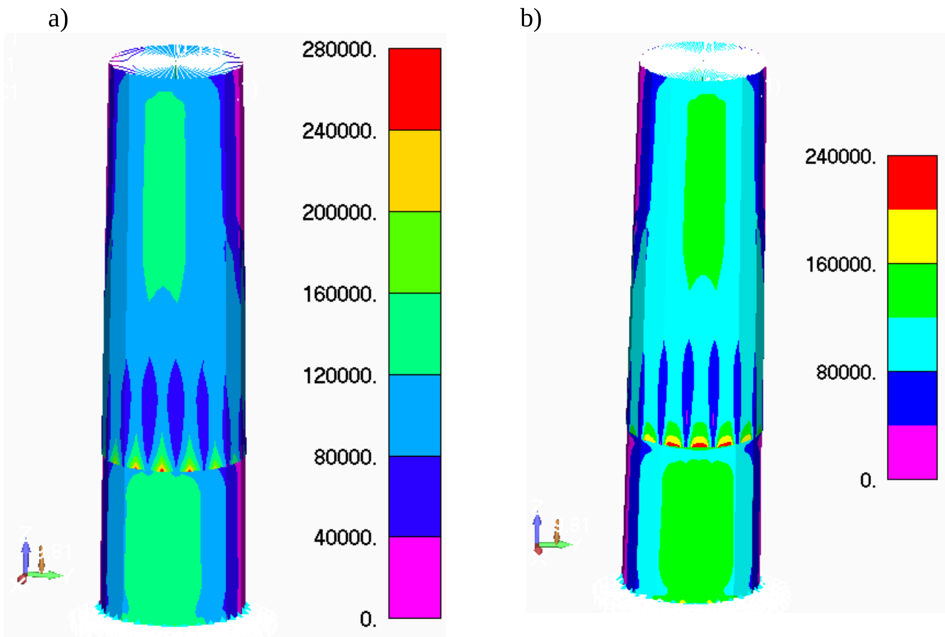

Fig. 5. Map of equivalent stresses for: a) 8 × 8 cm, b) 4 × 4 cm finite element mesh sizes

3. Summary

A comparative analysis of load coefficients for variable wind and icing actions has been performed in this paper with respect to the supporting structures of the overhead power transmission lines according to the Eurocode 3 and European code PN-EN 50341-2-22:2016 [8], for various reliability classes of such structures. The results of the analysis indicate, that the specifications present in these codes raise doubts, as they do not satisfy the analytical criteria resulting from the functional relationships between reliability classes of building structures. In addition, the coefficients mentioned above, and listed in the code PN-EN 50341-2-22:2016 according to the national recommendations are underestimated with respect to the specifications assumed for the structures of this type in the Eurocode PN-EN 1993-3. The load factor calibration errors demonstrated here are combined with modeling errors due to the application of linear analysis of telescopic joints in the tubular towers supporting overhead power transmission lines. Using computer modeling tools and linear, elastic shell theory, the modeling error of telescopic joints in a sample tower supporting overhead power transmission line rated at 110 kV is estimated. It should be noted, that modeling of a telescopic joint, based on the linear analysis leads to computational errors due to the simplifying assumptions and arbitrarily assumed dimensions of finite elements in the finite element mesh. In the example considered here the error is equal to γ_{Sd} = 275/241 = 1.14.

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