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INFLUENCE OF BOLTED SPLICE CONNECTIONS ON THE GLOBAL BEHAVIOUR OF STEEL LATTICE TELECOMMUNICATION TOWERS

[1]**Wojnar A.**, PhD, Assistant Professor
awojnar@prz.edu.pl, ORCID: 0000-0002-0537-3864
[2]**Marszałek K.**, MSc, Eng.
krzysztofgrzegorzmarszalek@gmail.com
[3] **Chernieva O.**, DSc., PhD, Associate Professor
chernieva@ogasa.org.ua, ORCID: 0000-0002-4807-6421
[1]**Śleczka L.**, DSc, PhD, Associate Professor
sleczka@prz.edu.pl, ORCID: 0000-0002-8979-7073
[1]*Rzeszow University of Technology*
Aleja Powstańców Warszawy 12, Rzeszow, 35-959, Poland
[2]*KARPAT-BUD sp. z o.o.*
Innowacyjna 5, Głogów Małopolski, 36-060, Poland
[3]*Odesa State Academy of Civil Engineering and Architecture*
Didrihsona 4, 65029, Odesa, Ukraine

Abstract. The bolted joints in the leg and the bracing members of the lattice transmission towers are always subjected to predominant axial forces, which will cause joint slip that greatly affects the global behaviour of the whole structure. The paper shows the results of the numerical modelling of the response of the steel lattice communication tower, with height h = 40.5 m located in Rzeszów. A comparison was made of five tower models, differing in the characteristics of the joint force-elongation relationship, including stiffness of the components and also joint slippage, coming from Category A joints. The paper presents the difference in displacements and rotations of chosen tower panels, internal forces in leg members, as well as in the fundamental flexural frequency obtained without considering the force-displacement characteristic and with four different ways of modelling of joints behaviour.

Keywords: steel lattice tower, global response, bolted splice connection, load-deformation of bolted joint.

1. Introduction. It is widely accepted that the effects of the behaviour of the joints on the distribution of internal forces within a structure, and on its overall deformations, should be taken into account during the analysis and design stage [1]. Consideration of the rotational stiffness of joints in steel frames made of the I or H sections is already well recognised and standardised.

Steel lattice towers are structures in which axial force is predominant, not bending moments. They are used as transmission lines or, especially recently, as supporting structures for mobile telephony base stations. They are often made of open profiles, frequently from hot-rolled angles that are joined by Category A bolted connections according to [1]. In such joints, the bolts are only slightly tightened, with the force of the worker's arm [2] and the clearance between the diameters of the bolt and the bolt hole is necessary to create useful dimensional tolerances during the fabrication and assembly of the structure. This clearance may cause the joined elements to slip when they are loaded. An additional source of their longitudinal deformability are components such as bolts in shear and bolts in bearing (for each element on which the bolts bear), so the stiffness of the bolted joint, even after the end of the slippage phase, has a lower value than the stiffness of the elements connected.

Such a slip in the bolted joints and their reduced longitudinal stiffness can influence the serviceability limit state of the steel lattice towers. It is reported that during the full-scale transmission lattice tower test, the experimentally obtained displacements can be up to three times greater than the numerical ones calculated without considering the influence of joints [3, 4].

Measures are taken to reduce the impact of slip on the behaviour of lattice structures of towers or masts. It is recommended [2] that for these structures, the nominal clearance for normal round holes be reduced by 0.5 mm. In this case for bolts M16-M24 the hole diameter should be only 1.5 mm larger than the diameter of the bolts instead of 2.0 mm. However, this only reduces the impact of slip, but does not fully eliminate it.

The paper presents the results of the numerical modelling of the response of the steel lattice communication tower, with height h = 40.5 m located in Rzeszów. A comparison was made of five tower models, differing in the characteristics of the joint force-elongation relationship, including stiffness of components and also joint slippage, from Category A use joints. The paper presents the difference in internal forces in the legs, displacements, and rotations of the chosen tower panels, as well as in the fundamental flexural frequency obtained without considering the force-displacement characteristic and with four different ways of modelling joint behaviour.

2. Case study – steel lattice tower with height 40.5 m.

The shaft of the considered tower is designed as a steel lattice space structure with height h = 40.5 m. It has a triangular plan, with a base width equal to 4.3 m, which gradually decreases to 1.5 m on the height 28.5 m, and above it has a constant width (Fig. 1). Cross bracing (type X) is applied as a bracing pattern. All members are made of hot rolled equal leg steel angles, of steel grade S235, and hot-dip galvanised for corrosion protection. The shaft is vertically divided into seven assembly sections, designated as S1÷S7, with their height equal to 5.0 or 6.0 meters.

The tower is used for telecommunication needs as a cellular base station. The equipment mounted on the tower consists of ladder placed within the shaft, with a guide for power cables and feeders, six panel antennas mounted at a height of 39.5 m, three microwave antennas with diameter 0.6 m (+38.0 m) and two working platforms (+36.5 and +38.5 m). The ladder is made of two vertically located cold-formed C profiles, with circular bars on one side, and brackets for feeders and cables on the other side. The ladder is self-supporting; the wind load on the ladder acts on the tower shaft, but self-weight of the ladder loads directly the foundation – so the bars constituting the ladder can be omitted from the analysis.

The steel profiles that make up the structure are joined using Category A bolt connections, according to [1]. Due to the uniform topology of the steel structure, the bolted connections are unified within the leg members and the bracing members. They are designated from ‘a’ to ‘e’ in Fig. 1. In all joints bolts grade 8.8 are used.

All bracing and secondary bracing members are connected to the legs with one bolt only (Fig. 2), using the gusset plates. Depending on the diameter of the bolts (M16 or M20), two types of joint are distinguished (‘d’ or ‘e’). In both types, the thicknesses of the walls of the connecting profiles are the same. Group ‘d’ includes joints of bracing members located within segments S1, S2 and S3; group ‘e’ includes joints in segments S4÷S7.

The splice joints of the leg members, linking the assembly sections, are double lap connections, with two shear planes (Fig. 3). Depending on the number of bolts and the thickness of the legs in connected angles, there are 3 recognised types, from ‘a’ to ‘c’. Throughout the height of the leg members, the ‘a’ joints are between the sections S1-S2, S2-S3, and between the foundation and the bottom section S1; the ‘b’ joints are between the segments S3-S4 and S4-S5, and also ‘c’ between the segments S5-S6 and S6-S7 (Fig. 1).

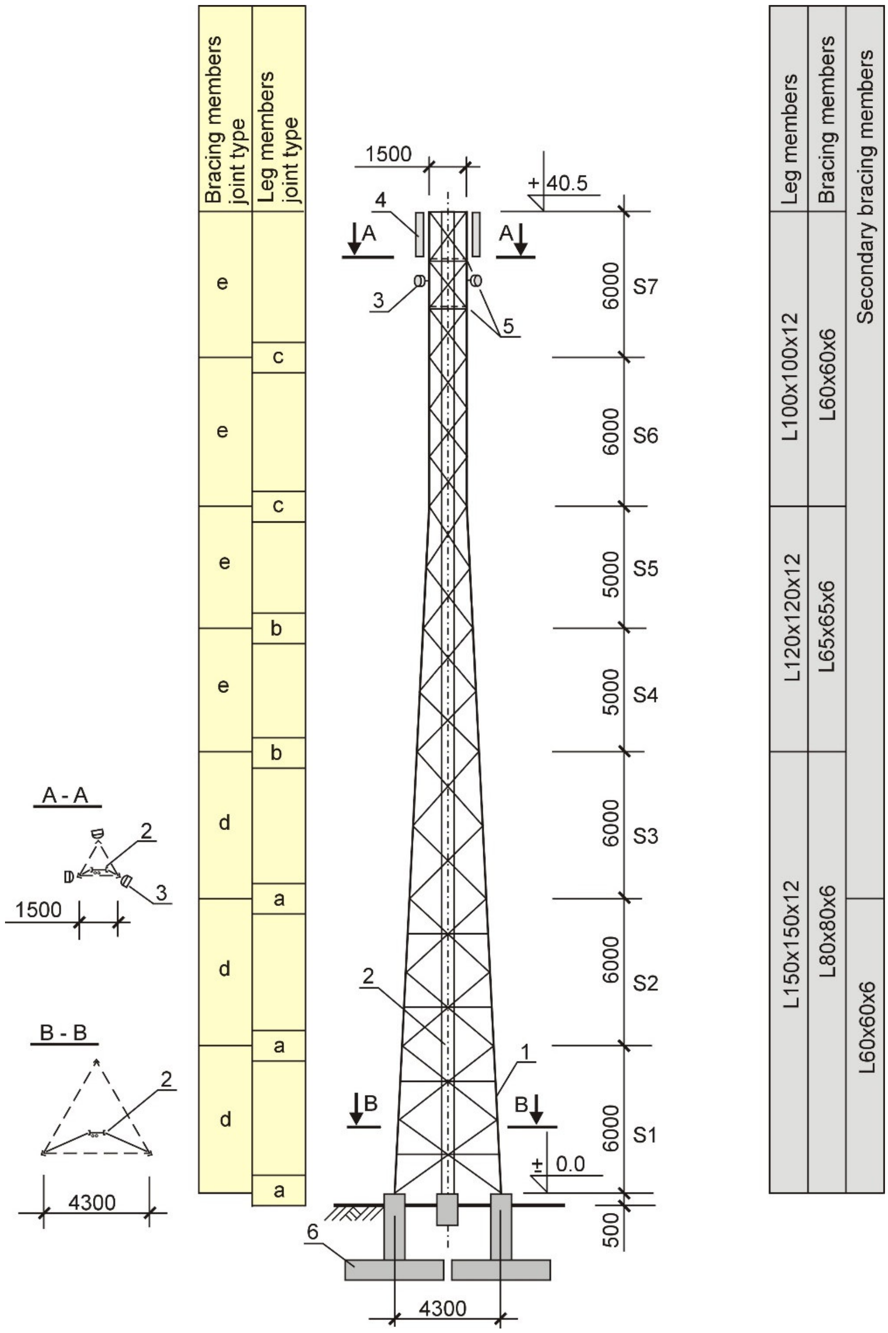

Fig. 1 The steel lattice tower analysed: 1 - tower shaft, 2 - ladder with cables and feeders, 3 - microwave antennas, 4 – panel antennas, 5 - working platforms, 6 - foundations

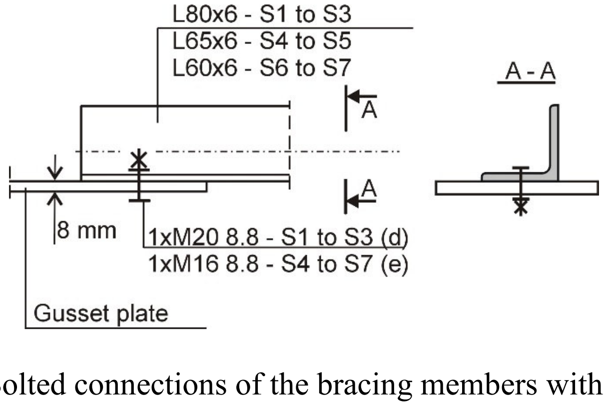

Fig. 2 Bolted connections of the bracing members with the legs

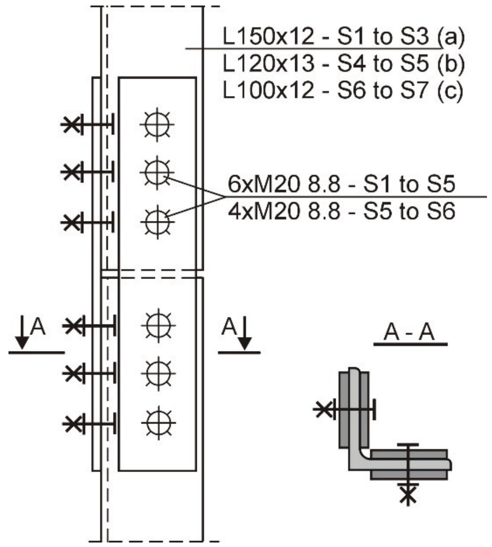

Fig. 3 Bolted splice joints in leg members

3. Wind load acting on the tower.

The predominant load on towers is wind actions, but permanent actions such as self-weight of structure, antennas, and working platforms, as well as imposed loads on the platforms, were also considered.

Wind forces come from the projected areas of structural elements and ancillaries, wind force coefficients, from the fundamental value of the basic wind velocity $v_{b,0}$ = 22 m/s, and the total height and location of the tower in category II terrain. The summary of the resultant equivalent gust wind load in the direction of the wind on structural components and linear ancillaries is presented in Table 1, determined according to Annex B of [5]. The resultant forces from wind acting on individual components are presented in Table 2. Three wind directions, shown in Fig. 4 were considered during the global analysis.

The atmospheric icing of the structure was not considered.

Table 1. Resultant forces for individual panels from wind on structural components and linear ancillaries

Panel	Resultant force $F_{T,W}(z)$ [kN]
S1	9.81
S2	10.92
S3	10.80
S4	8.37
S5	8.59
S6	9.94
S7	11.05

Table 2. Resultant forces for individual components from wind

Component	Level [m]	Resultant force $F_{T,W}(z)$ [kN]
Panel antennas	39.5	3.91
Microwave antennas	38.0	3.97
Platform	38.5	0.21
Platform	36.5	0.21

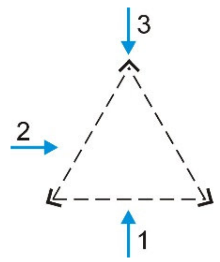

Fig. 4. Wind directions considered

4. Characteristics of bolted joints.

The numerical model of the structure is based on beam elements, with proper geometrical properties of cross sections and spatial orientation. The specified properties of the steel material are the nominal values used in the design. To capture the effect of reduced stiffness and slippage of the bolted joint on tower behaviour, all bolted joints were treated as nonlinear springs, with appropriate force-displacement characteristics, Fig. 5. In each bolted joint, the nonlinear spring is defined with proper translational stiffness along the longitudinal axis of the elements. In simple joints due to the bending moment, zero rotational stiffness was also defined at the end of the element.

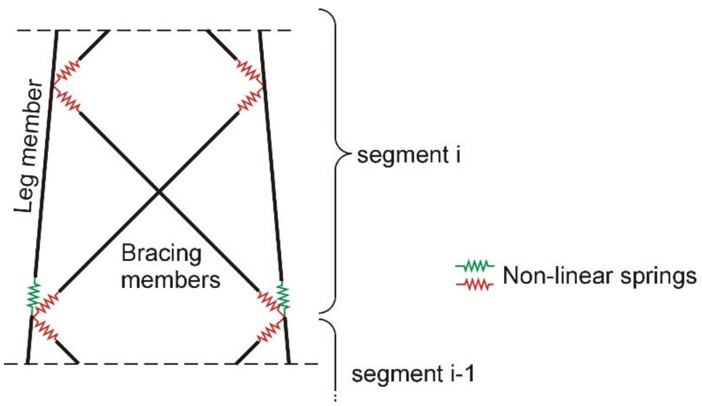

Fig. 5. Mechanical model of bolted joints

The force-displacement characteristics used, including slip effects, vary depending on the joint type (from ‘a’ to ‘e’ – see paragraph 2). In addition, some joint models of each joint were studied. The models of connection behaviour adopted are an idealisation of the model developed by Ungkurapinan et al. [6], which is now widely used [3, 4, 7].

The first model ignores the influence of longitudinal stiffness of the joints and slippage. The second model implemented in the global analysis includes only the axial stiffness of the joint, ignoring the effect of slippage; Fig. 6.

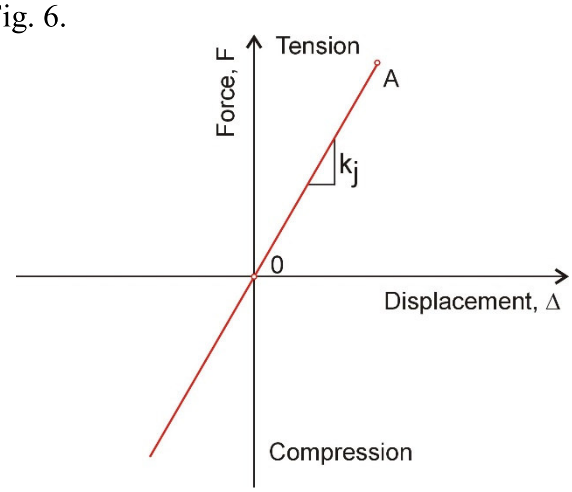

Fig. 6. Bolted joints influencing only stiffness of components (without slippage)

The longitudinal stiffness of the bolted joints was estimated based on formulas presented in EN 1993-1-8 [1]. It depends on the stiffness of the basic components (bolts in shear and bolts in bearing) and the number of cover plates and also the global configuration of the joints (single- or double-sided joints). A list of formulas used to determine initial longitudinal stiffness, together with mechanical models of bolted connections, is presented in [8, 9]. The determined stiffnesses are listed in Table 3 for each type of joint.

Table 3. Force-displacement parameters for joints

Joint type	Longitudinal stiffness k_i [N/mm]
a	171360
b	156870
c	94710
d	9870
e	7770

The idealised load-deformation characteristics for the joints taking into account the stiffness of the joint components and the slippage between the connecting elements are presented in Figs. 7 and 8, separately for connections of the bracing members with the legs, and splice joints of legs. In both cases slippage can occur immediately at zero load, but from the numerical point of view a light slope of 0-A branch was given, equal to $k_{0\text{-}A} = 0.1k_j$ (Figs. 7 and 8). The slip range considered was equal to ±1.5 mm in case of bracing members joints (single-sided connections) and its doubled value ±3.0 mm in case of double lap connections of the leg members. In both cases, the centred position of the bolts relative to the connected elements was assumed. The stiffness k_j of the A-B region (bearing stage) was calculated according to the component method and their values are the same as in Table 3.

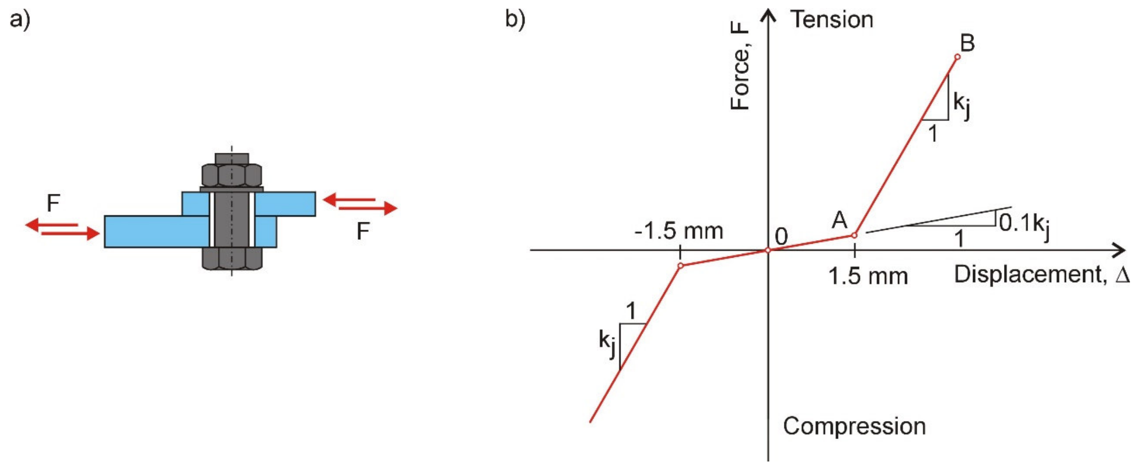

Fig. 7. Bracing member joints with centred position of the bolt

Next force-displacement model of the bolted connections was applied only to leg member joints, Fig. 9. In this case, a shifted position of the bolts was used to influence the assembly conditions. During the erection stage, each section of the tower is assembled on the ground, then lifted as a unit and bolted to the previously installed segment. In this case, the weight of the segment affects the small compression force and bearing of the bolts, and then they are tightened. Therefore, later in the working life stage, slippage in such joints is possible only in the case of a tensile force and their range is equal to 6 mm. The stiffness k_j of the A-B region (bearing stage) is the same as in Table 3.

In any case, the plastic region (failure stage) of the force-displacement characteristic, which occurs after reaching the shear or bearing resistance of the bolts, was not modelled due to checking the failure criteria during structural design.

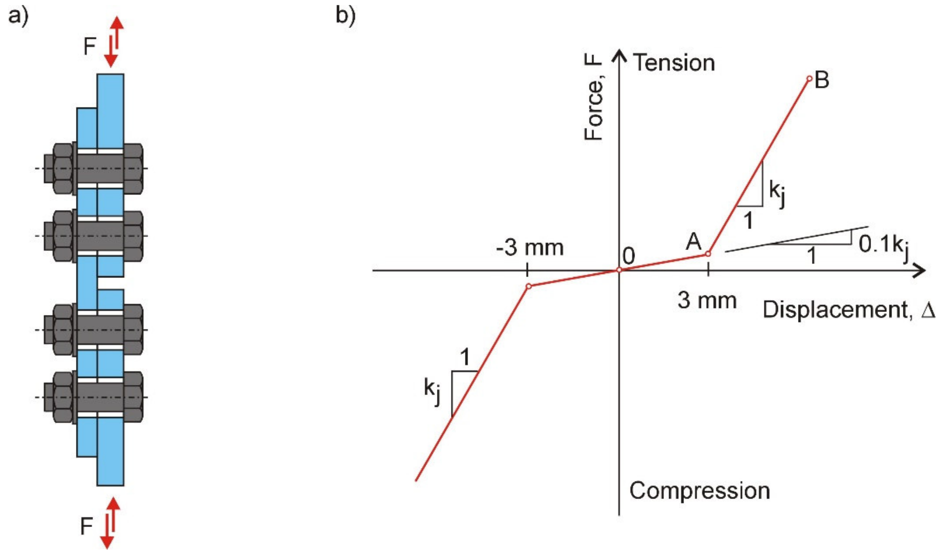

Fig. 8. Splice joints in leg members with centred position of the bolts

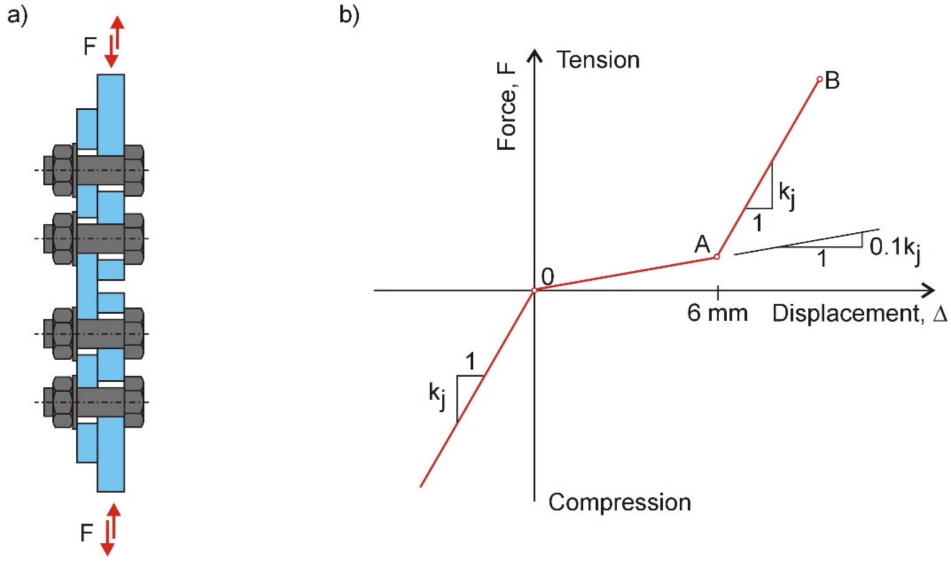

Fig. 9. Splice joints in leg members with shifted position of the bolts

5. Modelling of the structure.

The response of the whole structure was determined using an elastic first-order analysis, without taking into account global or local imperfections.

Three wind directions were considered. Basic loading combinations include permanent actions and wind loading, with partial factors for actions appropriate for the ultimate limit state or the serviceability limit state. Structural analysis was performed in Autodesk Robot Structural Analysis Professional 2022 software [10]. All tower members were checked to meet the ULS provisions of Eurocode 3.

The computational models considered for the tower differed in the degree of detail of the force-displacement characteristics for the connections. Model 1 does not consider the stiffness characteristics of the joints for longitudinal force or the effect of slippage. Model 2 includes only the linear stiffness characteristics of the joints, according to Fig. 6, for all joints, both the leg and the bracing members, with the appropriate stiffness assignment. In Model 3, for the bracing members, the force-displacement characteristic with slippage was considered (according to Fig. 7) and for the leg connections, neither the reduction in stiffness nor slippage was modelled. Models 4 and 5 consider the force-displacement characteristics with slippage with centred position of the bolts (acc. to Fig. 7) for bracing members, and for leg joints characteristic with slippage with centred position of the bolts (Fig. 8), and respectively, with shifted position of the bolts (acc. to Fig. 9).

The computational models considered, due to the degree of force-displacement characteristic, are compared in Table 4.

Table 4. Considered numerical models of the tower, due to the behaviour of joints

Designation	Bracing member joints	Splice joints in leg members
Model 1	- simple due to bending moment - no joint effect due to longitudinal force and slippage	- continuous due to the bending moment - no joint effect due to longitudinal force and slippage
Model 2	- simple due to bending moment - force-displacement characteristic according to Fig. 6	- continuous due to the bending moment - force-displacement characteristic according to Fig. 6
Model 3	- simple due to bending moment - force-displacement characteristic according to Fig. 7	- continuous due to the bending moment - no joint effect due to longitudinal force and slippage
Model 4	- simple due to bending moment - force-displacement characteristic according to Fig. 7	- continuous due to the bending moment - force-displacement characteristic according to Fig. 8
Model 5	- simple due to bending moment - force-displacement characteristic according to Fig. 7	- continuous due to the bending moment - force-displacement characteristic according to Fig. 9

6. Obtained response.

The analysis showed that the change in internal forces in leg members caused by the different ways of influencing (or omitting) the force-displacement characteristic of the joints and slippage is practically insignificant. The change in longitudinal forces in the leg members obtained in Models 2÷5, were not bigger than ±2.0% in bottom segments (S1-S3) compared to Model 1, where the influence of longitudinal stiffness of the joints and slippage was omitted. In the upper segments, the change was slightly larger (up to ±4.0%) in Segments S4-S6, and even ±8.0% in the highest segment S-7, although the internal forces there are the smallest and do not determine the check of ULS.

However, the influence of the longitudinal stiffness of the joints and slippage on the behaviour of the tower in the serviceability limit state was significant.

Neglecting the effect of the connection characteristic results in lower tower deflection and the angle of rotation of the segment in which the antennas are mounted (Figs. 10 and 11 – wind direction 2). Similarly, the basic dynamic characteristics, which is the basic flexural frequency of the tower computational model, in which the influence of connections was not considered, are overestimated compared to models with proper force-displacement characteristic taken into account (Fig. 12).

All of these results are based on a single tower analysis, but it seems to be the basis for generalisations.

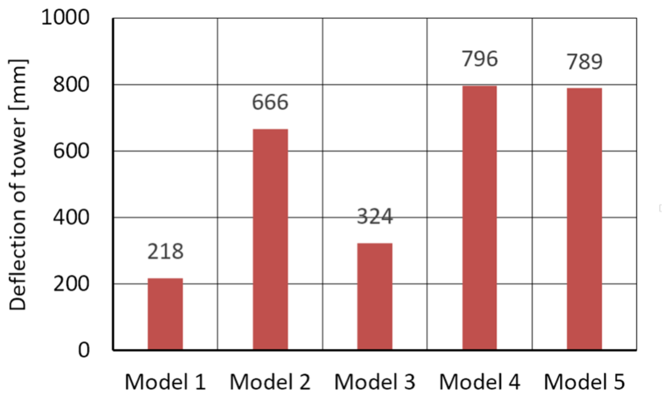

Fig. 10. Tower deflection depending on the force-displacement characteristic of the joints

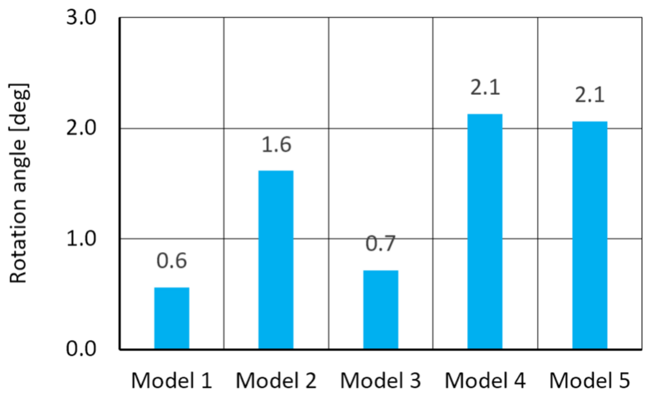

Fig. 11. Rotation angle of the mounting point of microwave antennas depending on the force-displacement characteristic of the joints

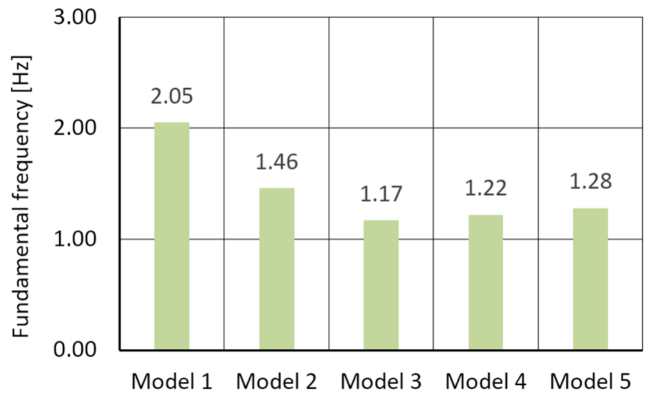

Fig. 12. The fundamental flexural frequency of the tower shaft depending on the force-displacement characteristic of the joints

7. Summary. The omission of the longitudinal stiffness of the joints and their slippage results in a significant underestimation of the resulting deflections and rotations of the tower.

The model without considering the slip but including the reduction of the connection stiffness gives results comparable to the models that include both the reduction of stiffness and the slip and is additionally numerically efficient (no calculation problems due to linearity of the problem).

The use of computational models that capture slippage makes the calculations non-linear, which sometimes manifests itself in a lack of convergence. Then you should change the task settings (increasing the load increment number, increasing the number of iterations, using the line-searching method, etc.).

There is practically no difference between the results of the calculations of the models that include the slippage of the joints resulting from the centred or shifted position of the bolts in the bolt holes.

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ВПЛИВ БОЛТОВИХ З'ЄДНАНЬ НА ЗАГАЛЬНУ ПОВЕДІНКУ СТАЛЕВИХ ҐРАТЧАСТИХ ТЕЛЕКОМУНІКАЦІЙНИХ ВЕЖ

[1]**Вожнар А.,** к.т.н.,
awojnar@prz.edu.pl, ORCID no. 0000-0002-0537-3864
[2]**Маршалек К**., магістр,
krzysztofgrzegorzmarszalek@gmail.com
[3]**Чернєва О.,** к.т.н., доц.
chernieva@ogasa.org.ua, ORCID no. 0000-0002-4807-6421
[1]**Сленчка Л**., к.т.н., доц.,
sleczka@prz.edu.pl, ORCID no. 0000-0002-8979-7073
[1]*Жешувський технологічний університет*
Алея Повстанців Варшави 12, Жешув, 35-959, Польща
[2]КАРПАТ-БУД ООО о.о.
Інноваційна 5, Глогув Малопольські, 36-060, Польща
[3]*Одеська державна академія будівництва та архітектури*
Дідріхсона 4, 65029, Одеса, Україна

Анотація. Сталеві ґратчасті опори часто виготовляються з відкритих оцинкованих секцій, з'єднаних болтовими з'єднаннями унапусток категорії А відповідно до EN 1993-1-8, які піддаються переважним осьовим навантаженням, що викликає прослизання стиків, що сильно впливає на загальну поведінку.

У роботі представлені результати чисельного моделювання поведінки сталевої ґратчастої вежі заввишки H=40,5 м. у Жешуві. Було проведено порівняння чотирьох моделей веж, які відрізнялися характеристиками відношення зусилля у з'єднанні до подовження, включаючи жорсткість компонентів, а також прослизання з'єднань, що виникає через зазор між діаметрами болтів (d) та діаметрами отворів під болти. (d_0). У статті проводиться порівняння прогинів, поворотів вибраних панелей вежі (місць установки антени), внутрішніх сил у стійках та елементах решітки, а також частоти основного вигину валу вежі.

Вежа була навантажена власною вагою конструкції та обладнання, а також вітровими впливами, зумовленими значенням швидкості вітру $v_{b,0}$ = 22 м/с та розташуванням вежі у II категорії місцевості.

У першій моделі з'єднання враховувалися як жорсткі (без прослизання та з нескінченною поздовжньою жорсткістю). У другій моделі поздовжня жорсткість і прослизання враховувалися лише сполуках розкосів. Третя модель враховувала поздовжню

жорсткість і прослизання у всіх з'єднаннях (в елементах розкосів, а також у стикових з'єднаннях в елементах опор). Четверта модель враховувала всі з'єднання (як і третя), але діапазон ковзання в стикових з'єднаннях гілок у цьому випадку враховувався трохи інакше.

Проведений аналіз вказує на необхідність урахування прослизання та поздовжньої жорсткості, що виникають у болтових з'єднаннях категорії A EN 1993-1-8 у граничному стані за придатністю до експлуатації. Це особливо важливо для телекомунікаційних вишок, оскільки деформації, що виникають при цьому, можуть призвести до неприпустимого зниження рівня обслуговування, що забезпечується розташованими на них антенами.

Ключові слова: сталева ґратчаста вежа, загальна реакція, болтове з’єднання, навантаження-деформація болтового з’єднання.