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Formulas for calculating deformations of a cross-shaped tower

Kirsanov, Mikhail Nikolaevich[1]

[1] Moscow Power Engineering Institute, Moscow, Russian Federation; c216@ya.ru (K.M.N.)
Correspondence:* email c216@ya.ru

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Abstract:

The object of research is a spatially regular tower truss with a cross-shaped plan. The truss rod material is elastic, the rod cross-sections have equal stiffness, and hinges connect the rods. The trusses of the twelve lateral faces of the truss are planar diagonal lattices. The truss structure is statically determinate. The truss is loaded at the nodes. **Method.** Analytical dependencies of deflection on the number of panels are derived in the Maple computer mathematics system using the induction method. To determine the forces, a system of algebraic equilibrium equations for the nodes is constructed. Nodal displacements are calculated using the Maxwell-Mohr formula. The effect of uniformly distributed and concentrated lateral loads is considered. **Results.** Formulas for the dependence of node displacements on the load magnitude and the structure dimensions are polynomials in the number of panels. Asymptotic forms of the solutions and forces in individual, most critical rods are obtained.

1 Introductions

The finite element method is typically used to calculate stresses and displacements in complex-shaped rod structures [1], [2]. Complex solutions obtained using the initial value function method are also known [3]–[5]. Recently, the induction method has been used to derive analytical relationships for the deflection and natural frequency of trusses. In [6], for example, the optimization problem for a regular truss was solved, taking into account material creep. The reference books [7], [8] contain formulas for calculating the deflection of planar regular trusses obtained in the Maple computer mathematics system using the induction method. The problem of calculating regular rod structures was apparently first addressed by Hutchinson, R.G., and Fleck, N.A. [9], [10]. The formula for the deflection of a regular planar truss was derived by the induction method in [11]. In [12], formulas were obtained for the deflection of a lattice truss taking into account its kinematic variability for a certain number of panels. A distribution scheme for virtual node velocities was found in the case of structure variability. The analytical method of initial functions for calculating a supported rectangular plate was used in [13]. The deflection of a composite truss was calculated analytically in [14]. The boundaries of the resonant safety zone of a planar truss structure with a cross-shaped lattice and a rise in the middle of the span with an arbitrary number of panels are calculated in [15]. The formula for the first natural frequency of oscillations of a two-span truss was obtained in [16]. The object of the study is a planar statically determinate truss of the regular type in [17]. The truss has a Sprengel grid. The load is evenly distributed over the nodes of the upper belt. The formula for the dependence of the deflection of a planar statically determinate truss with an arched lattice on the height and number of panels was derived. Efficient reduced-order modeling for non-linear dynamic analysis of beamlike truss structures is presented in [18]. A novel approach to build a reduced-order model of planar beamlike periodic truss with both high accuracy and high computational efficiency is presented in [19]. This approach merges the classical equivalence method and the discrete adaptive mesh refinement. Analysis of frame buckling without sidesway classification is given in [20]. An analytical calculation of the deflection of a flat external statically indeterminate truss with an arbitrary number of panels in the Maple computer mathematics system was performed in [21]. Formulas for calculating the deflection and natural frequency of vibrations of a planar arched truss depending on the number of panels using the induction method are derived in [22]. In [23], a design method of an aeroelastic model for a long-span, three-tower steel truss girder cable-stayed bridge is proposed for the first time. In [24], a solution is presented using induction and the Maple computer mathematics system to determine the deflection of a planar model of a tower structure depending on the number of identical panels. Analytical formulas for determining the deflection are derived. The deflection is determined for four horizontal loading cases: a distributed load and a point load on the top node on one and the other side, respectively. The static calculation of truss deformations in analytical form is performed in [25]. The formulas for the dependence of the deflection of a trapezoidal beam truss on the number of panels are obtained and analyzed in [26]. The statics and dynamics of spacecraft antenna trusses are discussed [27]. The natural frequencies calculated by the frequency equation are compared with those obtained by the finite element software ANSYS (www.ansys.com) to verify the GMM model proposed in this paper. Using the mode shapes and their orthogonality, the reduced-order decoupled dynamic equations of the spacecraft are obtained. A rational analysis of the influence of geometric and material parameters on the triangular lattice truss beam's performance is presented in [28], along with a novel method for correlating static stiffnesses to natural frequencies in bending and torsion. The equivalent bending and torsional stiffness of the structure is estimated using finite element modeling with variations in the cross-section, material properties, location, and orientation of individual composite tubes in the repeating lattice truss element. The optimization problems of truss structures are considered in [29]. The validity and reliability of the truss calculation method proposed here are confirmed using the example of planar and spatial truss structures, demonstrating that the method significantly reduces the computation time in structural analysis – up to 25 times – compared to traditional Monte Carlo methods. A space frame theory is presented in [30], which can be used as the basis for implementing an analysis platform for first-order linear analysis of space frames with significant warping restraint at their nodes. An analytical assessment of the deflection of a rod model of a hipped roof structure using the induction method and a computer system of symbolic calculations is given in [31].

This paper proposes a new design for a spatially axisymmetric, statically determinate tower structure with an arbitrary, parametrically specified number of panels (Fig. 1). The truss is externally statically indeterminate. The purpose of this paper is to derive formulas for the dependence of the structure's deflections under various nodal loads on its dimensions and the number of panels. The truss under consideration has $K = 36n + 39$ elements, including 15 support elements, 12 of which are vertical struts of length h. External static indeterminacy is revealed by solving a combined system of equilibrium equations for all truss nodes.

2 Materials and Methods

The tower-type cruciform hinge-rod structure has twelve identical flat lateral faces—diagonal lattices of height nh. The structure is supported by twelve vertical rigid support rods and three horizontal rods, modeling a spherical support at A and a cylindrical one at B. Four rods of length $\sqrt{h^2 + a^2/2}$ and $\sqrt{h^2 + 5a^2/2}$ converge at the apex C. The task is to obtain an analytical dependence of the structure's deflections on the number of panels. The calculation of forces in the rods is performed analytically using the computer mathematics system Maple [32]. Expressions for forces depending on the current nodal load are calculated by the method of cutting out nodes. The direction cosines of the forces in the rods included in the projection equations are determined by the coordinates of the nodes and the structure of the rod connections. The truss consists of $K=36n+39$ bars, including 15 support bars and $12n+13$ internal hinge joints.

The forces in the bars due to the external load are determined by solving a system of linear equilibrium equations for the bars in projections. The direction cosines of the unknown forces in the bars are determined from the node coordinates. The coordinates of the nodes are as follows:

$$\begin{aligned}
&x_i = x_{i+11(n+1)} = 3a,\ x_{i+n+1} = x_{i+2n+2} = x_{i+9n+9} = x_{i+10n+10} = 2a,\\
&x_{i+3n+3} = x_{i+4n+4} = x_{i+7n+7} = x_{i+8n+8} = a,\\
&x_{i+5n+5} = x_{i+6n+6} = 0,\ \ y_i = y_{i+n+1} = y_{i+4n+4} = y_{i+5n+5} = a,\\
&y_{i+2n+2} = y_{i+3n+3} = 0,\ y_{i+6n+6} = y_{i+7n+7} = y_{i+10n+10} = y_{i+11n+11} = 2a,\\
&y_{i+8n+8} = y_{i+9n+9} = 3a,\ z_{i+(n+1)j} = h(i-1),\ \ j = 0,..,11,\ i = 1,..,n+1.
\end{aligned} \tag{1}$$

Coordinates of vertex *C*:

$$x_{12n+13} = y_{12n+13} = 3a/2,\ z_{12n+13} = h(n+1). \tag{2}$$

The structure of the connection of individual rods into a lattice is specified by special vectors $\overline{q}_i$, $i=1,...,\eta$, containing the numbers of the ends of the corresponding rods by analogy with the assignment of a list of edges of a graph in discrete mathematics. Vectors defining the 12 side posts of the truss:

$$\overline{q}_{i+(j-1)n} = [i+(j-1)(n+1), i+(j-1)(n+1)+1], \quad i=1,...,n,\ j=1,...,12. \tag{3}$$

Vectors defining the dome of a truss with vertex *C*:

$$\overline{q}_{i+36n+12} = [i(n+1), 12n+13],\ i=1,...,12. \tag{4}$$

The vertical support rods at the base of the tower have the following numbers:

$$\overline{q}_{36n+24+i} = [1+(n+1)(i-1), 12n+13+i],\ i=1,...,12. \tag{5}$$

Other bars of the truss lattice are defined similarly.

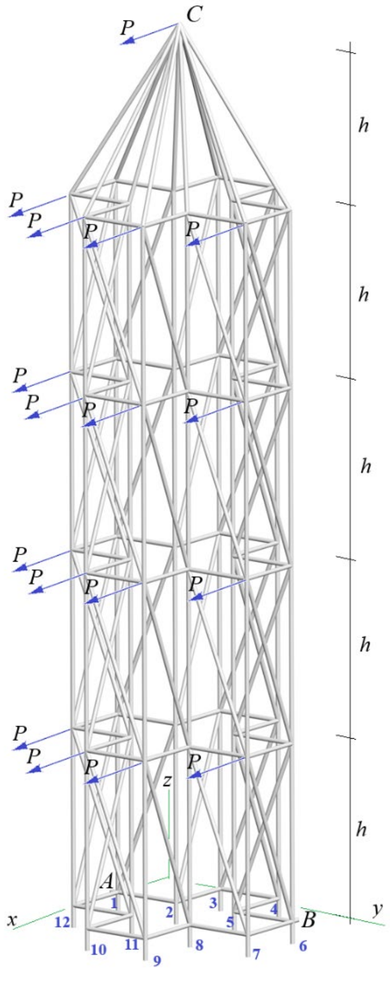

Fig. 1 – Truss, $n=4$. Load and support numbers

Kirsanov, M.
Formulas for calculating deformations of a cross-shaped tower;
2026; Construction of Unique Buildings and Structures; 123 Article No 12301. doi: 10.4123/CUBS.123.1

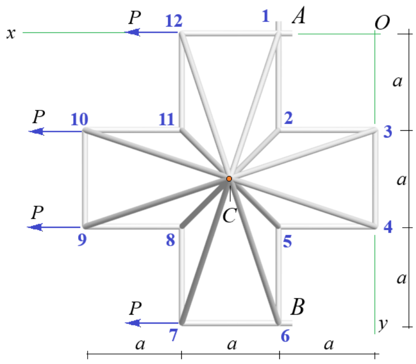

Fig. 2 – Horizontal section of the pyramid and top of the truss

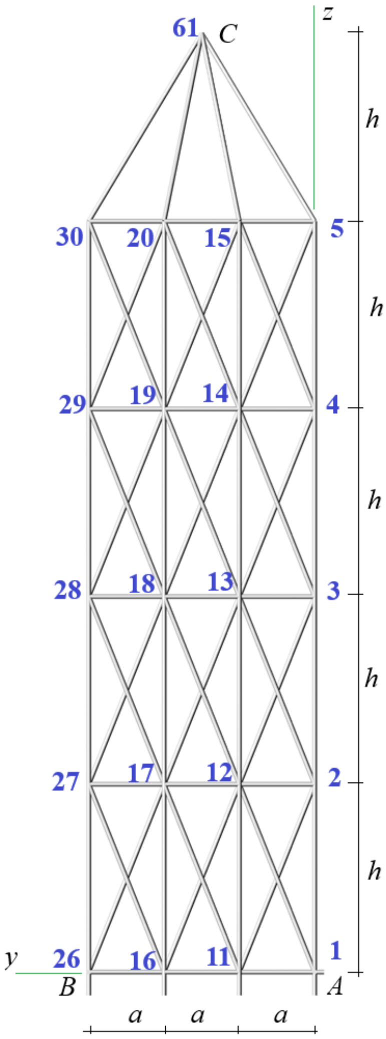

Fig. 3 – Hinge numbers

The elements of the matrix of the system of equilibrium equations for nodes in projections onto the coordinate axes are calculated using these data. In the matrix version, the system of equilibrium equations for all internal nodes of the truss has the form:

$$\mathbf{GS}=\mathbf{B}, \tag{6}$$

where **G** is the matrix of direction cosines of forces, and the vector of loads on nodes is designated as **B.** The vector of unknown forces in the rods and fifteen support reactions is equal to **S**. The length of these vectors is equal to the number of truss elements, *K*. The elements of the load vector in the right part of (6) of type B_{3i-1} are the load on node *i* in projection onto the *x*–axis, the value B_{3i-2} is the load in projection onto the *y*–axis, B_{3i} is the vertical load on node *i*. The elements of the matrix **B** (direction cosines of the force vectors on the coordinate axes) are calculated using data on the lattice structure and node coordinates:

$$l_{x,i} = (x_{q_{i,1}} - x_{q_{i,2}}) / l_i,\ l_{y,i} = (y_{q_{i,1}} - y_{q_{i,2}}) / l_i,\ l_{z,i} = (y_{q_{i,1}} - y_{q_{i,2}}) / l_i,\ \ i = 1,\ldots,K, \tag{7}$$

where $l_i = \sqrt{l_{x,i}^2 + l_{y,i}^2 + l_{z,i}^2}$ is the length of the rod with number *i*. In every three consecutive rows of matrix B, projections of forces in the rods onto the node are written. In the Maple program, the inverse matrix method is used to find a solution to system (6) in analytical form.

To obtain an analytical dependence of the deflection of the structure on the number of panels, the induction method, previously well tested in [11] and [12], is used. The problem of the horizontal displacement of the vertex C in the direction of the x-axis (Fig. 1) is considered under the action of a nodal load on the four edges of the tower in the direction of the x-axis. The non-zero elements of the right-hand side of (6), respectively, for each of the four edges, have the form

$$\begin{aligned} &j = i + 6(n+1) + 1,\ j = i + 8(n+1) + 1,\ j = i + 9(n+1) + 1,\ j = i + 11(n+1) + 1, \\ &B_{3j-1} = P,\ i = 1,\ldots,n. \end{aligned} \tag{8}$$

3 Results and Discussion

The Maxwell–Mohr formula for calculating deflection involves summing over all structural members:

$$\Delta_n = \sum_{i=1}^{K} S_i^{(p)} S_i^{(1)} l_i \,/\, (EF), \tag{9}$$

where $S_i^{(1)}$ is the force in member number *i* under the action of a unit horizontal force in the *x*-axis direction on node *C* with number 12*n*+13, where the displacement is recorded; $S_i^{(p)}$ are the forces in the truss members due to the distributed load; and l_i is the length of this member. The member stiffness *EF* is assumed to be the same for all truss members. Sequential calculation of deflection for *n*=1, 2, 3,… using formula (1) yields the following values:

$$\begin{aligned} &\Delta_1 = P(436a^3 + 60c^3 + 16d^3 + 81f^3 + 2020h^3)/(24h^2EF), \\ &\Delta_2 = P(812a^3 + 192c^3 + 26d^3 + 135f^3 + 6168h^3)/(24h^2EF), \\ &\Delta_3 = P(420a^3 + 132c^3 + 12d^3 + 63f^3 + 4724h^3)/(8h^2EF), \\ &\Delta_4 = P(1780a^3 + 672c^3 + 46d^3 + 243f^3 + 2824h^3)/(24h^2EF), \\ &\ldots \end{aligned} \tag{10}$$

where $c = \sqrt{h^2 + a^2}$, $d = \sqrt{4h^2 + 10a^2}$, $f = \sqrt{4h^2 + 2a^2}$.

By the method of induction with the use of special operators of the Maple system, one can obtain the general term of the resulting sequence:

$$\Delta_{x,n} = P(C_1a^3 + C_2c^3 + C_3d^3 + C_4f^3 + C_5h^3)/(h^2EF) \tag{11}$$

where

$$\begin{aligned} &C_1 = (9n^2 + 67n + 33)/6,\ \ C_2 = n(2+3n)/2,\ C_3 = (5n+3)/12, \\ &C_4 = 9(2n+1)/8,\ C_5 = (11n^4 + 56n^3 + 353n^2 + 458n + 132)/12. \end{aligned} \tag{12}$$

The coefficients are obtained from solutions of recurrence equations. For coefficient C_1, such an equation has the form $C_{1,n} = 3C_{1,n-1} - 3C_{1,n-2} + C_{1,n-3}$. To solve this equation with initial data

$C_{1,1}=109/6,\ C_{1,2}=203/6,\ C_{1,3}=105/2,\ C_{1,4}=445/6$, the rsolve operator in Maple was used. You might as well use the computer math system Mathematics [32] here.

The curves in Figure 4 show the dependence of the dimensionless deflection $\Delta'_x=\Delta_{x,n}EF/(\tilde{P}H_0)$, on the number of panels, where $H_0=h(n+1)$ is the truss height, and $\tilde{P}=P(4n+1)$ is the total load on the truss. These curves have a clearly defined minimum, which can be used to optimize the rigidity of the designed structure. Beginning at a certain value of *n*, the dimensionless deflection increases monotonically. The limit $\lim_{n\to\infty}\Delta'_x/n^2=11/48$ shows that the dependence of the deflection on the number of panels tends to a quadratic law.

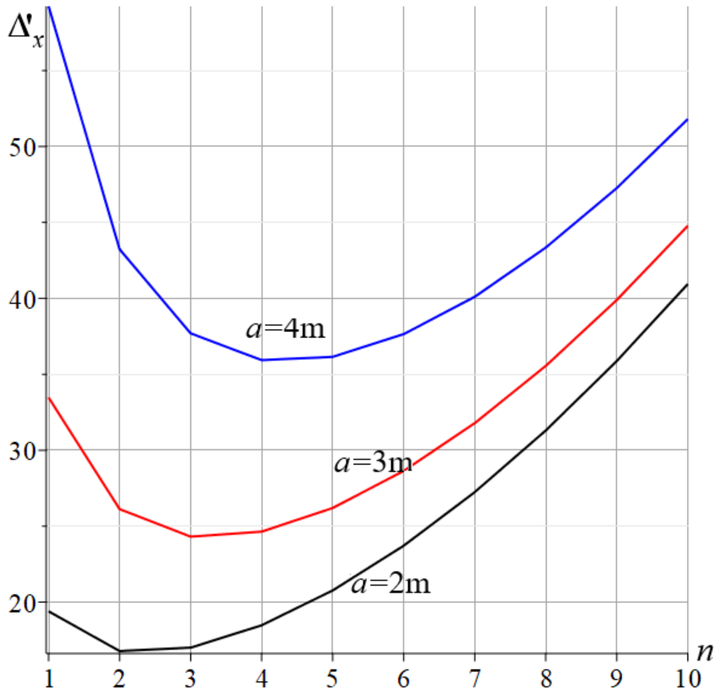

Fig. 4 – Offset of the vertex C along axis x; *h=2m*

From the solution of the system of equilibrium equations, the vertical reactions of supports 1-12 are obtained (Fig. 1, 2). Due to the symmetry of the load, the reactions are pairwise equal:

$$\begin{gathered}R_1=R_6=Ph(9n^2+14n+3)/(6a),\ R_2=R_5=-Ph(3+8n)/(2a),\\ R_3=R_4=Ph(3+8n)/(6a),\ R_7=R_{12}=-Ph(9n^2+10n+3)/(6a),\\ R_8=R_{11}=-Ph(n^2-5n-3)/(2a),\ R_9=R_{10}=Ph(n^2-n-1)/(2a).\end{gathered}\tag{13}$$

Fig. 5 shows the curves of the dependence of support reactions, related to load *P*, on the number of panels for $a=2m,\ h=3m$. These curves have linear or quadratic asymptotes: $\lim_{n\to\infty}R_1/n^2=3Ph/(2a)$, $\lim_{n\to\infty}R_2/n=-4Ph/a$, $\lim_{n\to\infty}R_3/n=4Ph/a$, $\lim_{n\to\infty}R_7/n^2=-3Ph/(2a)$, $\lim_{n\to\infty}R_8/n^2=-Ph/(2a)$, $\lim_{n\to\infty}R_9/n^2=Ph/(2a)$.

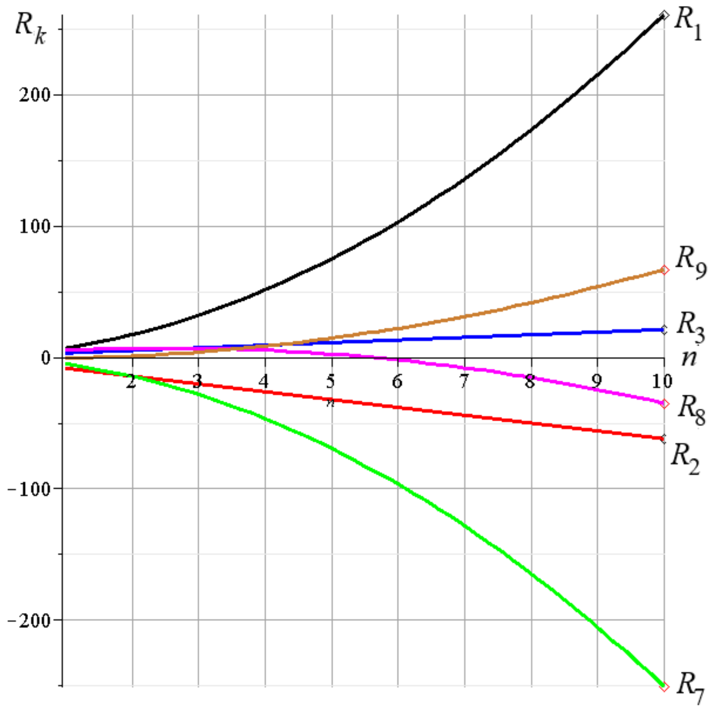

Fig. 5 – Vertical reactions of supports

From the equilibrium equation of the entire truss in projection onto the vertical axis *z,* without taking into account the weight of the structure, it follows $\sum_{i=1}^{12} R_i = 0$.

Despite the external symmetry of the structure, it is still asymmetrical due to the location of the spherical hinge support *A* and the cylindrical one *B*. Hence, the expression for the deflection of the top *C* in the direction of the *y*-axis under the action of the same load along the *y*-axis has other coefficients in (2):

$$C_1 = (168n^2 + 362n + 51)/36,\ C_2 = n(19 + 84n)/18,\ C_3 = (4n+1)/9,$$
$$C_4 = (4n+1)/4,\ \ C_5 = (879n^4 + 3656n^3 + 4821n^2 + 3424n + 624)/216. \quad (14)$$

The curves of the dependence of the displacement of the vertex along axis *y* are shown in Figure 6. As the number of panels increases, the curves constructed for different values of the parameter a converge.

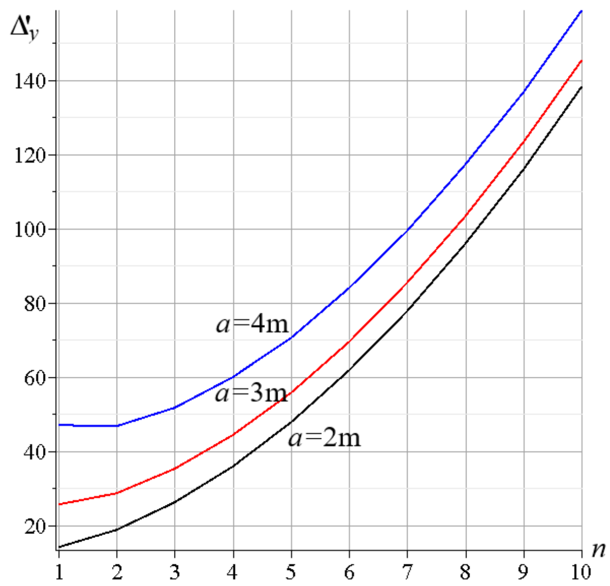

Fig. 6 – Offset of the vertex *C* along axis y; *h=2m*

Kirsanov, M.
Formulas for calculating deformations of a cross-shaped tower;
2026; Construction of Unique Buildings and Structures; 123 Article No 12301. doi: 10.4123/CUBS.123.1

The solution for the *y*-axis displacement differs significantly from the analogous displacement along the x-axis. The characteristic minimum deflection is absent, and the magnitude of the displacement is almost three times greater. The limiting value is also greater: $\lim_{n\to\infty} \Delta'_y / n^2 = 293/288$.

4 Conclusions

The following main conclusions can be made:

1. A new mathematical model of the stress state of a spatial rod tower has been developed.
2. Simple analytical relationships between the deflection and support reactions of a spatial truss and the number of panels were obtained.
3. Quadratic asymptotic solutions to the deflection problem were found.
4. Significant asymmetry in deflections with respect to the direction of load was detected.
5. The asymptotic behavior of vertical support reactions is quadratic and linear.

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