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Optimisation of steel trusses with a choice of multi-stage prestressing conditions

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Abstract. This paper presents a methodology for optimisation of flat steel trusses with a system of high-strength tie bars, for each of which the possibility of multiple pre-stresses is provided. Pre-stress is one of the most effective ways to increase the carrying capacity of steel trusses with minimum material costs. Besides, a significant effect, according to the literature data, can be achieved by alternating stages of pre-stresses and payloads. At the same time, algorithms for designing such objects while choosing a sequence of force actions still need to be worked out. The problem of minimising the truss cost is considered taking into account strength, stiffness, and stability constraints. A search has been specified for the sequence of alternating the stages of pre-stress and the application of portions of useful loads, pre-stress forces, bar profiles, and cable cross-section area values. Multiple allowable scenarios of force impacts on a redundant template are used. It is acceptable to indicate the load absence condition in such template in some positions. A possibility has been implemented for the cable system to set a redundant topology which is controlled by including the ability to select negligible cross-section areas. As a result, the mathematical statement of the problem is reduced to discrete parametrical optimisation. A scheme of genetic algorithm is implemented with a mixed approach to the mutation operator in order to find efficient solutions. A methodology has been developed to calculate the stress-strain state of steel frameworks pre-stressed using high-strength cables in a single computational process to efficiently check for compliance with the problem constraints. The performance of the suggested procedure of the optimum search has been illustrated by the example of a steel arch truss. A possibility to use bars made of round pipes together with high-strength tie bars was provided. The efficient parameters of the framework and force impact modes have been determined. The expediency of alternating impacts caused by pre-stresses of cables and application of useful load parts is confirmed. The approach proposed will significantly increase the possibility of obtaining cost-effective design solutions for steel trusses.

1. Introduction

Today attention is being increasingly focused on the development of unique structures, including various types of long span constructions. It is often required to use a pre-stress for strained systems of this kind so as to increase their load-carrying capacity while minimising the material costs. Such structure types can be designed efficiently on the basis of an optimum search. Herewith, it is expedient to vary their parameters and topologies on discrete sets of admissible variants determined via existing standards and construction conditions.

Classical approaches to the optimisation of pre-stressed steel structures are well-known [1–4]. These algorithms specify, first of all, the application of mathematical programming methods, and the optimum design process is divided into several stages. Work [1], on the basis of such methodology types, considers the issues of selecting rational structural layouts for pre-stressed systems, determination of rational pre-stress forces, and optimum distribution of material in the structure, unification of framework cross-section area values, and selection of the sequence of pre-stress operations. Article [2] presents stage-by-stage optimisation of pre-stressed flat steel trusses made of tubular profiles. Topological optimisation of the design and selection of bar cross-section areas from the regulatory requirements are performed during the first and second stages, respectively, without regard to pre-stress. At the third stage, the influence of pre-stress of a truss by means of cables built into its lower flange on the structure's load-carrying capacity is assessed, and the structure's

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parameters are adjusted. Work [3] covers the optimum design of steel cable trusses in which the cable pre-stress is provided. The truss weight is minimised at restrictions on stresses. The structure topology, pre-stress forces, and cable cross-section area values vary. Initially a uniform-strength system is used. Then the truss topology is corrected by providing an opportunity of selecting "zero" element cross-section area values. In [4] a procedure for minimising the weight of a flat framed structure subjected to pre-stress with restrictions on stresses and displacements is given. At the first stage, the pre-stress force and geometric characteristics of bars are determined. At the second stage, topological optimisation is implemented on the basis of exclusion of the least-loaded structural elements from the structure.

Stage-by-stage optimisation allows to simplify the problem, but, in some cases, it results in a loss of efficient design solutions. This problem can be solved on the basis of up-to-date achievements in the development of the optimum design of different complex engineering systems using metaheuristic iteration methods [5, 6]. Such approaches are efficient in finding global extrema, they do not require that derivatives of functions are considered, while ensuring the possibility of searching for variable parameters on discrete sets. Evolutionary metaheuristic procedures have gained fairly wide popularity for optimisation of deformable objects. They are usually implemented as genetic algorithms [7−11]. Some other metaheuristic methods were also used for this purpose: Particle Swarm Optimization [12], Simulated Annealing [13], Tabu Search [14], Harmony Search [15], Ant Colony Optimization [16], Big Bang – Big Crunch Algorithm [17], Imperialist Competitive Algorithm [18], Ray Optimization [19], Mine Blast Algorithm [20], Cuckoo Search Algorithm [21], Firefly Algorithm [22], Dolphin Echolocation [23], Teaching-Learning-Based Optimization [24], Chaotic Swarming of Particles [25], Bat-Inspired Algorithm [26], Colliding Bodies Optimization [27], Search Group Algorithm [28], etc. Quite detailed information on the use of metaheuristic procedures for optimisation of load-bearing systems is given in the reviews [29−31].

The research [32−35] uses metaheuristic schemes that allow to solve optimal search problems for pre-stressed frames according to all design parameters in a single computation process. Work [32] presents the procedure for optimisation using a genetic algorithm of pre-stressed reinforced concrete multi-span bridges. The selected parameters are the number of object spans, the bridge cross-section area, and the principal reinforcement parameters. Article [33] considers the optimum design of pre-stressed, pre-cast reinforced concrete bridges for pedestrians. A bridge structure with a framework, that includes a pre-stressed reinforced concrete beam with a U-shaped cross-section and a reinforced concrete slab, is examined. The bridge's cost is minimised in variation of beam and slab dimensions, material grades, as well as parameters of principal and design reinforcement. Herewith, a metaheuristic procedure based on a Simulated Annealing algorithm is used. Works [34, 35] describe the development of an evolutionary algorithm for the optimum design of pre-stressed steel flat trusses with system of tie bars consisting of high-strength cables. The truss minimisation problem is solved taking into account the strength, stiffness, and stability limitations. Simultaneously the search is performed both for bar profiles and for the cross-section area of tie bars and their pre-tension. Herewith, in article [34] the possibility of excluding tie-bars from the redundant cable system topology is considered.

Work [36] presents an optimisation algorithm using an ANSYS v12 software system for an innovative suspended structure with the main load-carrying elements as pre-stressed cable trusses. The flooring panels in such a framework are made of perpendicularly glued board layers and are located over the lower truss flange. The optimisation goal is to obtain the values of the design parameters while ensuring minimum material consumption. The stress and displacement limitations were taken into account. The cable cross-section areas and pre-stress forces were considered as variable parameters.

It should be noted that one of the important methods of using the possibilities of pre-stress is the implementation of alternating pre-stresses and useful load applications [1, 37]. At the same time, the methodology of the efficient design of objects in a single iteration procedure for such tasks is still to be adjusted as applied to various framework classes.

The aim of the present work is to develop of an algorithm to ensure the possibility of discrete optimisation on the basis of an evolutionary approach to pre-stressed steel flat trusses with a comprehensive selection of rational alternation of the stages of pre-stress and useful load application, cable structure, pre-stress forces, bar profiles, and cable cross-section areas.

2. Methods

2.1. Problem statement

Let us assume that the truss is fastened from its plane in nodes. We should take into account the tension-compression and flexure strains when dealing with bars, and tensile strain when dealing with cables. We provide for setting a redundant structure for the cable system in general case, and a redundant number of force impacts of different types. The control of these redundant possibilities is provided by taking into account parameters of conditional tie bar variants with negligible cross section areas and zero pre-stress forces and useful load portions in sets of permissible values. Then the topology and parametric optimisation is reduced to parametric one.

Let us minimise the design cost C:

$$C\left(\{V\}_1,\{V\}_2,...,\{V\}_{k_o},\{A\},\{H\}\right)\Rightarrow \min, \quad (1)$$

where

$$\{V\}_k=\left\{K_{g(k)}\ \ \left(n_{1(k)},\beta_{1(k)}\right)\ \left(n_{2(k)},\beta_{1(k)}\right)\ ..\ \left(n_{m(k)},\beta_{m(k)}\right)\ K_{p1(k)}\ \ K_{p2(k)}\ \ ..\ \ K_{ps_o(k)}\right\}^T$$

$\left(k=1,2,..,k_o\right)$ is the vector determining the action group k; k_o is the total number of possible impact groups; $K_{g(k)}$ is the portion of design gravity forces related to group k; $\left(n_{j(k)},\beta_{j(k)}\right)$ $\left(j=1,2,..,m\right)$ is a pair of numbers determining for group k the number $n_{j(k)}$ of tie bar T_j in the general numbering system of load-carrying elements and share $\beta_{j(k)}$ of the tension force increment at this stage of its total pre-stress; m is the maximum number of tie bars which may be used in the truss according to the problem statement; $K_{ps(k)}$ is the share of the useful load s for group k $\left(s=1,2,..,s_o\right)$; s_o is the number of useful loads; $\{A\}=\{A_1\ A_2\ ..\ A_r\}^T$ is a vector of independently variable areas A_l of cross sections for bar groups $\left(l=1,2,..,r\right)$; $\{H\}=\left\{\left(D_{T1},\alpha_{T1}\right)\left(D_{T2},\alpha_{T2}\right)..\left(D_{Tm},\alpha_{Tm}\right)\right\}^T$ is a vector of number pairs, each of which determines for tie bar T_j $\left(j=1,2,..,m\right)$ diameter D_{Tj} and share α_{Tj} of force S_j of its total pre-stress of the breaking load R_{Tj} $\left(S_j=\alpha_{Tj}R_{Tj}\right)$.

Let us take into account the following limitations the compliance with which is checked with consideration the requirements of standard “SP 16.13330.2017. Steel structures. The updated edition of SNiP II-23-81*” on each of the considered force impact stages:

1. Limitation on stresses in the truss bars:

$$\left|\sigma_M\right|\le R_y, \quad (2)$$

where σ_M is von Mises stress; R_y is the design steel resistance assigned on the basis of the yield strength.

2. Limitation on forces in the tie bars:

$$N_T\le R_T/k_T, \quad (3)$$

where N_T is the longitudinal force in any tie bar; R_T is the breaking load for the tie bar; k_T is the safety margin.

3. Limitation on stiffness:

$$\left|\delta\right|\le f, \quad (4)$$

where δ is a projection of the truss node displacement vector to any Cartesian coordinate system axes; f is the permissible value of the modulus of such displacement.

4. The bar stability condition.
5. Design and technological requirements.

2.2. Algorithm for calculating stress-strain state of a truss during multiple pre-stresses

Let us assume that each tie bar can be pre-stressed to the structure in several stages. We take into account the design non-linearity expressed by change of the framework structure as each new tie bar is included into it. n impacts are considered as tie bar pre-stresses and components of design loads. Let us discretise the object using the finite element method according to displacement method [38]. For each impact the calculation will be performed in a linear setting by solving the following linear algebraic equation system:

$$\left(\left[K \right] + \sum_{j=1}^{J_i} \left[K_T \right]_j \right) \left\{ \Delta_i \right\} = \left\{ R_i \right\}, \tag{5}$$

where $\left[K \right]$ is the global stiffness matrix for a truss without tie bars; J_i is a number of tie bars which shall be included into the design diagram of the ith impact; $\left[K_T \right]_j$ is an addition to the global stiffness matrix taking into account tie bar T_j; $\left\{ \Delta_i \right\}$ is a vector of nodal displacement increments caused by impact i; $\left\{ R_i \right\}$ is a vector of statically equivalent nodal forces for the ith impact.

After that the nodal displacement values are adjusted:

$$\left\{ \delta \right\}^{(i)} = \left\{ \delta \right\}^{(i-1)} + \left\{ \Delta_i \right\}, \tag{6}$$

where $\left\{ \delta \right\}^{(i)}$, $\left\{ \delta \right\}^{(i-1)}$ are the nodal displacement vectors obtained on the basis of the results of impacts i and $i-1$ $\left(\left\{ \delta \right\}^{(0)} = 0 \right)$.

Calculation of the structure pre-stress using tie bar T_j is taken into consideration in two stages using conditional temperature strains. Let us assume that with due allowance for losses, the tie bar shall get the positive increment ΔS_{jA} of the tension force. At the 1st stage we set the test change of temperature Δt in the tie bar resulting in the appearance of auxiliary nodal forces $F_{Oj}^{(1)}$ (Fig. 1), the modulus of which is

$$\left| F_{Oj}^{(1)} \right| = \alpha_T \left| \Delta t \right| E_T A_T, \tag{7}$$

where α_T, E_T, A_T are the linear temperature expansion coefficient, elasticity modulus, and cross section area of the cable.

The object is calculated taking into consideration only such forces with computation of longitudinal force $N_{Fj}^{(1)}$ in the tie bar which will work in compression conditionally. Then the actual increment of the longitudinal force in the tie bar is determined:

$$\Delta N_j^{(1)} = \left| F_{Oj}^{(1)} \right| + N_{Fj}^{(1)}, \tag{8}$$

where longitudinal forces are considered as algebraic values.

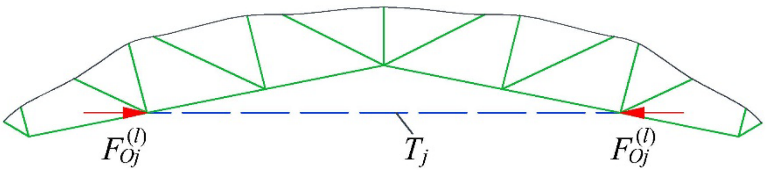

Figure 1. Auxiliary forces caused by conditional temperature effect on tie bar T_j at stage l $(l = 1, 2)$.

At stage 2 the value of auxiliary forces is adjusted:

$$\left| F_{Oj}^{(2)} \right| = \frac{\left| F_{Oj}^{(1)} \right| \Delta S_{jA}}{\Delta N_j^{(1)}}, \tag{9}$$

the truss calculation is repeated, and the increment of longitudinal force $\Delta N_j^{(2)}$ for this stage is determined.

Since the calculations within one impact are performed in a linear setting, we will have $\Delta N_j^{(2)} = \Delta S_{jA}$, fulfilling the condition of modelling the stress-strain state of the structure caused by pre-stress. If the tensioning is carried out to stops, then by setting force $F_{Oj}^{(1)} = \Delta S_{jA}$ we ensure modelling of the object state caused by the considered impact after only one calculation stage.

During each subsequent impact i force in tie bar T_j is re-calculated as follows:

$$N_{j(i)} = N_{j(i-1)} + \Delta N_{j(i)}, \quad (10)$$

where $N_{j(i)}$, $N_{j(i-1)}$ are tension forces in tie bar T_j before and after impact i; $\Delta N_{j(i)}$ is change of tension force in tie bar T_j, obtained due to impact i.

2.3. Optimum search procedure

Limitation 5 is taken into account when the permissible values of variable parameters are set. All other limitations are considered during optimisation as active ones. To achieve an optimum design, we use an approach to evolutionary modelling of frameworks in the form of a genetic algorithm the basic provisions of which are detailed in [39, 40]. Let us consider the main population Π_A with length L of chromosome and auxiliary population Π_B of elite individuals, the size of which depends on the results of operation of genetic algorithm, but does not exceed L. Population Π_B is used for keeping the efficient genetic material taken into account during replenishment of population Π_A. The limitations are considered by simple rejection of non-operable structure variants. A single-point crossover is implemented. We apply a mixed procedures ensuring random change of parameter values with alternation of selection from variants with the nearest number in the chromosome and from the elements which are randomly located in the chromosome. Table 1 explains the parameter value variation scheme, where p_a, p_b are numbers randomly generated on numeric line segment [0, 1] at uniform distribution law, r_j is the current position number in a set of its permissible values, w_j is the number of elements in such set, m_a, m_{b1}, m_{b2}, m_{b3} are the specified values. During optimisation of truss structures it is expedient to assume $m_a = 0.9$, $m_{b1} = 0.5$, $m_{b2} = 0.75$, $m_{b3} = 0.9$ [40].

Table 1. Scheme of parameter value change during mutation.

Condition on p_a	Condition on p_b	Condition on r_j	Parameter value selection method
$p_a \le m_a$	$p_b < m_{b1}$	$r_j \ge 3$	$r_j = r_j - 2$
		$r_j = 2$	$r_j = r_j - 1$
	$m_{b1} \le p_b < m_{b2}$	$r_j \ge 2$	
	$m_{b2} \le p_b < m_{b3}$	$r_j \le w_j - 1$	$r_j = r_j + 1$
	$p_b \ge m_{b3}$	$r_j = w_j - 1$	
		$r_j \le w_j - 2$	$r_j = r_j + 2$
$p_a > m_a$	is not taken into consideration	is not taken into consideration	The value is randomly selected from a set of permissible values

It is assumed in this task that the chromosome can contain the following information in the general case:

$$[G] = [\zeta \quad A_1 \quad A_2 \quad .. \quad A_r \quad (D_{T1}, \alpha_{T1}) \quad (D_{T2}, \alpha_{T2}) \quad .. \quad (D_{Tm}, \alpha_{Tm})], \quad (11)$$

where ζ is the number of scenario of impact on the truss.

It should be noted that for exhaustive type problems, the only reliable criteria for achieving the global optimum is usually the complete exhaustion of all possible variants associated with a significant number of calculations. Numerical experiments show that at the optimum synthesis of pre-stressed steel trusses on the basis of the approach presented herein, the absence of changes in the group of elite projects for 500–600 iterations indicates the expediency of completion of the evolutionary search. Usually the continuation of such process does not result in any significant change to the objective function.

3. Results and Discussion

Let us provide the results of designing the steel flat truss shown in Fig. 2. It was assumed that the bars are made of round pipes in accordance with Russian State Standard GOST 32931-2015 “Steel shaped tubes for steel structures. Specifications.” The truss bar material is steel S245 (SP 16.13330.2017). A provision was made to introduce up to two high-strength cable-type tie bars in accordance with GOST 3081-80 “Two lay rope of LK-O type, design 6 19 (1+9+9)+7×7 (1+6).” In accordance with SP 16.13330.2017, the elasticity modulus for the bar material was assumed as $E = 2.06 \cdot 10^5$ MPa, the elasticity modulus of rope material was assumed as $E_T = 1.47 \cdot 10^5$ MPa. In accordance with SP 20.13330.2016 “Loads and impacts. Updated edition of SNiP 2.01.07-85*,” the maximum permissible deflection for 1/300 of the truss span was set. The cost of rolled metal products was calculated using the cost of metal pipes of Metallokonstruktsii LLC (http://www.metalconstr.ru/09.html), the cost of tie bars manufactured by Optimist LLC (http://optimist-32.ru/produktsiya/kanaty-i-veryovki/kanat-stalnoj/). The prices taken into account were current as of May 2019.

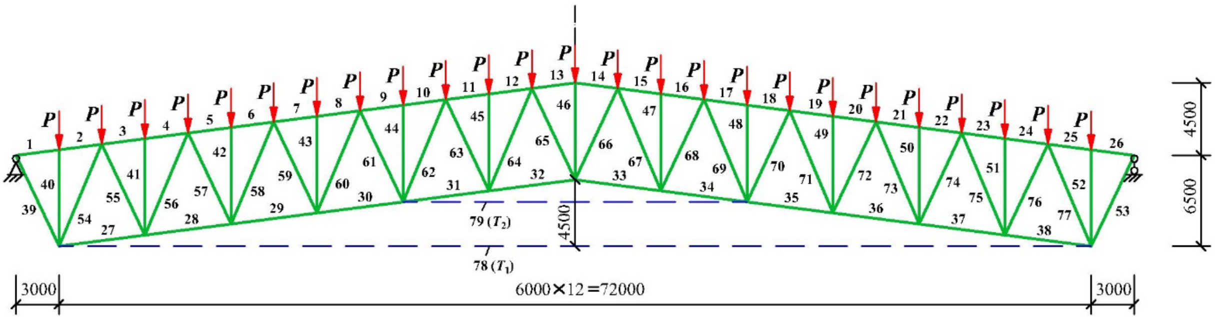

Figure 2. Long-span truss: 1–77 are bars, 78 (T_1), 79 (T_2) are tie bars.

Permitted possibilities included using two tie bars T_1, T_2 during tensioning on the structure, and the absence of one or both ropes. Weights of bars and ropes depend on the parameter values and useful load as a system of concentrated forces applied to the upper truss flange, where $P = 30$ kN were taken into consideration. The impact sequence was considered at $k_o = 2$, $s_o = 1$ based on the following template:

$$\{V\}_1 = \left\{1 \ \left(n_{1(1)}, \beta_{1(1)}\right) \ \left(n_{2(1)}, \beta_{2(1)}\right) \ K_{p1(1)}\right\}^T,$$
$$\{V\}_2 = \left\{0 \ \left(n_{1(2)}, \beta_{1(2)}\right) \ \left(n_{2(2)}, \beta_{2(2)}\right) \ K_{p1(2)}\right\}^T. \qquad (12)$$

This template shows that initially the object gravity forces are taken into account, then the consecutive rope tensioning and useful load application can be performed. Then a provision is made to finally tension the ropes and take into consideration the remaining portion of the useful load.

The set of permissible impact scenarios was specified (Table 2). The bar grouping is detailed in Table 3. For each group a possibility was specified to use the following values of the pipe outer diameter and thickness $(D \times t)$: 70×4, 89×5.5, 127×5.5, 159×6, 177.8×8, 219×8, 219×9, 219×10, 219×12, 219×13, 219×14, 273×12, 273×14, 325×13, 325×14, 355.6×14, and 377×14 (mm). Couples of numbers $\left(D_{Tj} \times \alpha_{Tj}\right)$ for ropes were chosen from these options: (33, 0.6), (33, 0.5), (33, 0.4), (33, 0.3), (33, 0.2), (33, 0.1), (33, 0), (31, 0.6), (31, 0.5), (31, 0.4), (31, 0.3), (31, 0.2), (31, 0.1), (31, 0), (29.5, 0.6), (29.5, 0.5), (29,5, 0.4), (29,5, 0.3), (29.5, 0.2), (29.5, 0.1), (29.5, 0), (27, 0.6), (27, 0.5), (27, 0.4), (27, 0.3), (27, 0.2), (27, 0.1), (27, 0), (25, 0.6), (25, 0.5), (25, 0.4), (25, 0.3), (25, 0.2), (25, 0.1), (25, 0), and (0, 0) (mm, -). Condition $D_{Tj} = 0$ corresponds to absence of rope. Case $\alpha_{Tj} = 0$ and/or $\beta_{j(i)} = 0$ indicates that the rope was only subjected to small technological tensioning. If $D_{Tj} = 0$ is considered for a tie bar, then its pre-stress order set in scenarios is not taken into consideration. Thus, the specified possibilities include both one-time and two-time rope tensioning, setting ropes without any significant tensioning, exclusion of ropes from the redundant topology of the framework.

Table 2. Scenarios of impact on truss.

Scenario	$n_{1(1)}$	$\beta_{1(1)}$	$n_{2(1)}$	$\beta_{2(1)}$	$K_{p1(1)}$	$n_{1(2)}$	$\beta_{1(2)}$	$n_{2(2)}$	$\beta_{2(2)}$	$K_{p1(2)}$
1	78	0.7	79	0.7	0.7	78	0.3	79	0.3	0.3
2	78	0.7	79	0.7	0.5	78	0.3	79	0.3	0.5
3	78	0.7	79	0.7	0.3	78	0.3	79	0.3	0.7
4	78	0.7	79	0.7	0.1	78	0.3	79	0.3	0.9
5	78	0.5	79	0.5	0.7	78	0.5	79	0.5	0.3
6	78	0.5	79	0.5	0.5	78	0.5	79	0.5	0.5
7	78	0.5	79	0.5	0.3	78	0.5	79	0.5	0.7
8	78	0.5	79	0.5	0.1	78	0.5	79	0.5	0.9
9	78	0.3	79	0.3	0.7	78	0.7	79	0.7	0.3
10	78	0.3	79	0.3	0.5	78	0.7	79	0.7	0.5
11	78	0.3	79	0.3	0.3	78	0.7	79	0.7	0.7
12	78	0.3	79	0.3	0.1	78	0.7	79	0.7	0.9
13	78	0.1	79	0.1	0.7	78	0.9	79	0.9	0.3
14	78	0.1	79	0.1	0.5	78	0.9	79	0.9	0.5
15	78	0.1	79	0.1	0.3	78	0.9	79	0.9	0.7
16	78	0.1	79	0.1	0.1	78	0.9	79	0.9	0.9
17	78	0	79	0	0.7	78	1	79	1	0.3
18	78	0	79	0	0.5	78	1	79	1	0.5
19	78	0	79	0	0.3	78	1	79	1	0.7
20	78	0	79	0	0.1	78	1	79	1	0.9
21	78	0	79	0	0	78	1	79	1	1
22	79	0.7	78	0.7	0.7	79	0.3	78	0.3	0.3
23	79	0.7	78	0.7	0.5	79	0.3	78	0.3	0.5
24	79	0.7	78	0.7	0.3	79	0.3	78	0.3	0.7
25	79	0.7	78	0.7	0.1	79	0.3	78	0.3	0.9
26	79	0.5	78	0.5	0.7	79	0.5	78	0.5	0.3
27	79	0.5	78	0.5	0.5	79	0.5	78	0.5	0.5
28	79	0.5	78	0.5	0.3	79	0.5	78	0.5	0.7
29	79	0.5	78	0.5	0.1	79	0.5	78	0.5	0.9
30	79	0.3	78	0.3	0.7	79	0.7	78	0.7	0.3
31	79	0.3	78	0.3	0.5	79	0.7	78	0.7	0.5
32	79	0.3	78	0.3	0.3	79	0.7	78	0.7	0.7
33	79	0.3	78	0.3	0.1	79	0.7	78	0.7	0.9
34	79	0.1	78	0.1	0.7	79	0.9	78	0.9	0.3
35	79	0.1	78	0.1	0.5	79	0.9	78	0.9	0.5
36	79	0.1	78	0.1	0.3	79	0.9	78	0.9	0.7
37	79	0.1	78	0.1	0.1	79	0.9	78	0.9	0.9
38	79	0	78	0	0.7	79	1	78	1	0.3
39	79	0	78	0	0.5	79	1	78	1	0.5
40	79	0	78	0	0.3	79	1	78	1	0.7
41	79	0	78	0	0.1	79	1	78	1	0.9
42	79	0	78	0	0	79	1	78	1	1

Table 3. Bar grouping.

Group	Numbers of bars in the group	Group	Numbers of bars in the group
1	1, 26	10	41, 42, 50, 51
2	2, 3, 4, 5, 22, 23, 24, 25	11	43, 44, 48, 49
3	6, 7, 8, 9, 18, 19, 20, 21	12	45, 47
4	10, 11, 12, 13, 14, 15, 16, 17	13	46
5	27, 28, 37, 38	14	54, 56, 58, 73, 75, 77
6	29, 30, 35, 36	15	60, 62, 64, 67, 69, 71
7	31, 32, 33, 34	16	55, 57, 59, 72, 74, 76
8	39, 53	17	61, 63, 65, 66, 68, 70
9	40, 52	–	–

30 independent optimisation process runs were carried out with implementation of 15,000 iterations of evolutionary algorithm in one run. Based on the results of the search, 3 different structure variants were found, in each of which the algorithm left one rope. The information about obtained results is provided in Tables 4 and 5. As Table 4 shows, the lowest cost is achieved in the 1st structure variant obtained in 2 runs. Herewith, the lower tie bar T_1 is left at $D_{T1} = 33$ mm, and two possible impact scenarios are selected, each of which stipulates two-stage application of useful load (see Table 2). At the same time, scenario 22 stipulates the two-time pre-stress of the rope, and scenario 39 stipulates initial introduction of the rope without any significant pre-stress and actual one-time pre-stress between stages of impact by useful load. The second structure variant achieved in 26 runs is associated with ten impact scenarios and stipulates the use of only tie-bar T_1, but at $D_{T1} = 31$ mm. In the third variant which was implemented in two runs with identical impacts, only the upper tie bar T_2 is left at $D_{T2} = 25$ mm.

Table 4. Results of optimisation on objective function, ropes and impact conditions.

Structure variant number	C, thous. RUB.	Remaining tie bar	D_{Tj}, mm	α_{Tj}	S_j, kN	Scenario number	Number of runs
1	689.7	T_1	33	0.4	248.6	22	1
						39	1
2	721.1	T_1	31	0.4	168.45	34	7
						39	3
						38	4
						36	4
						26	1
						27	2
						23	1
						35	1
						22	2
						30	1
3	870.6	T_2	25	0.5	174.0	17	2

Table 5. Results for bar profiles.

Bar group	$D \times t$, mm for structure variants		
	Variant 1	Variant 2	Variant 3
1	127×5.5	127×5.5	127×5.5
2	177.8×8	177.8×8	219×8
3	219×8	219×9	219×12
4	219×8	219×9	219×12
5	127×5.5	127×5.5	177.8×8
6	159×6	177.8×8	219×9
7	177.8×8	177.8×8	219×8
8	127×5.5	127×5.5	159×6
9	89×5.5	89×5.5	89×5.5
10	89×5.5	89×5.5	89×5.5
11	89×5.5	89×5.5	70×4
12	89×5.5	89×5.5	89×5.5
13	159×6	127×5.5	89×5.5
14	219×8	219×8	219×8
15	159×6	159×6	177.8×8
16	89×5.5	127×5.5	127×5.5
17	89×5.5	70×4	70×4

The behaviour of decrease of the objective function in the developed computational scheme is illustrated in Fig. 3 by examples of runs with the best and the worst convergence for each of the obtained structure variants. As a whole, for all 30 runs in the best case, in terms of convergence, the considered result was achieved by iteration 154, in the worst case – by iteration 10,012.

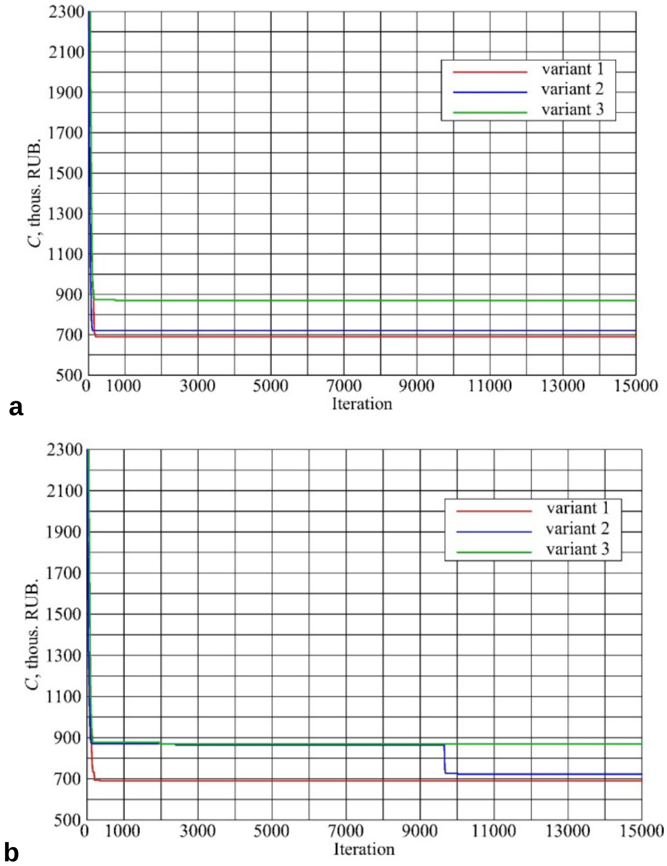

Figure 3. Change of the objective function during optimisation:
a – best convergences, b – worst convergences.

In none of 30 runs impact scenario 42 was accepted, stipulating one-time pre-stress of tie bars and subsequent application of all useful load. At the same time, this scenario, according to its position in the set of permissible impact combinations, was accepted by us for all individuals of the initial population. This result corresponds to the provision of principal efficiency of alternating impacts caused by the application of useful load portions and shares of tie bar pre-stresses [1, 37].

It should be noted that the optimum search organisation variant with the setting of permissible scenarios suggested herein allows the designer to significantly take into consideration the peculiarities of real civil construction conditions. The developed methodology may become the basis for expanding the use of multi-stage pre-stresses in building structures, because it permits automation of the framework development process with such control of force impacts.

4. Conclusions

1. We have suggested a computational scheme for the evolutionary optimisation of pre-stressed steel truss structures with the possibility to vary the system of high-strength tie bars, pre-stress sequence and application of useful loads, pre-stress forces, bar profiles and tie bar cross section areas on discrete topology sets. The active limitations in terms of strength, stiffness, and stability are taken into consideration. Design and process requirements are taken into account during the creation of sets of permissible parameter values.

2. A methodology has been developed for calculation of the stress-strain state of steel trusses subjected to pre-stress using a rope system in a single computational process. The design's non-linearity associated with the change of the framework structure during inclusion of each new tie bar is taken into consideration.

3. The operability of the suggested computational scheme of optimum design has been confirmed by the example of a pre-stressed steel truss with 78 m span tie bars. As a result of the performed optimisation processes, three structure variants have been obtained which stipulate alternating tie bar pre-stresses and the application of useful load shares.

4. The research results may be used for the design and reconstruction of pre-stressed structure systems in civil construction and primarily for unique structures. It is expedient to implement the basic provisions of the developed algorithms in finite element analysis software application packages.

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