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Structural Optimization of 3D Trusses considering the Dynamic Effect of the Wind

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Abstract. In the civil engineering and construction industry there exists a consistent search for projects that are more functional, safe and efficient. Based on those exigencies, new technologies and analysis methods are being developed, where the common objective is to find the optimal structure in terms of lightweight and efficiency. The use of trusses-kind structures has increased over recent years, this fact is mainly due to its practicity in terms of structural assembly, lightness and high resistance; allowing, for example, its use in large roof structures or stadiums. Nonetheless, an accurate analysis of its behavior is necessary for correctly design these kind of structures. In many cases, a static analysis is sufficient. However, in structures such as roofs or bridges, the dynamic effects could be relevant, as they can generate vibration. Resulting in discomfort for the users, or even, leading to structural failure. Thus, the dynamic effect of the wind should be considered in order to have a more realistic behavior of the structural response. The main objective of the present work is the parametric optimization of 3D trusses, considering the dynamic effect of the wind. The brazilian standard NBR 6123 is herewith used in order to account for this dynamic wind effect. Restrictions are also in accordance with the brazilian standard NBR8800, which features project restrictions for steel structures. The computational implementation was made using Matlab programming language. The structural dynamic response was implemented using the Finite Element Method (FEM), while the optimization procedure was accounted using the Teaching-Learning-Based Optimization (TLBO) method. The objective function for the optimization method is the reduction of the total weight of the structure and project restrictions were considered in accordance with brazilian standards. Some numerical examples are presented to show the applicability of the proposed methodology.

Keywords: Wind Dynamic Effect, Genetic Algorithms, Finite Element Method (FEM), Brazilian Standards.

1 Introduction

Optimization is the process of minimize or maximize a specific objetive function, while fulfilling some predefined restrictions. Optimization is of great interest in the industrial area, this fact is due to the gain that is obtained in terms of material use. Its industrial application has been greatly boosted by scientific research and computational growth, simplifying the analysis of more complex structural systems with celerity and efficiency.

The success of the optimization is strictly subordinated to a robust and adequate formulation of the problem. It is of great importance the correct definition of the mathematical/physical model and of optimization parameters in order to achieve satisfactory results. There exists many optimization algorithms available in the literature, however, no single algorithm is suitable for all problems. Moreover, the search for efficient algorithms still forms a major effort among researchers. The search for the *perfect algorithm* will continue unless some proves in an analytical way otherwise [1].

Generally, structural optimization problems are of great complexity in terms of the wide search space and design constraints. In these cases, metaheuristic algorithms are preferred to gradient based methods [2]. There exist a wide number of nature-inspired optimization algorithms such as the Genetic Algorithm (GA), Particle Swarm Optimization (ACO), Harmony Search (HS), among others. Genetic algorithms (GAs) are a class of algorithms based on Darwinian evolution of biological systems. It was firstly proposed by Holland [3] in 1975, however, the original method has been widely explored and modified to account for several type of problems. In general, GAs use genetic operators such as *crossover*, *recombination*, *mutation* and *selection*. Thus, the main problem with GAs is the difficulty in the determination of these parameters. Rao et al. [4] recently introduced an innovative approach called Teaching-Learning-Based Optimization (TLBO). The TLBO is based on the effect of the influence of a better individual (teacher) on the ouput of a given population (learners). The main advantage is that, different from GAs, TLBO does not rely on the value of any control parameter to direct its search, being quite straightforward to implement and to adapt to a wide range of numerical problems. The present work uses the TLBO for 3D truss structures for weight optimization, while considering the dynamic effect of the wind.

First, a brief theoretical background is presented. The dynamic wind effect is accounted by acting in accordance with the brazilian standard NBR 6123 [5], while the optimization method is the TLBO, as presented by Rao et al. [4] and modified by Camp et al. [2]. In such a way, main features of both theories are herewith exposed. Finally, examples are shown in order to validate and confirm the applicability of the methodology.

2 Theoretical Background

2.1 Wind Simplified Dynamic Effect

The effect of the wind occurs randomly on buildings, affecting all horizontal directions. Thus, taking as a big reference for the study of most critical situations for the structure [6].

According to Chávez [7], the study of the wind action in buildings should consider static solicitation, which depends on its mean velocity and fluctuations. These fluctuations are the gusts or turbulences that give rise to vibrations due to the various ways in which their force acts on the structure, producing a short term random loading that makes direct stress analysis difficult.

Loads from the wind effect can cause dynamic effects on the structures and these actions can damage the structure, reaching service limit states and ultimate limit states due to excessive vibration or material fatigue, and may also cause discomfort to building users [8].

The brazilian standards NBR 6123 [5] regulates the study of the effects of winds on structures and specifies the conditions required to consider the forces due to static and dynamic wind action, for the purpose of building calculation.

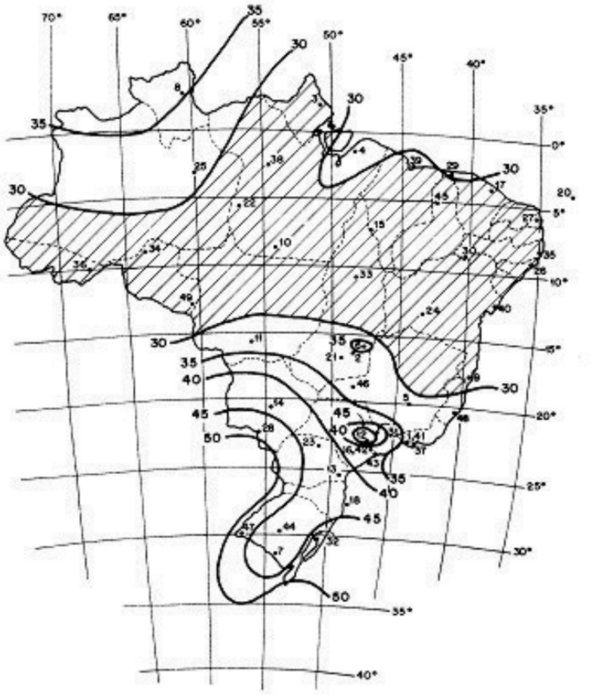


Figure 1. Basic velocity of the wind [9].

Simplified dinamyc effect of the wind by the ABNT NBR 6123:1988

The velocity fluctuations can cause flexible structures, especially tall and slender constructions, to shift in the direction of the mean velocity (floating response). In constructions with a fundamental period longer than 1 s, in particular those that are weakly damped, a fluctuating response in the mean wind direction may be present. The total dynamic response equals the superposition of the mean and fluctuating responses [5]. Even if the models presented by the standard are referred to as dynamic, wind loading is only considered as static loading.

The basic wind velocity (V_0), obtained from Figure 1, is defined as the velocity of a 3 s gust, exceeded on average once in 50 years, 10 m above ground, in the field open and flat field. It is admitted that the basic wind can blow from any horizontal direction [5].

The topographic factor (S_1) considers the topographic characteristics of the land on which the construction will take place. This coefficient is 1.0 for flat or slightly rugged locations, 0.9 for deep wind-protected valleys, and has a variation for slope-side buildings [10].

The statistical factor (S_3) considers five groups for the required degree of safety and the useful life of the building, Table 1.

From the values of V_0, $S1$ and $S3$ is possible to calculate the design velocity (V_p) of the wind, in (m/s), according to Equation (1):

$$V_p = 0,69 V_0 S_1 S_3 \tag{1}$$

The dynamic pressure $q(z)$, from which forces are obtained, is a continuous function of height above ground and the first term of the expression corresponds to the mean response, while the second is the maximum amplitude of the floating response. The dynamic pressure is expressed through Eq.(2):

$$q(z) = q_0 b^2 \left[\left(\frac{z}{z_r}\right)^{2p} + \left(\frac{h}{z_r}\right)^{p} \left(\frac{z}{h}\right)^{\gamma} \frac{1+2\gamma}{1+\gamma+p}\xi \right] \tag{2}$$

The basic pressure at the reference height, in N/m^2, can be determined according to Eq.(3):

$$q_0 = 0,613 V_p^2 \tag{3}$$

Group	Description	S_3
1	Buildings whose complete or partial ruin may affect the safety or possibility of help of people following a destructive storm, such as hospitals.	1,10
2	Buildings for hotels and residences. Buildings for commerce and industry with high occupancy factor.	1,00
3	Low occupancy industrial buildings and facilities such as rural buildings.	0,95
4	Fences such as tiles, glass, fence panels.	0,88
5	Temporary buildings. Group 1 to 3 structures during construction.	0,83

Table 1. Minimum values of the statistical factor [5].

The values of the dynamic pressure coefficients are determined according to Table 2 and the exponent γ represents the modal form dependent on the type of construction. The coefficient of dynamic amplification (ξ) can be obtained by means of abacuses presented in the standard and is a function of the dimensions of the building, the critical damping ratio (ζ) and the frequency (f). Coefficients b and p depend on the roughness of the terrain and are determined by Table 3. z_r is the reference height at 10 m, h is the height of the building above the ground measured to the top and z the height above the ground in each coordinate.

Type of construction	γ	ζ	$T_1 = 1/f_1$
Concrete frame buildings, without curtains.	1,2	0,020	0,05h + 0,015h (h in meters)
Concrete frame buildings with curtains for absorbing horizontal forces.	1,6	0,015	0,05h + 0,012h
Concrete towers and chimneys, variable section.	2,7	0,015	0,02h
Concrete towers, masts and chimneys, uniform section.	1,7	0,010	0,015h
Welded steel structure buildings.	1,2	0,010	0,29$\sqrt{h}$ - 0,4
Steel towers and chimneys, uniform section.	1,7	0,008	-
Wooden structures.	-	0,030	-

Table 2. Parameters for determining dynamic effects.

Roughness category	I	II	III	IV	V
p	0,095	0,15	0,185	0,23	0,31
b	1,23	1,00	0,86	0,71	0,50

Table 3. Coefficients p and b, [5].

The force on the structure can be obtained from the product between the dynamic pressure, drag coefficient and area of influence of the building according to Equation (4). The drag coefficient is established according to graphs of the standard.

$$F(z) = q(z)C_a(z)A(z) \tag{4}$$

2.2 Teaching-Leaning-Based Optimization (TLBO)

The Teaching-Leaning-Based Optimization (TLBO) was first defined by Rao et al. [4]. One of the advantages of the TLBO is its versality to account for different kind of numerical problems, this fact is due to the necessity of defining just a few number of parameters when compared with other meta-heuristic methods. The main parameter, which affects directly the performance of the algorithm, is the size of the population [2, 4]. As pointed out by Camp et al. [2], a population of 75 students (also used in the present work) is adequate to provide good results while balancing computational efficiency and algorithm perfomance. The population size is also compatible with several references in literature [11–14].

The process is divided in two parts. The first, called the *Teacher Phase*, in which the individuals learn from a better prepared individual (teacher). The second is called the *Learner Phase*, in which individuals (learners) learn from interaction among them. Figure 2 shows a flowchart describing both phases.

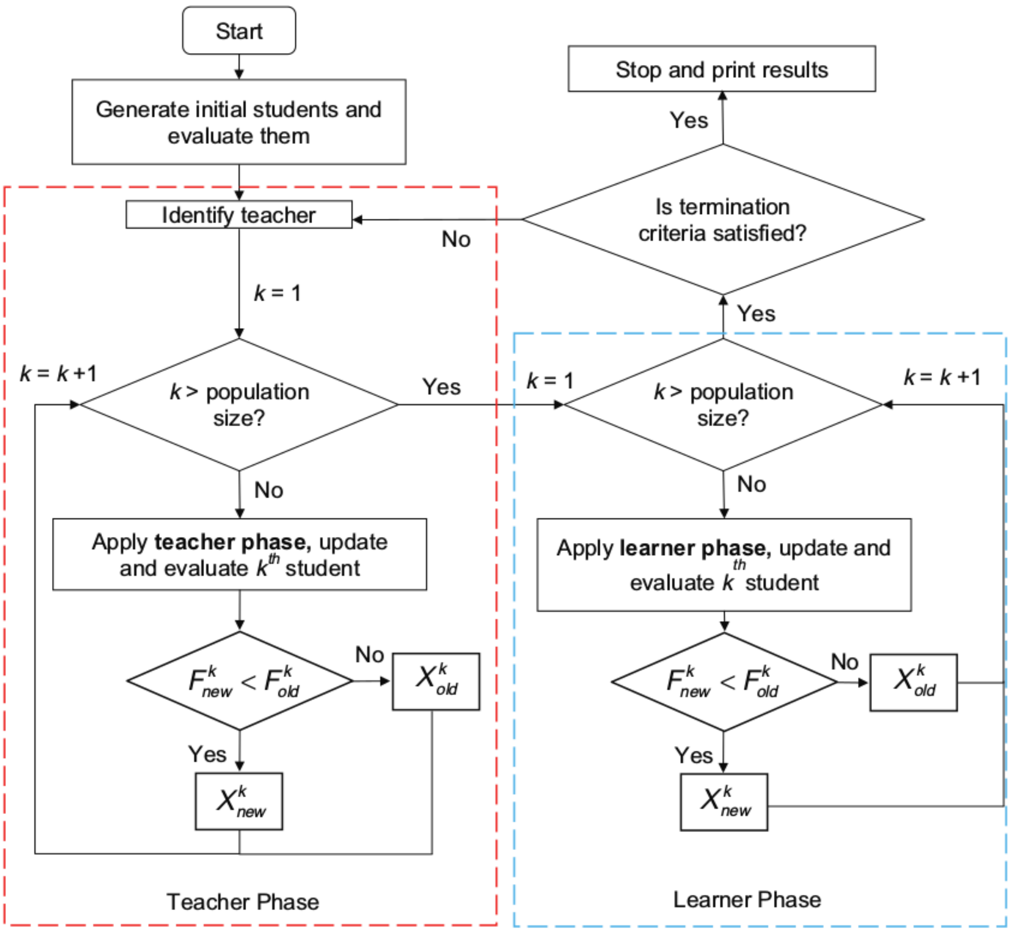


Figure 2. Algorithm flowchart for the TLBO, as presented by Camp et al. [2].

Teacher Phase

In this phase, the teacher, which is the best individual from a population, transfer knowledge to the rest of the population. This knowledge is distributed among all design variables, that can be considered as being different subjects in a course. In this process, students try to update their knowledge based on information provided by the teacher. Camp et al. [2] defines this phase mathematically as,

$$X^k_{new}(j) = X^k_{old}(j) \pm T_F \times r|M(j) - T(j)|, \tag{5}$$

where $X^k(j)$ denotes the design variable in position j, corresponding to the design vector k; T_F is a teaching factor; r is a random number within the range of $[0, 1]$; $M(j)$ is the mean of the class; and $T(j)$ is the state of the teacher. The second term of the right in Equation (5) represents the difference between the teacher and the class mean. Its sign should be selected in order to induce always the approximation towards the teacher.

The optimization process is highly dependent in how the mean of the class is obtained. In the original formulation proposed by Rao et al. [4], the mean was calculated in the traditional way. However, Camp et al. [2] presented a weighted mean based on the individual fitness value,

$$M(j) = \frac{\sum_{k=1}^{N} \frac{X^k(j)}{F^k}}{\sum_{k=1}^{N} \frac{1}{F^k}}, \tag{6}$$

where F^k is the individual fitness value of the student k. This procedure gives more emphasis to highly qualified student, further improving the perfomance of the TLBO algorithm.

Learner Phase

In this phase, students learn from each other by interaction. Learners interact randomly with other learner passing knowledge from best prepared individuals to the less prepared. This process is performed in pairs. The procedure was summarized by Camp et al. [2] on the following steps:

(a) Randomly select an individual p from the class.

(b) Ramdomly select another individual q ($q \neq p$).

(c) Evaluate the fitness of each student.

(d) If $F^p < F^q$ (p is better than q), then

$$X^p_{new}(j) = X^p_{old}(j) + r\left[X^p_{old}(j) - X^q(j)\right], \tag{7}$$

otherwise

$$X^p_{new}(j) = X^p_{old}(j) + r\left[X^q(j) - X^p_{old}(j)\right], \tag{8}$$

where r is a random number within the range of $[0, 1]$.

(e) The process continues until N student pairs have been selected.

3 Results

Three numerical examples are presented to show the application of the proposed optimization methodology. The first two examples are well-known benchmark examples for the validation of optimization methods. In the first example, a ten-bar planar truss structure is tested under two load conditions. The optimization is accounted considering continuous areas. In the second example, a 25-bar space truss structure is analyzed. For this example, bars are arranged in project sets. Components of the same set share the same area. In this case, the optimization is achieved considering discrete areas. Finally, the last example present the optimization of a realistic tower structure. In this example, the wind dynamic effect is accounted using the simplification proposed by the brazilian standard NBR 6123 [5]. Moreover, discrete areas are selected from commercial profiles.

3.1 Ten-Bar Planar Truss Structure

The Ten-Bar truss structure is a well known benchmark problem for the optimization analysis of planar trusses, Figure 3. Results for this truss design have been presented by many researchers, such as Schmit and Farshi [15], Lee and Geem [16], Li et al. [17] and Camp et al. [2]. The material density is 0.1 lb/in^3 and the Elasticity modulus is 10,000 ksi. The design variable is the cross-sectional area A, which has continous values between 0.1 and 35.0 in. Two load cases were considered, as can be seen in Table 4.

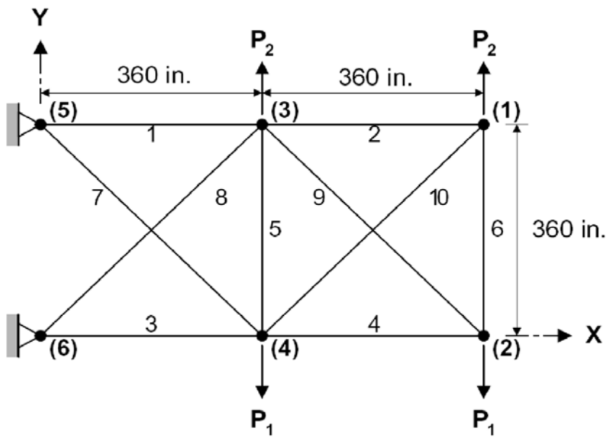


Figure 3. Ten-Bar Plane truss problem, [16].

Case	Restrictions
I	Nodal displacements restricted to $\pm$ 2.0 in; Normal Stress restricted to $\pm$ 25.0 ksi; $P_1 = 100$ kips and $P_2 = 0$.
II	Nodal displacements restricted to $\pm$ 2.0 in; Normal Stress restricted to $\pm$ 25.0 ksi; $P_1 = 150$ kips and $P_2 = 50$ kips.

Table 4. Restrictions and load values for both cases.

The stopping criterion adapted was a total of 2000 analyses without overall change of the best feasible design ($\epsilon_d \leq 10^{-6}$). In order to estimate the general performance of the optimization, the process was run 20 times. For the first case, the mean value of the optimized design was 5065.11 lb, while for the second case, the mean value was 4677.66 lb. Typical convergence history diagrams for both cases are shown in Figure 4. Table 5 compares the best design obtained from the present study to others obtained in the literature.

Element	Cross-sectional areas (in^2)							
	Case I				Case II			
	Schmit [15]	Lee [16]	Li [17]	Present	Schmit [15]	Lee [16]	Li [17]	Present
1	33.430	30.150	30.704	30.451	24.290	23.250	23.353	23.500
2	0.100	0.102	0.100	0.100	0.100	0.102	0.100	0.100
3	24.260	22.710	23.167	23.232	23.350	25.730	25.502	25.338
4	14.260	15.270	15.183	15.254	13.660	14.510	14.250	14.344
5	0.100	0.102	0.100	0.100	0.100	0.100	0.100	0.100
6	0.100	0.100	0.100	0.546	1.969	1.977	1.972	1.970
7	8.388	7.541	7.460	7.444	12.670	12.210	12.363	12.381
8	20.740	21.560	20.978	20.977	12.540	12.610	12.894	12.832
9	19.690	21.450	21.508	21.612	21.970	20.360	20.356	20.341
10	0.100	0.100	0.100	0.100	0.100	0.100	0.101	0.100
Weight (lb)	5089.00	5057.88	5062.92	5060.90	4691.84	4668.81	4677.29	4676.94

Table 5. Truss optimal design comparison for the Ten-bar plane truss.

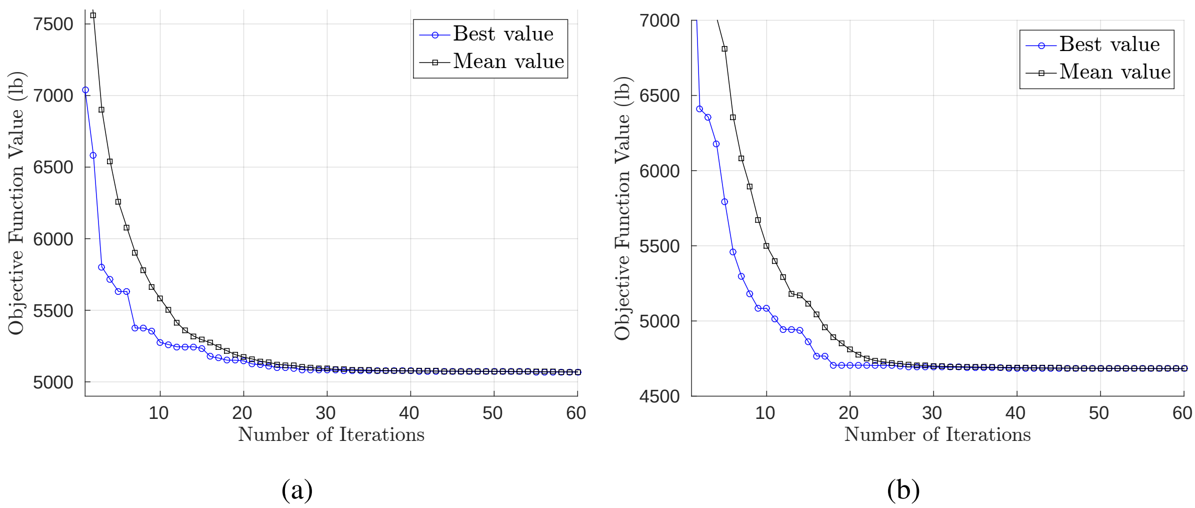


Figure 4. Typical convergence history of the ten-bar truss: Case I (a), Case II (b).

3.2 25-Bar Space Truss Structure

The 25-bar space truss structure is also a well known benchmark problem for the optimization analysis of space trusses. This optimization problem has been widely studied on literature. Chao et al. [18] used Quadratic Programming (QP), Rajeev and Krishnamoorthy [19] and Cao [20] used modified algorithms based on Genetic Algorithms (GA), Camp and Bichon [11] used Particle Swarm Optimization (PSO), Li et al. [17] used Heuristic Particle Swarm Optimization (HPSO), among others. Figure 5 shows the geometry and node numbering, while Table 6 shows the node coordinates of the problem.

The structure of the 25-bar tower is distributed into eight design groups. All the members of each group should share the same material and cross-sectional properties. Table 8 shows the elements of each group, as well as the nodal conectivity of those elements. The material has a density of 0.1 lb/in^3 and an Elasticity modulus of 10,000 ksi. The design variable is the cross-sectional area A, which has discrete values between 0.1 and 3.4 in^2 with a 0.1 in^2 increment. The maximum stress for each member is ± 40

ksi, while the maximum allowable displacement is ± 0.35 in the x, y and z directions. Moreover, the following loads were applied into the structure: $P_1 = (1.0, -10.0, -10.0)$ kips, $P_2 = (0.0, -10.0, -10.0)$ kips, $P_3 = (0.5, 0.0, 0.0)$ kips and $P_6 = (0.6, 0.0, 0.0)$ kips, applied on nodes 1, 2, 3 and 6, respectively.

Node	x (in)	y (in)	z (in)
1	-37.5	0.0	200.0
2	37.5	0.0	200.0
3	-37.5	37.5	100.0
4	37.5	37.5	100.0
5	37.5	-37.5	100.0
6	-37.5	-37.5	100.0
7	-100.0	100.0	0.0
8	100.0	100.0	0.0
9	100.0	-100.0	0.0
10	-100.0	-100.0	0.0

Table 6. Coordinates for the 25-bar space truss.

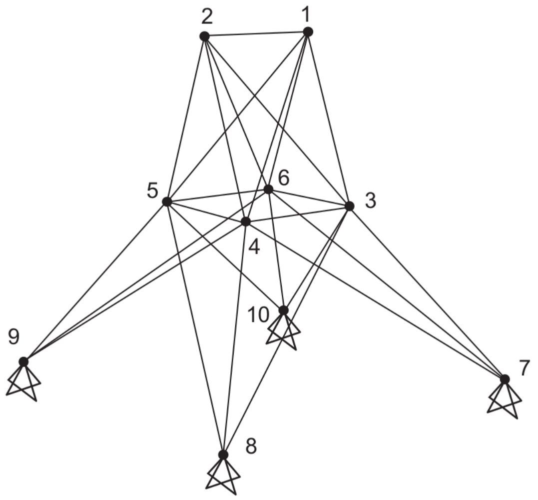


Figure 5. Geometry of the 25-bar space truss.

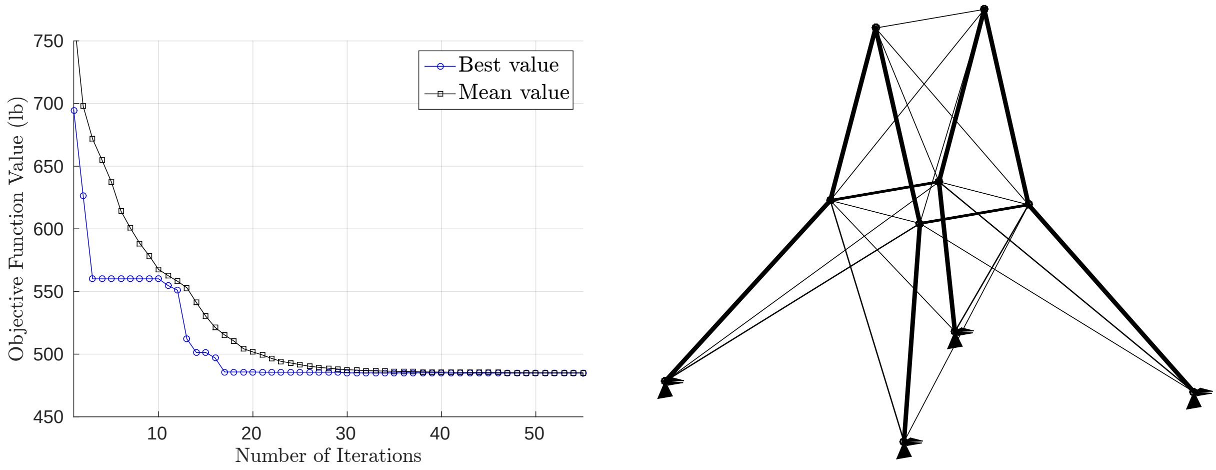
Element group number							
1	2	3	4	5	6	7	8
1:(1,2)	2:(1,4)	6:(2,4)	10:(6,3)	12:(3,4)	14:(3,10)	18:(4,7)	22:(8,6)
	3:(2,3)	7:(2,5)	11:(5,4)	13:(6,5)	15:(6,7)	19:(3,8)	23:(3,7)
	4:(1,5)	8:(1,3)			16:(4,9)	20:(5,10)	24:(4,8)
	5:(2,6)	9:(1,6)			17:(5,8)	21:(6,9)	25:(5,9)

Table 7. Design group information for the 25-bar space truss.

Variable		Cross-sectional areas (in^2)			
Group	Members	Rajeev [19]	Cao [20]	Camp [11]	Present
1	1	0.10	0.10	0.10	0.10
2	2, 3, 4, 5	1.80	0.50	0.30	0.30
3	6, 7, 8, 9	2.30	3.40	3.40	3.40
4	10, 11	0.20	0.10	0.10	0.10
5	12, 13	0.10	1.90	2.10	2.10
6	14, 15, 16, 17	0.80	0.90	1.00	1.00
7	18, 19, 20, 21	1.80	0.50	0.50	0.50
8	22, 23, 24, 25	3.00	3.40	3.40	3.40
	Weight (lb)	546.01	485.05	484.85	484.85

Table 8. Truss optimal design comparison for the 25-bar space truss.

The stopping criterion adapted, as well as in the first example, was a total of 2000 analyses without overall change of the best feasible design ($\epsilon_d \leq 10^{-6}$). Also, twenty independetly runs were achieved in order to test the performance of the optimization process. The minimum optimal design obtained from the process was 484.85 lb, while the mean value of the optimal design was 484.96 lb. A typical convergence history diagram is shown in Figure 6 (a). The final optimal design is shown in Figure 6 (b), where the thicker line represents the element with the greater area (3.4 in^2). Furthermore, Table 8 compares the best design obtained from the present study to others obtained in the literature.



(a) (b)

Figure 6. (a) Typical behaviour of the convergence optimization history; (b) final truss geometry.

3.3 Real Tower Structure

A real tower structure is herewith considered for the validation of the proposed methodology. The tower was first proposed by Pappalardo and Agnelo [21], and then used by Borges et al. [22] for the validation of a FEM software. In the present example, the same geometry and boundary conditions are used for the structural optimization of their components in terms of overall weight reduction.

Figure 7 shows the geometry and node numbering, while Table 10 shows the node coordinates of the problem.

Node	F_x (kN)	F_y (kN)	F_z (kN)	Node	F_x (kN)	F_y (kN)	F_z (kN)
1	0.0000	0.0000	1.3737	10	2.7769	1.6124	3.9166
2	0.0000	0.0000	1.3737	11	2.7769	1.6124	3.9166
3	0.0000	0.0000	1.3737	12	2.2394	1.2541	3.9166
4	1.7930	1.0411	3.9166	13	3.5569	2.0653	3.9166
5	1.7930	1.0411	3.9166	14	3.5569	2.0653	3.9166
6	1.4460	0.8097	3.9166	15	2.8685	1.6064	3.9166
7	1.9969	1.1595	3.9166	16	2.1716	1.2609	2.5429
8	1.9969	1.1595	3.9166	17	2.1716	1.2609	2.5429
9	1.6104	0.9018	3.9166	18	1.7513	0.9807	2.5428

Table 9. Nodal loads for the real tower structure example

The dynamic wind effect was also considered attending the brazilian standard NBR 6123 [5]. The weight is also considered in the present analysis. The resulting nodal loads are shown in Table 9.

Node	x (m)	y (m)	z (m)
1	4.500	0.000	0.000
2	-2.250	3.897	0.000
3	-2.250	-3.897	0.000
4	4.500	0.000	7.500
5	-2.250	3.897	7.500
6	-2.250	-3.897	7.500
7	4.500	0.000	15.000
8	-2.250	3.897	15.000
9	-2.250	-3.897	15.000
10	4.500	0.000	22.500
11	-2.250	3.897	22.500
12	-2.250	-3.897	22.500
13	4.500	0.000	30.000
14	-2.250	3.897	30.000
15	-2.250	-3.897	30.000
16	4.500	0.000	37.500
17	-2.250	3.897	37.500
18	-2.250	-3.897	37.500

Table 10. Coordinates for the real tower example.

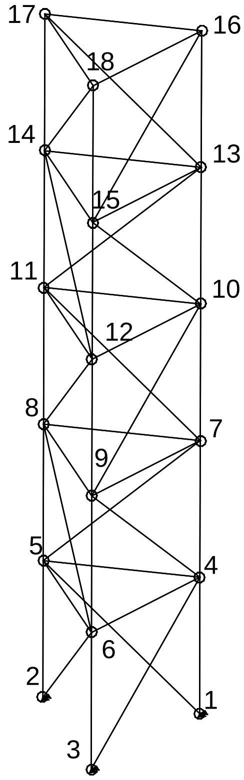


Figure 7. Geometry of the real tower example.

The tower structure is distributed in fifteen design groups. As in the last example, all the members of each group have the same material and cross-sectional properties. Table 12 shows the elements of each group, as well as the nodal conectivity of those elements. The material considered is steel with a density of 7850 kg/m^3 and an elasticity modulus of 200 GPa.

The design problem consider discrete areas for its optimization. Thus, commercial profiles are used for this purpose. Table 11 shows all the options considered on the analysis.

Commercial Name	Weight kg/m	Area cm^2	Commercial Name	Weight kg/m	Area cm^2
L 1x1/8	1.19	1.48	L 1.3/4x3/16	3.15	4.00
L 1x3/16	1.73	2.19	L 1.3/4x1/4	4.12	5.22
L 1x1/4	2.22	2.84	L 2x1/4	4.74	6.06
L 1.1/4x1/8	1.50	1.93	L 2x5/16	5.83	7.42
L 1.1/4x3/16	2.20	2.77	L 2x3/8	6.99	8.76
L 1.1/4x1/4	2.86	3.62	L 2.1/2x3/16	4.57	5.80
L 1.1/2x1/8	1.83	2.32	L 2.1/2x1/4	6.10	7.67
L 1.1/2x3/16	2.68	3.42	L 2.1/2x5/16	7.44	9.48
L 1.1/2x1/4	3.48	4.45	L 3x3/16	5.52	7.03
L 1.3/4x1/8	2.14	2.71	L 3x1/4	7.29	9.29

Table 11. Commercial profiles considered on the tower structure example.

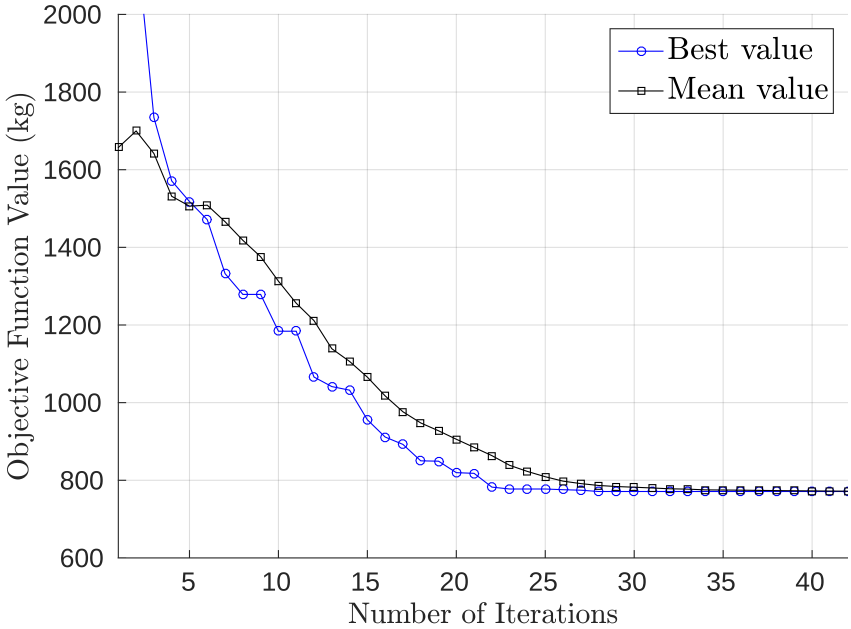


Figure 8. Typical behaviour of the convergence optimization history.

Variable		Profile
Group	Members	
1	3:(4,5), 6:(5,6), 9:(4,6)	L 1x1/8
2	12:(7,8), 15:(8,9), 18:(7,9)	L 1x1/8
3	21:(10,11), 24:(11,12), 27:(10,12)	L 1x1/8
4	30:(13,14), 33:(14,15), 36:(13,15)	L 1x1/8
5	39:(16,17), 42:(17,18), 45:(16,18)	L 1x1/8
6	4:(2,5), 1:(1,4), 7:(3,6)	L 2x3/8
7	5:(2,6), 2:(1,5), 8:(3,4)	L 1.3/4x1/8
8	13:(5,8), 10:(4,7), 16:(6,9)	L 2x1/4
9	17:(6,8), 11:(4,9), 14:(5,7)	L 1x3/16
10	22:(8,11), 25:(9,12), 19:(7,10)	L 1.1/4x1/4
11	23:(8,12), 20:(7,11), 26:(9,10)	L 1.1/4x1/8
12	31:(11,14), 34:(12,15), 28:(10,13)	L 1.1/4x1/8
13	35:(12,14), 29:(10,15), 32:(11,13)	L 1x1/8
14	40:(14,17), 43:(15,18), 37:(13,16)	L 1x1/8
15	41:(14,18), 38:(13,17), 44:(15,16)	L 1x1/8

Table 12. Final areas for the tower example.

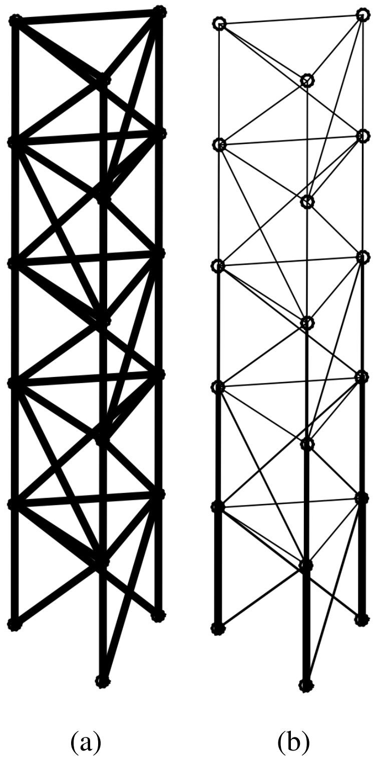


Figure 9. Final geometries of structures attending design restrictions: (a) Components with same area, (b) Components with different areas.

The maximum allowed stress is 150 MPa, while the maximum displacement is considered H/300 (0.125 m), as recommended by the brazilian standard NBR 8800 for this type of structure. As in previous examples, the stopping criterion considered was a total of 2000 analysis without overall change of the best feasible design. The minimum optimal design obtained from the process was 771.135 kg. A typical convergence history diagram is shown in Figure 8. Final commercial areas obtained from the optimization process are shown in Table 12. For comparison purposes it was considered the case were all components share the same area, while attending the restrictions. The optimal structure for this case has a weight of 2352 kg and is shown in Figure 9 (a), while the optimal structure with different areas is shown in Figure 9 (b). The structure obtained on the present work represents only 32.8% of the total weight from a optimization considering the same area for its structural components.

4 Conclusions

The present work proposes a methodology for the optimization of 3D trusses considering the dynamic wind effect. The TLBO method demonstrated to be accurate and versatile. Thus, it showed to be straightforward to implement. Results were compatible with those obtained in the literature, presenting better results in almost all situations. It is worth to point out that all designs presented were feasible. For the example of the tower optimization design, a reduction of 67.2% was achieved. This fact shows that substantial gain in terms of weight and, therefore, in terms of economy can be obtained by applying the methodology herewith presented. The dynamic effect was accounted by acting in accordance with the brazilian standard NBR 6123 [5]. However, the formulation is a simplification, hereby, better and more accurate results are expected if the real dynamic effect is considered. This topic should be presented in future works.

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