AKADÉMIAI KIADÓ

Pollack Periodica • An International Journal for Engineering and Information Sciences

17 (2022) 2, 109–114

DOI: 10.1556/606.2022.00545

ORIGINAL RESEARCH PAPER

Comparison of optimized steel frame structures

Máté Petrik* and Károly Jármai

Institute of Energy Engineering and Chemical Machinery, Faculty of Mechanical Engineering and Informatics, University of Miskolc, H-3515, Miskolc, Hungary

Received: December 24, 2021 • Revised manuscript received: February 8, 2022 • Accepted: February 12, 2022
Published online: April 28, 2022

ABSTRACT

Frame structures are defined as structures built of straight, less often curved bars, which are dimensioned to carry a planar or spatial load. These frames are generally considered statically indeterminate structures so that several methods can be used to determine their loads, but all of them require some simplification. This paper is not concerned with investigating these theories for determining the stresses but with the optimum design of a frame structure for a given geometry. Several different loads have been considered, where the value of the wind load in the horizontal direction has been considered. The optimization problem is mathematically formulated so that both compressive forces and bending moments acting on the horizontal beam and the vertical column, and their composite loads, are below the limit set by the material properties. The column connections were assumed to be fully rigid, and welded I-section were considered for both columns. For local bending conditions, the Eurocode 3 specification was applied. Several steel grades were tested during the investigations, and fire loading was considered an additional load. In this case, a higher safety factor was assumed to make the times to collapse comparable.

KEYWORDS

steel structures, frames, optimization, fire load

1. INTRODUCTION

Steel structures are an essential part of any factory or plant; they are practically its skeleton. During the investment, at the beginning of the construction, the installation of this structure is very resource-intensive. In practice, an optimization process is already underway in the plant's design; there would be neither a theoretical limit to building these plants without a significant steel structure, using horizontal construction, nor building production in very tall buildings. The disadvantages of the horizontal construction method are that it would require a huge area, which would reduce the size of the agricultural land, and that the flow of media would not be gravity fed, so that many more pumps would be needed, which would increase the energy consumption of the plant excessively (which would be an ongoing expense). On the other hand, very tall frame structures would require excessively large section cross-sections, both because of the weight of the structure itself and because of the mass of the equipment it supports. In neither of these extreme cases is the intervening time of the operating personnel negligible, which is not ideal in either case [1, 2].

The present paper deals with the optimal design of frame structures, considering wind loads in addition to natural loads and loads due to the self-weight of the horizontal beam.

2. MATERIALS AND METHODS

2.1. Boundary conditions and variables

The main goal of this paper is to prove that the optimization process is feasible and appropriate for the calculation of the framework structures. A symmetrical, rectangular frame

*Corresponding author.
E-mail: mate.petrik@uni-miskolc.hu

AKJournals

has been considered for the comparisons, with a constant column length and height. The schematic drawing of this frame is described in Fig. 1.

As it clearly can be seen from the figure, the vertical columns on opposite sides are the same (symbol 1 in Fig. 1), and the horizontal beam has another dimension (symbol 2 in Fig. 1). Both are welded I-beams, so the calculation process is also the same. Due to the high variation possibilities, the height of the vertical columns, the length of the horizontal beam, the used steel grade and the type of the sections were the same in every case; the variables were the magnitude of the load and the type of the load. The following will explain why these parameters have been taken as a constant.

The present paper deals with S355JR steel grade. According to the Authors' previous paper [3], this steel grade provides the smallest cross-section areas for I-sections. This is the explanation, why the I-section was chosen for the comparison. However, the types of I-section can be distinguished, which are the rolled and welded sections. The dimensions of the rolled section are standardized in DIN 1025 standard [4], so usage of these types, the optimization process simplifies into a selection procedure. The optimization will be relevant for welded sections in terms of determining the dimensions shown in Fig. 2, where h is the height of the web; t_w is the thickness of the web; b is the width of the flanges; and t_f is the thickness of the flanges.

With these boundary conditions, the aim of the optimization process is to determine the geometric dimensions just described for both the beam and columns while using as few amount of steels as possible. The objective function was the amount of steel; however, as the column lengths and density are constant, this mass is directly proportional to the cross-section areas [5].

2.2. Model for calculating overall planar buckling

The checking verification of the overall buckling should start with determining the inertial moments [6]. The values shown below must be determined for both beams because it is unknown, which axis the collapse will occur [7]. Critical force, which causes overall buckling:

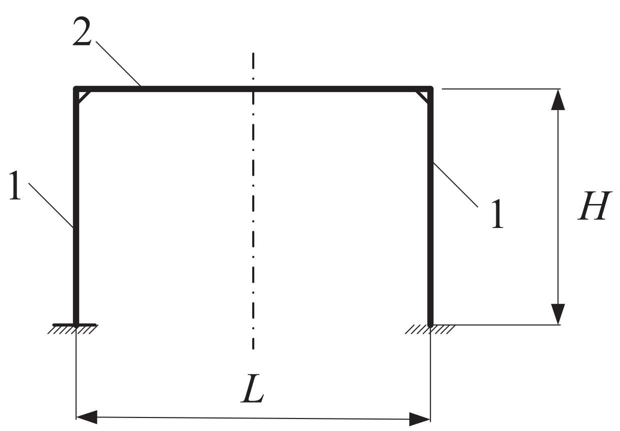

Fig. 1. Schematic drawing of the investigated frame

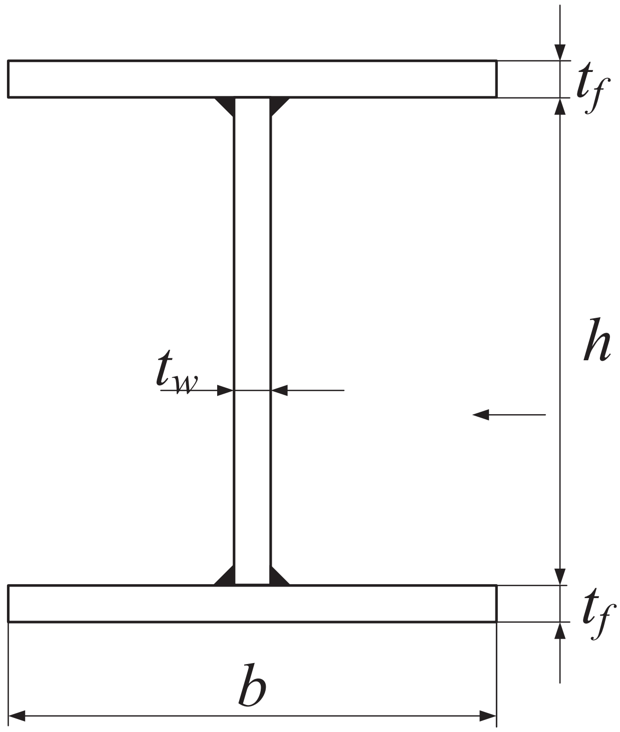

Fig. 2. Schematic drawing of the I-section

$$F_E = \frac{\pi^2 \cdot E \cdot I}{(K \cdot l)^2}, \quad (1)$$

where I is the moment of inertia in the investigated axis (with m^4 dimension); E is the Young modulus of the structural material at the design temperature; K is a constant, which depends on the method of the connections [8], and l is the length of the column.

Knowing the moment of inertia, the following correlation can determine the value of the radius of gyration:

$$r = \sqrt{\frac{I}{A}}. \quad (2)$$

The slenderness of the investigated beam from the projected length and the radius of gyration is

$$\lambda = \frac{K \cdot l}{r}, \quad (3)$$

while the equivalent slenderness depends on the material properties of the structural material:

$$\lambda_E = \pi \cdot \sqrt{\frac{E}{f_y}}, \quad (4)$$

where f_y is the yield stress. From the ratio of the slenderness defined below, a $\bar{\lambda}$ proportional factor must be determined,

$$\bar{\lambda} = \frac{\lambda}{\lambda_E}. \quad (5)$$

The effects of the nonconformities are calculated using Eurocode III (EC3) [9] correlations,

$$\eta_b = \alpha \cdot (\bar{\lambda} - 0.2). \tag{6}$$

The value of the factor α is 0.34 for cold-formed hollow sections, welded box sections and I-sections

$$\varphi = 0.5 \cdot \left(1 + \eta_b + \bar{\lambda}^2\right). \tag{7}$$

The proportional factor is

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \bar{\lambda}^2}}. \tag{8}$$

2.3. Model for calculating bending

To calculate the effect of bending moment, it is essential to calculate the overall buckling, even if there is no compressive force on the beam,

$$k = \min\left\{1 - \frac{\mu \cdot N}{\chi \cdot A \cdot f_y}; 1.5\right\}. \tag{9}$$

The value of k can be a maximum of 1.5, while the value of μ is a maximum of 0.90.

$$\mu = \min\{\bar{\lambda} \cdot (2 \cdot \beta_M - 4);\ 0.9\}. \tag{10}$$

In the correlation, β_M is the equivalent uniform moment factor, which value in the case of a simply supported beam is 1.4. It is seen from the equations that the ratio of the slenderness has a high impact on the results.

2.4. Compound load

It can be seen from the above that the compressive force causes a failure that is practically independent of the allowable stress, only a function of the slenderness. The following relationship should be applied to take these two different effects into account,

$$\frac{N}{\chi_{\min} \cdot A \cdot f_{y1}} + \frac{k \cdot M}{W \cdot f_y} \leq 1. \tag{11}$$

The calculations assume that the bending moment acts only in the direction of the axis of higher inertia in all cases. The connection point between the horizontal beam and the vertical column can be considered rigid.

2.5. Frame loads

The individual loads outlined in the previous chapters can be determined by calculating the assembled frame structures. Two loads were considered twice for the comparisons.

The geometry of the connecting elements has a significant influence on the forces and moments at the nodes,

$$k^* = \frac{I_2}{I_1} \cdot \frac{H}{L}. \tag{12}$$

According to Glushkov [10], these forces and moments are the functions of this ratio. Two different options for vertical loading are considered, which are described in Fig. 3.

In Fig. 3 part a) shows the concentrated force and part b) shows the uniformly distributed load. With a concentrated

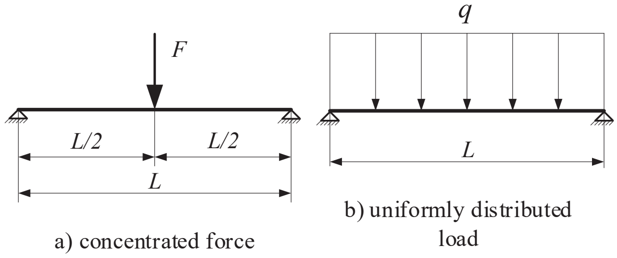

Fig. 3. Schematic of the investigated vertical loads

force, equipment with a higher mass should be considered, the weight of the beam or weight of pallets for uniformly distributed loads. It should be noted here that the mass of the horizontal beams is considered as distributed load in all calculations.

In the case of the second part, there is a horizontal load also, which comes from the wind load.

A frame structure can be formed by placing several assumed frames one after the other and the wind load is defined as the force on the resulting sidewall. An agricultural building in Hungary was taken as a basis. An averaged wind pressure of 0.911 kN m^{-2} was assumed for an area with low vegetation and isolated obstacles so that a uniformly distributed load in the horizontal direction was calculated.

As mentioned above, only the values and types of loads have been changed, not the dimensions of the investigated structure, to allow comparison. The direction of the loads with the main geometric parameters of the structure is shown in Fig. 4.

3. RESULTS IN A STATIONARY CONDITION

3.1. Optimization process

The generalized reduced gradient method was used [11], and the implementation was done with the MS-Excel Solver

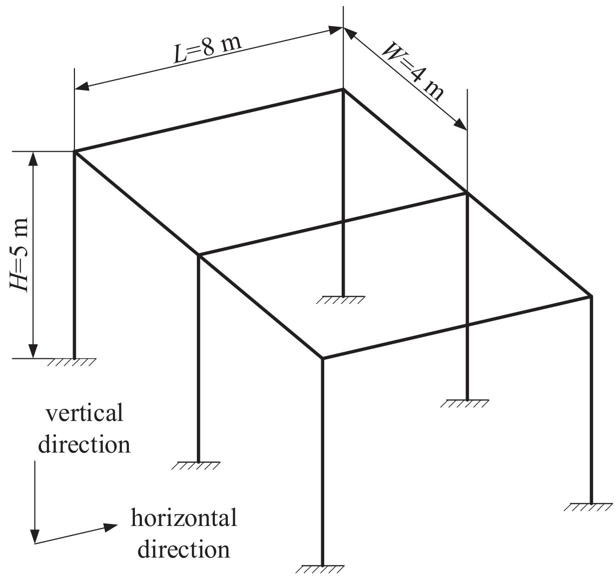

Fig. 4. The main geometric parameter of the frame structure

extension. As it is written in the previous section, the objective function was the total volume of the steel.

This present study aimed to determine the dimensions of both vertical and horizontal beams. Since these beams are supposed to be welding beams, there were 8 dimensions: the height of the flange (h); the thickness of the flange (t_w); the width of belts (b) and thickness of belts (t_f) for both beams (which are described in Fig. 2). The objective function, which is the total mass of the structure, is directly proportional to these variables.

$$m = 2 \cdot H \cdot \left(2 \cdot b \cdot t_f + h \cdot t_w\right)_{vert} + L \cdot \left(2 \cdot b \cdot t_f + h \cdot t_w\right)_{hor} \rightarrow \text{minimized.} \quad (13)$$

Since the height of the frame structure (H) and span length (L) are constants, the optimized value depends on the cross-sectional areas.

During the optimization, the third group is the condition functions. These conditions fall into two categories. The first category includes simple geometric constraints, for example the thicknesses of the flange and belts must be higher than 5 mm, or the width of the belt must be equal to or less than the height of the flange. In the second category, there are the operating parameters, which ensure safety use. One of these conditions is the compound load described in Point 2.4. This is the condition, which will cause the algorithm to increase the values of the variables. However, other conditions should be used to avoid local buckling. According to EC3 [9], this shall be checked by the ratio of the height and the thickness of the plates forming the I-section.

Table 1 summarizes the optimizations carried out. The different cases will be referred to by their identifier in Table 1.

3.2. Optimization results

This subsection contains the results of the optimizations. The graphs are constructed in the same way: the cross-section areas of the horizontal beam and the vertical column on the left side y-axis, and the total mass of the frame on the right y-axis.

Figure 5 shows the results of Case 1.

The comparisons were made to ensure that the loads were of the same magnitude. This means that for the calculations with a concentrated load of 100,000 N, this force has been distributed over the 8 m length, giving a distributed value of 12,500 N m^{-1}. It can be seen from Fig. 6 that the optimized results are about 30% lower than the results obtained with concentrated force. This was an expected result, as this load will result in a lower moment on both the horizontal beam and the corner junctions as well.

Table 1. Loads on the different cases

Case ID	Vertical load	Horizontal load	Figure reference
Case 1	concentrated force	-	Figure 5
Case 2	uniformly distributed load	-	Figure 6
Case 3	concentrated force	wind	Figure 7
Case 4	uniformly distributed load	wind	Figure 8

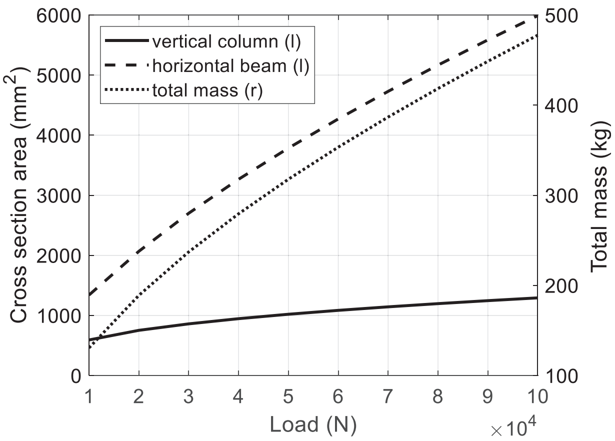

Fig. 5. Cross-section areas and total mass of Case 1

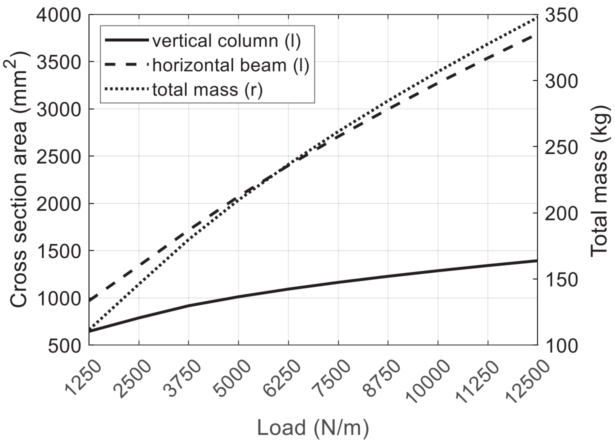

Fig. 6. Cross-section areas and total mass of Case 2

The cases with only vertical loads (Case 1–2) have in common that the cross-sectional areas of the vertical beams do not have changed significantly; they gave roughly the same results.

Under the same wind load, the cross-section area of the vertical column does not change significantly for Case 3–4, but its value is larger than the first two cases due to the increased bending moment.

Figure 7 represents the optimal cross-section areas in the case of concentrated force on the horizontal beam and uniformly distributed load on the vertical column. First, the function between the load and the optimized values is not directly proportional.

According to the compound load on this horizontal column, the increase in the cross-section between the minimum and maximum load will be much smaller. While in the case without wind load (Case 1–2), it was on average 120%, in the case with wind load (Case 3–4), it was less than 10%. According to the rigid junction mode, the bending moments in the corners increase; furthermore, the cross-sectional areas will have higher values than without wind loads (Fig. 8).

Furthermore, it can be shown that the more significant bending moment is acting on the vertical column results in a larger cross-sectional area. While in the case without wind load (Case 1–2), it was on average 120%, in the case with

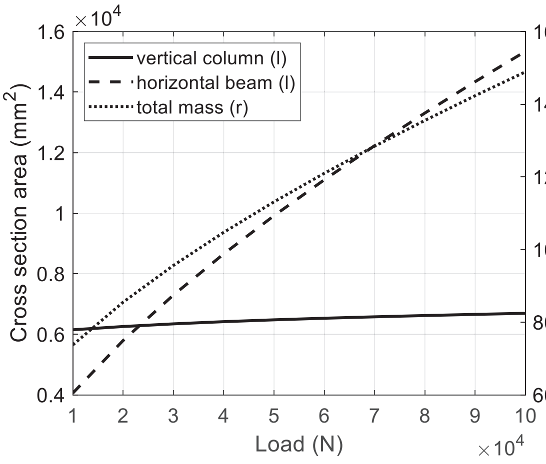

Fig. 7. Cross-section areas and total mass of Case 3

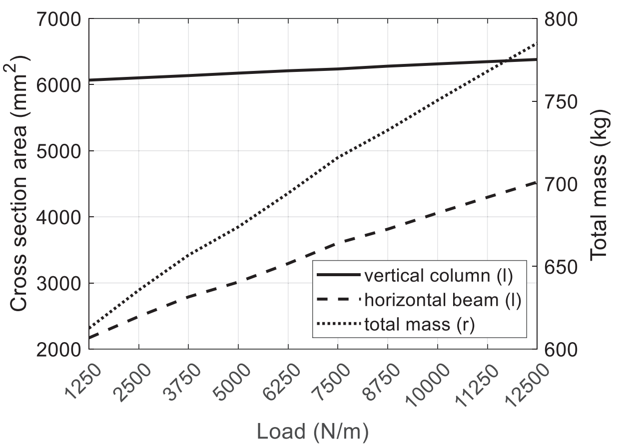

Fig. 8. Cross-section areas and total mass of Case 5

wind load (Case 3–4), it was less than 10%. According to the rigid junction mode, the bending moments in the corners increase; furthermore, the cross-sectional areas will have higher values than without wind loads.

The vertical column cross-section area change is also not significant (5.15%), so the amount of steel to be used is proportional to the horizontal column cross-section area difference.

4. EFFECT OF FIRE LOAD

As it is written in the introduction, these frame structures could be the supporting structure of a production facility, a chemical plant or even an agricultural building. Either way, dust and gas explosions are potential hazards [12]. These explosions do not directly put an extra load on the steel structure, but they are likely to result in potentially hazardous fires [13]. The temperature increase depends on the geometry of the section, the heat transfer conditions and the effects of shielding. The heat radiation and convection conditions during the fire have been described in detail in the Authors' previous paper [14]. As the temperature of the beam increases, the value of the mechanical characteristics (yield strength and Young modulus) decreases, causing the value of exploitation to increase continuously. When it reaches 100%, the global deflection of global buckling occurs.

The fire load calculation was done with the lumped heat capacity model. According to this, the temperature of the cross-section is uniform, the only function of the time. Changes in the material properties of the steel were also considered in the calculations.

Due to the geometry of the I section, the heat transfer takes place over a large surface area, which allows the assumption of a uniform temperature and the application model.

Since the optimization problem was designed for ambient temperature, the time to collapse is relatively short. This effect has been considered in the calculations by varying the value of γ_{M1} material safety factor in the optimization algorithm. The effect of these factors is shown in Figs 9 and 10, both the horizontal and vertical beams.

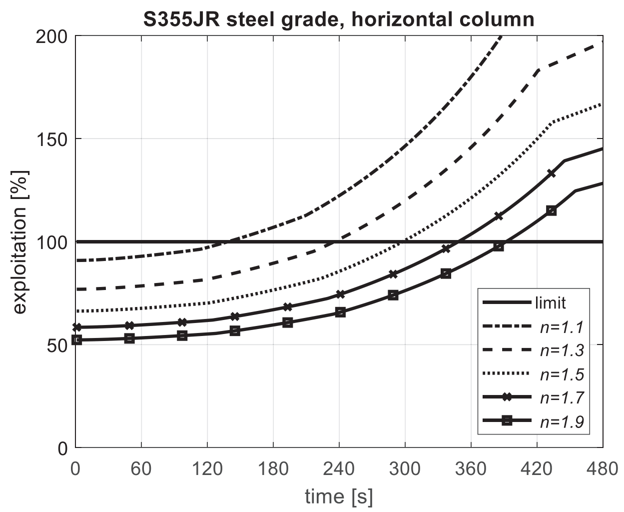

Fig. 9. Effects of the safety factor on horizontal beam in case of fire, for Case 1 load

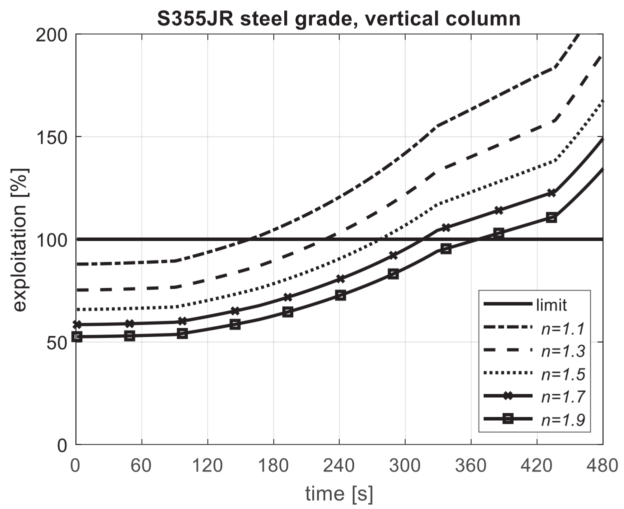

Fig. 10. Effects of the safety factor on vertical beams in case of fire, for Case 1 load

For Case 1 load, the horizontal beams, the possible time in fire at a safety factor of 1.1 is 139 s, while at 1.9 safety factor it is 394 s, which is 2.83 times larger, while the optimum cross-section area is changed from 5989.3 mm^2 to 8654.77 mm^2, which is only 0.44 times larger. This increased time is sufficient for fire-fighting work to begin, which involves extinguishing the fire and cooling the steel structure.

Figure 10 shows the vertical beams for Case 1 load. Also, in this case, the higher safety factor guarantees a longer time (for vertical beams, a 20% larger cross-section area causes 2.36 times longer time). The investigations were carried out for each load case, and the results show similar ratios between the times and cross-section areas.

It must be noted that in all cases, the calculations were made using the optimization results. During construction, the beams would be constructed from standardized sizes since all dimensions will be larger, so that the exploitation will start from a lower value for static load, and a longer time would be obtained for fire loads.

In the case of fire loading, the temperature increment rate could be decreasing if the beams are hiding by architectural solutions or applying fire retardant paint on the surfaces of the beams.

5. CONCLUSIONS

The presented paper shows that the optimization of frame structures with subjected to bending moment and compressive load can be made for beam and columns using the generalized reduced gradient optimization technique. For the optimization, the Excel-solver was used. The optimization can be done for every investigated case, gives accurate results, and meets the restrictive conditions in all cases. To make the comparison, only the type and magnitude of the loads acting on the frame were changed, not the geometric dimensions of the frame and the steel grade used. A previous study showed that steels with the yield strength of 355 MPa provide the smallest cross-sections in compressed and bent beams due to the local buckling condition, and therefore this steel grade was chosen. Three types of loads were selected as vertical loads, and the magnitude of this load was varied during the optimization process, but the equivalent load was the same for different loads. The weight of the horizontal beam was considered in every case. The wind load in the horizontal direction has a high impact on the optimized values. Without a wind load, the increments of the cross-section areas of the vertical columns are 118.1% and 115.7%, while the increments of the horizontal beam are 346.9%, and 290.9%, respectively. The same increments with wind load are 8.9% and 5.2% for the vertical columns are 278.1% and 108.7%, respectively.

The results show that the presented optimization method is only and exclusively valid at ambient temperature. In the case of fire load next to the normal operating loads, the time required for collapse is short and can be increased if the safety factor of the yield strength is chosen to be higher.

ACKNOWLEDGEMENTS

The research was supported by the Hungarian National Research, Development and Innovation Office under the project number K 134358.

REFERENCES

[1] J. Farkas and K. Jármai, *Optimum Design of Steel Structures*. Heidelberg: Springer Verlag, 2013.

[2] M. Ismail and H. El-Sokkary, “Seismic retrofit of steel frame structures,” *Pollack Period.*, vol. 15, no. 2, pp. 106–117, 2020.

[3] K. Jármai and M. Petrik, “Optimization and comparison of different standards for compressed welded box columns,” *Pollack Period.*, vol. 15, no. 1, pp. 3–14, 2020.

[4] *DIN 1025-1-1995:* Hot-Rolled I and H Sections, DIN Deutsches Institut für Normung e. V. Berlin, 1995.

[5] S. Nemer and F. Papp, “Influence of imperfections in the buckling resistance of steel beam-columns under fire,” *Pollack Period.*, vol. 16, no. 2, pp. 1–6, 2021.

[6] N. Kallitsis and K. N. Anyfantis, “Buckling strength assessment of composite patch repair used for the rehabilitation of corroded marine plates,” *Appl. Mech.*, vol. 2, no. 3, pp. 482–500, 2021.

[7] S. Zhang, “A review and study on ultimate strength of steel plates and stiffened panels in axial compression,” *Ships Offshore Struct.*, vol. 11, no. 1, pp. 81–91, 2016.

[8] F. Romdhani, The Flaming According to the Eurocode, (in French), 2017, https://doi.org/10.13140/RG.2.2.33370.34247.

[9] *EN 1993-1-2 2005*, Eurocode 3: Design of Steel Structures – Part 1-2: General Rules – Structural Fire Design, The European Union, European Committee for Standardization, Brussels, 2005.

[10] G. S. Glushkov, I. R. Egorov, and V. V. Ermolov, *Formulas for Designing Frames*. Moscow: Mir Publishers, 1975.

[11] L. S. Lasdon, R. L. Fox, and M. W. Ratner, “Nonlinear optimization using the generalized reduced gradient method,” *Oper. Res.*, vol. 8, no. V3, pp. 73–103, 1974.

[12] G. László, F. Hajdu, and R. Kuti, “Experimental study on examining the fire load of a small compartment,” *Pollack Period.*, pp. 1–6, 2021, https://doi.org/10.1556/606.2021.00422.

[13] V. Mikáczó, G. L. Szepesi, and Z. Siménfalvi, “Simulation of propane explosion in closed vessel,” *Ann. Fac. Eng. Hunedoara*, vol. 15, no. 3, pp. 49–54, 2017.

[14] M. Petrik, A. Erdős, K. Jármai, and G. Szepesi, “Optimum design of an air tank for fatigue and fire load,” *Acta Polytech. Hungarica*, vol. 18, no. 3, pp. 163–177, 2021.