

Designing a flexible facade lift and development of optimal shape of the column

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Abstract. The procedure of designing the columns of the flexible facade lift is presented in this paper. Choice of the optimal variant of the lattice column was realized using the AHP method. The ranking procedure was conducted for 4 criteria and 5 alternatives. The optimal alternative includes circular hollow section (CHS) and square hollow section (SHS). The column design was carried out for 6 combinations of loads according to ASCE 7-05. The calculation of the strength and stability of the column was carried out using the EN 1993-3 and the structural analysis software (SAP 2000). A column with greater lateral rigidity has a very adverse effect on local stability. A greater number of connecting points increases global, and reduces local stability of the column. Properly designed columns must have a certain degree of elasticity. The loss of stability in the case of overloading must only be manifested by global buckling due to significantly higher deformation energy. The presented analysis examines the interaction of global and local buckling. Conclusions in the form of stability criteria can be implemented in the design process.

1. Introduction

Modern concepts of high rise construction require the application of integrated facade systems [1]. Facade lifts play an important role in the process of building and maintaining buildings, shipbuilding, energy and heavy machine construction. The world's leading manufacturers of facade lifts are the Swedish company Alimak [2] and German company Geda [3]. The main part of the facade lift is a lattice column. Recommendations for the design of lattice structures of tower cranes are based on the implemented optimization for the triangular, rectangular and trapezoidal shape [4]. Of particular importance are triangular lattice structures whose optimization is considered in [5]. The theoretical basis for the above mentioned research is given in [6]. The mathematical model of optimization is most often based on the method of Lagrange multipliers [7]. The most important results in terms of optimization of lattice columns are achieved taking into account the constraints due to buckling [8]. The mathematical optimization model in the case of buckling is very complex, which makes it difficult for a wider engineering application. This fact has influenced the existence of scarce research into the defining optimization of lattice columns. The design process is a planned activity with limited human, material and time resources. Therefore, the author in this paper proposes and demonstrates software optimization of the lattice column by applying SAP2000 [9]. For the purpose of optimizing the lattice, the AHP method can also be useful [10]. The lattice column calculation is standardized and takes place in several steps according to EN 1993-1-1 [11].



2. Optimal shape of the column

The basic structural units of the facade lift are a column and a movable work platform. The basic task in designing a facade lift requires defining: a) Types of profiles for column; b) The shape of the column; c) The way the platform is run; d) Mechanism for lifting (drive mechanism) and e) Work platform type. These activities during projecting are not mutually independent, which significantly complicates the work on the optimal choice of solution. Profiles that are the subject of an analysis for the selection of supporting elements of the lattice column (verticals, horizontals and diagonals) include: a) Open rolled thin-walled profiles (INP, LPG, 2UNP, TNP, LNP and 2LNP); b) Seamless tubes of circular and welded tubes of square cross-section, as well c) Profile of full circular and square cross-section. Previous analysis is systematized and graphically illustrated in Table 1.

Table 1. Profiles for generating variant solutions.

Number combination	Profile types of lattice		
	Vertical V	Horizontal H	Diagonal D
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			

The total number of variant solutions that can be generated according to the profiles classified in Table 1 corresponds to the product of the number of profiles from columns V, H and D:

$$\text{Number of column variants} = \text{number (V)} \times \text{number (H)} \times \text{number (D)} = 8 \cdot 3 \cdot 6 = 144 \quad (1)$$

This number of combination is quite large for ranking the alternative, so it is necessary to eliminate certain profiles according to some intuitively clear and obvious criteria. The revised profile table for generating variant solutions is given in Table 2.

Table 2. Revised profiles for variants of column.

Number combination	Profile types of lattice		
	Vertical V	Horizontal H	Diagonal D
1			
2			
3			

The total number of variant solutions in the revised case, using (1) is 12. In order to determine whether all 12 alternative solutions will be fully constructive and technologically suitable for the column performance, a generative matrix of combinations should be formed (Table 3).

Table 3. Alternative solution matrix for column construction.

Vertical (V)	Number of combinations for horizontals and diagonals (H × D)					
	1	2	3	4	5	6
1	V1H1K1	V1H1K2	V1H1K3	V1H2K1	V1H2K2	V1H2K3
2	V2H1K1	V2H1K2	V2H1K3	V2H2K1	V2H2K2	V2H2K3

According to the conducted analysis, 7 variants which were not suitable for assembly were rejected, and the final number of consideration options is $12 - 7 = 5$ (Table 4). Therefore, the variants in which we have the coupling of these profiles are marked with the red color in Table 3. The variant solutions that are the subject of further analysis are given in Table 4 and there is no sharp difference between them as in the previous cases, so it is necessary to apply some of the multi-criteria optimization methods.

Table 4. Alternative solution matrix for lattice.

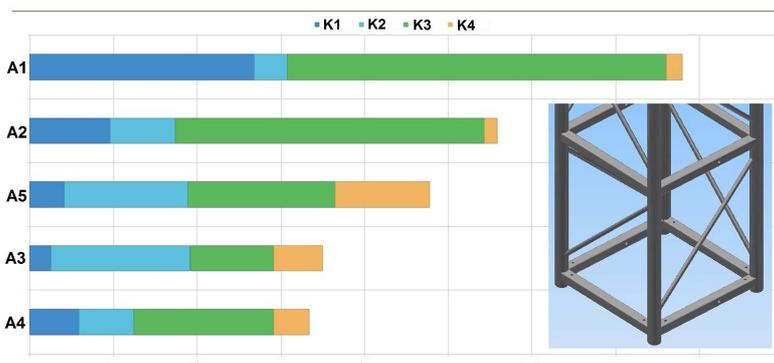
Number of alternatives	Mark of alternatives	Code of combination
1	A1	V1H2K2
2	A2	V1H2K3
3	A3	V2H2K1
4	A4	V2H2K2
5	A5	V2H2K3

2.1. Ranking criteria

The criteria under which the ranking of the profiles for the construction of the columns of the facade lift is carried out include: a) Construction conditions; b) Technological requirements; c) Sensitivity of structure and convenience of guidance as well as d) Production costs.

2.2. Ranking alternatives and choosing the optimal variant

Ranking alternatives and choosing the optimal variant is carried out using the method called: Analytical Hierarchy Process (AHP), implemented in a software solution. After the procedure of mutual subjective ranking of the criteria for individual alternatives, as well as the alternatives itself, an optimal variant for the structural solution of the column is obtained (Figure 1). Recommendations according to SRPS EN 1004:2011 have been taken into consideration [12]. The results from Figure 3 show that the optimal solution for the construction of the column refers to the alternative A1 (V1H2K2). The constructive elements of this alternative are V – CHS; H – SHS and D – CHS. The structure of the facade lift is given in Figure 2

**Figure 1.** Results of the ranking of the variant solutions according to the AHP.

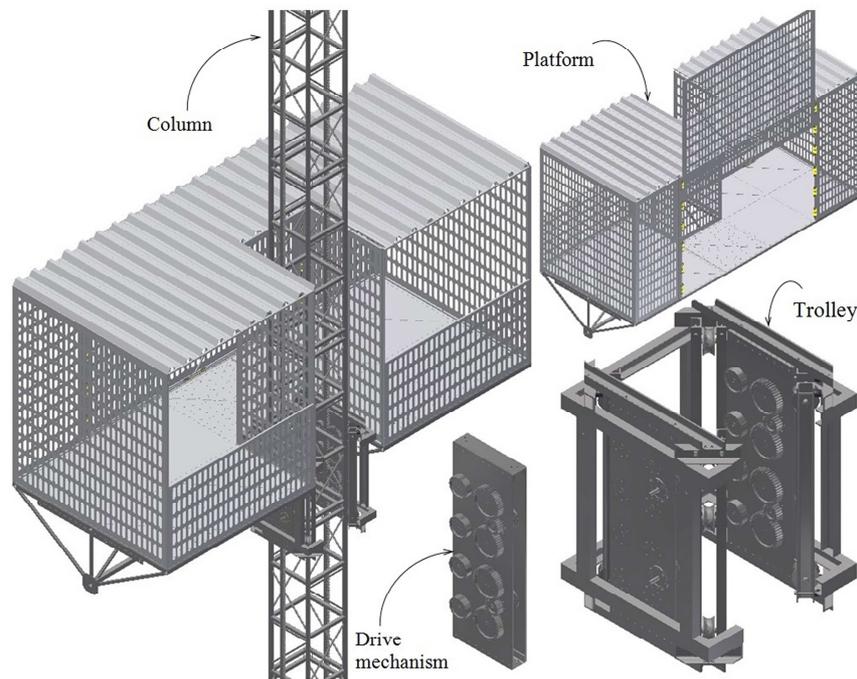


Figure 2. Constructive units of the facade lift and operating principle of the device.

3. Buckling of the column

An approximate calculation of the stability of the lattice column without material axes involves an buckling analysis of a freely supported rod, the length of which corresponds to the distance between the two adjacent supports for lateral fastening. The stability calculation of the column is carried out according to EN 1993-1-1 [11]. The calculation of the load-bearing capacity due to the buckling of the lattice column around the non-material axis includes the following steps (it is enough to carry out a single axle check due to the symmetry of the section): a) Calculation of shear stiffness S_v ; b) Calculation of critical force $N_{cr,v}$; c) Calculation of equivalent relative slenderness and d) Calculation of the buckling capacity with pre-calculated equivalent relative slenderness as for one-piece cross-sections. The shear stiffness of components of laced compression members for the configuration according to Figure 3 in accordance with the recommendations given by EN 1993-1-1 [11] is:

$$S_v = \frac{n \cdot E \cdot A_d \cdot a \cdot h_0^2}{d^3} \Rightarrow S_v = \frac{4 \cdot 21000 \cdot 1,488 \cdot 63 \cdot 50^2}{63,5^3} = 76884 \text{ kN} \quad (2)$$

Where is n – number of planes of lacings, E – modulus of elasticity of steel, A_d – area of one diagonal of a built-up column, a – distance between restraints of chords, h_0 – distance of centrelines of chords of a built-up column, d – length of a diagonal of a built-up column. Critical buckling force is calculated according to:

$$N_{cr,v} = \frac{1}{\frac{1}{N_{cr}} + \frac{1}{S_v}} = \frac{N_{cr}}{1 + \frac{N_{cr}}{S_v}} \Rightarrow N_{cr,v} = \frac{2397}{1 + \frac{2397}{76884}} = 2324 \text{ kN} \quad (3)$$

Where is S_v – shear stiffness of built-up member from the lacings or battened panel, N_{cr} - effective critical force of the built-up member:

$$N_{cr} = \pi^2 \frac{EI_{eff}}{L^2} \Rightarrow N_{cr} = \pi^2 \frac{21000 \cdot 7387}{799,2^2} = 2397 \text{ kN} \quad (4)$$

Where is L – member length ($L = 4 \times 1998 = 7992 \text{ mm}$), I_{eff} – effective second moment of area of the built-up member:

$$I_{eff} = 0.5 \cdot h_0^2 \cdot A_{ch} \Rightarrow I_{eff} = 0.5 \cdot 50^2 \cdot 5,91 = 7387 \text{ cm}^4 \quad (5)$$

Where is A_{ch} – area of one chord of a built-up column ($A_{ch} = 5,91 \text{ cm}^2$). The equivalent relative slenderness of the rod is:

$$\bar{\lambda}_{z,eq} = \frac{\lambda}{\lambda_v} = \sqrt{\frac{N_{pl}}{N_{cr,V}}} \Rightarrow \bar{\lambda}_{z,eq} = \sqrt{\frac{555}{2324}} = 0,49 \quad (6)$$

Where is N_{pl} – plastic bearing capacity of cross section:

$$N_{pl} = A \cdot f_y = (4 \cdot 5,91) \cdot 23,5 = 555 \text{ kN} \quad (7)$$

$N_{cr,V}$ – the critical buckling force the rod around an intangible axis. The appropriate non-dimensional slenderness is:

$$\chi = \begin{cases} 1, & \bar{\lambda} \leq 0,2 \\ \frac{1}{\beta + \sqrt{\beta^2 - \bar{\lambda}_{z,eq}^2}}, & \bar{\lambda} > 0,2 \end{cases} \Rightarrow \chi = \frac{1}{0,65 + \sqrt{0,65^2 - 0,49^2}} = 0,928 \quad (8)$$

Where is

$$\beta = 0,5 \cdot \{1 + \alpha \cdot (\bar{\lambda}_{z,eq} - 0,2) + \bar{\lambda}_{z,eq}^2\} \Rightarrow \beta = 0,5 \cdot \{1 + 0,21 \cdot (0,49 - 0,2) + 0,49^2\} = 0,65 \quad (9)$$

α – imperfection factor ($\alpha = 0,21$ for buckling curve b). The condition for the stability of the column according to EN 1993-1-3 is as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \Rightarrow N_{Ed} \leq N_{b,Rd} = 515 \text{ kN} \quad (10)$$

Where is N_{Ed} – the design value of the compression force, $N_{b,Rd}$ – the design buckling resistance of a compression:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \Rightarrow N_{b,Rd} = \frac{0,928 \cdot (4 \cdot 5,91) \cdot 23,5}{1} = 515 \text{ kN} \quad (11)$$

The value of the compressive force in the preliminary phase of the calculation is the sum of the active load ($F_{uk} = 20 \text{ kN}$) and the part of the passive load ($3G_{uk}/11 \approx 2.2 \text{ kN}$), i.e. we have:

$$N_{Ed} = F_{uk} + G_{uk} = 22,2 \text{ kN} \quad (12)$$

Force in the chords of the lattice column is:

$$N_{ch,Ed} = \frac{N_{Ed}}{4} + \frac{M_{Ed} \cdot h_0}{4 \cdot I_{eff}} \cdot A_{ch} \Rightarrow N_{ch,Ed} = \frac{22,2}{4} + \frac{45 \cdot 50}{4 \cdot 7387} \cdot 5,91 = 7 \text{ kN} \quad (13)$$

Where is M_{Ed} – the moment of bending in the middle of the field of the lattice column:

$$M_{Ed} = \frac{N_{Ed} \cdot e_0}{1 - \frac{N_{Ed}}{N_{cr,V}} - \frac{N_{Ed}}{S_V}} \Rightarrow M_{Ed} = \frac{22,2 \cdot 1,6}{1 - \frac{27,8}{2324} - \frac{27,8}{76884}} = 45 \text{ kNcm} \quad (14)$$

Where is e_0 – maximum amplitude of a member imperfection ($e_0 = L/500 = 7992/500 = 16 \text{ mm}$). The slenderness of a chord is determined by following expression:

$$\lambda_{ch} = \frac{L_{ch}}{i_{min}} \Rightarrow \lambda_{ch} = \frac{555}{16,7} = 33,2 \quad (15)$$

Where is L_{ch} – buckling length of chord ($L_{ch} = 630 - 75 = 555 \text{ mm}$), i_{min} – minimum radius of gyration of single angles. The non-dimensional slenderness of chord from the seamless tube $\text{Ø}51 \times 4$ with material S235JRG2 is:

$$\bar{\lambda}_{ch} = \frac{\lambda_{ch}}{\lambda_y} \Rightarrow \bar{\lambda}_{ch} = \frac{33,2}{93,9} = 0,35 \quad (16)$$

Reduction factor for relevant buckling mode according to Figure 3 is:

$$\chi = \begin{cases} 1 & , \bar{\lambda} \leq 0,2 \\ \frac{1}{\beta + \sqrt{\beta^2 - \bar{\lambda}_{z,eq}^2}} & , \bar{\lambda} > 0,2 \end{cases} \Rightarrow \chi = \frac{1}{0,57 + \sqrt{0,57^2 - 0,35^2}} = 0,98 \quad (17)$$

Where is:

$$\beta = 0,5 \cdot \{1 + \alpha \cdot (\bar{\lambda}_{ch} - 0,2) + \bar{\lambda}_{ch}^2\} \Rightarrow \beta = 0,5 \cdot \{1 + 0,21 \cdot (0,35 - 0,2) + 0,35^2\} = 0,57 \quad (18)$$

The stability condition of the chord as a stand-alone element according to EN 1993-1-1 is as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \Rightarrow \frac{27,8}{136} = 0,2 < 1 \text{ (the stability condition is satisfied)} \quad (19)$$

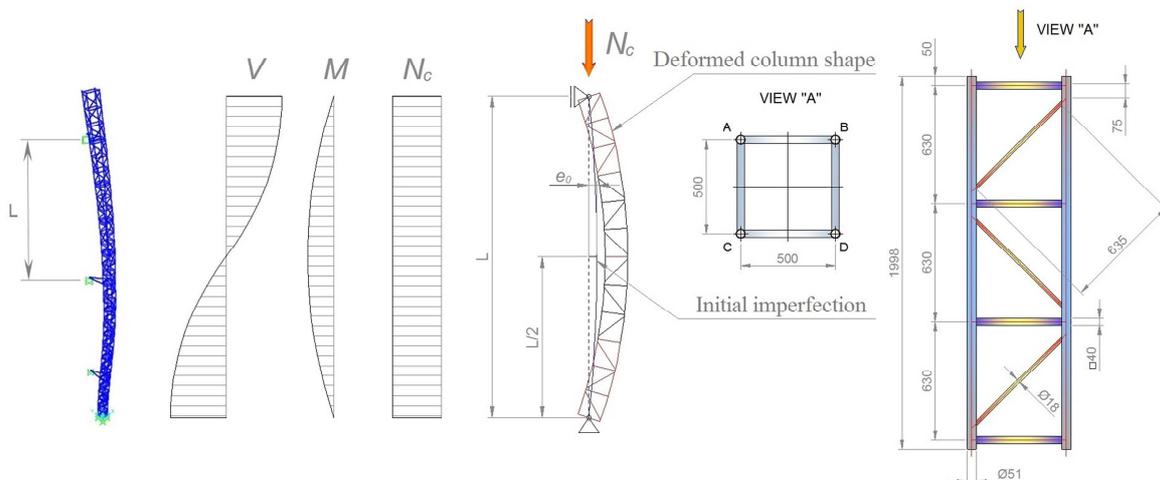


Figure 3. Load of multi-piece cross-section and estimated buckling length.

The design resistance of the compression element for the cross-section class 1 is:

$$N_{b,rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \Rightarrow N_{b,rd} = \frac{0,98 \cdot 5,91 \cdot 23,5}{1} = 136 \text{ kN} \quad (20)$$

4. Software analysis of stability

The implemented calculation of stability according to the concept of limit state is based on the recommendations of EN 1993-1-1 [11]. Software analysis in the SAP2000 V18 provides the ability to identify important parameters related to this issue, referring to the aspect of stiffness of rods for fixing column, the number and layouts of the joining points (Figure 4). Properly designed lattice columns must have a certain degree of elasticity, in order to overcome the loss of stability in the case of its overloading, it is manifested by the global buckling and accumulation of significant elastic potential energy (eg in the case of impact loads, seismic activity, hurricane winds, etc.). Columns that are

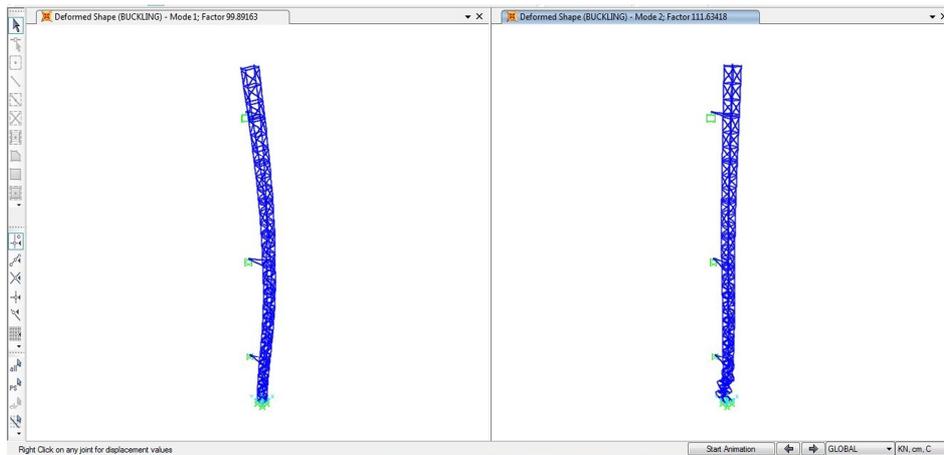


Figure 4. Buckling modes for the lattice column of the device.

designed that the first mode of buckling follow the occurrence of loss of stability of individual elements carry with them the risk that in case of minor overload they can be plastic deformed and damaged. The exception to this analysis are the individual elements of a column with a large wall thickness in relation to the dimensions of the cross-section or full cross-sections.

The critical buckling force calculated according to EN 1993-1-1 amounts $N_{cr,V} = 2324$ kN. The value of the compressive force for which the column is designed amounts $N_{Ed} = 22,2$ kN. Buckling load factor (BLF) is defined as the ratio of forces $N_{cr,V}$ and N_{Ed} :

$$(BLF)_{EN1993-1-1} = \frac{N_{cr,V}}{N_{Ed}} = \frac{2324}{22,2} = 104,6 \tag{21}$$

$(BLF)_{EN 1993-1-1}$ is calculated for the adopted buckling length $L = 4 \times 1998 = 7992$ mm. These data show a good accuracy of the calculation according to EN 1993-1-1 [11] and an adequately estimated buckling length of $L = 7992$ mm.

5. Optimization of the lattice column

Optimization of the bearing structure of the column was carried out according to Eurocode 3 in the software package SAP2000 V18 [9]. The starting calculatory model of the column was with lateral supported in two points. Optimization results are presented in Table 5.

Table 5. Calculatory model and results of software optimization.

Legend	Software model of the column	Number of column segment
	Buckling: mode 1, factor 101.52	1

6. Conclusion

The organization of the execution of works in the high-rise building depends to a large extent on the reliably designed facade lifts. The choice of an optimum cross-section of the lattice column was made using the AHP method. The stability calculation of the column is realized with EN 1993-1-1. Software analysis of stability was performed with the SAP 2000. Comparative analysis of the results shows a well estimated length of buckling according to EN 1993-1-1. A critical mode of buckling the column is defined and recommendations for lateral support of the column are given. The buckling factor for the column of the square cross section 500×500 mm amounts approximately 102. Properly designed lattice column must have greater resistance to local stability versus global buckling. Local loss of stability is accompanied by failure to the entire column. Global buckling is characterized by higher elastic deformation energy, which reduces sensitivity to potential damage to the column. The first mode of buckling the column must always be accompanied by a loss of global stability. These conclusions should be used as the rules during the design of the facade lift.

Acknowledgments

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