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Experimental and Numerical Research on the Behavior of Steel Columns with Circular Hollow Cross Sections

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Abstract

A circular, hollow tubular steel column is introduced for experimental and analytical analysis in this study. A series of axial compression tests for the variation of static schemes are reported in this study. All theoretical, numerical, and experimental analyses are based on the European Standards for the steel structure, respectively EN 1993-1-1. The experimental models of steel columns are conducted on actual steel columns with a length of 3000 mm and a circular hollow section of 114.3/2.8 mm. To assess the behavior and stress values of the columns, various schematically supported systems are modeled, starting from the axial-centered columns to the symmetrical eccentric load and asymmetrical loaded columns. 3D modeling of the steel columns using the finite element program SEISMOSOFT is also developed for such elements. The accuracy of the model is compared with the experimental results using numerical analysis by the finite element method. Finally, the numerical comparison of the results provides a recommendation for the engineers regarding the design and construction of such columns.

Keywords: Steel Columns; European Standards; Circular Hollow Section; Action on Columns.

1. Introduction

Steel structures are increasingly being used in the field of construction engineering due to their excellent properties and homogeneous structure. This places them in the category of homogeneous materials. Steel offers an excellent combination of strength, flexibility, and resistance to tension, making it a preferred choice for various constructions, including complex structures such as high-rise buildings, bridges, and earthquake-resistant structures. This widespread use of steel structures is providing construction engineers with more opportunities to create innovative designs and ensure high performance and safety in their projects.

Currently, the civil engineering industry is witnessing a surge in demand for the construction of tall buildings, especially in developing countries. In such scenarios, the utilization of steel materials for structural systems proves to be highly suitable. Steel materials are renowned for their ability to create stable structures, owing to their high load-bearing capacity and consistent mechanical properties of isotropy. This opens up opportunities for constructing lightweight structures, which are well-suited for seismic zones due to their low mass and excellent ductile properties.

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Taking into consideration the mechanical properties of steel used in construction, it will result in smaller cross-sections of vertical structural members and more attractive architectural shapes for buildings compared to other structural materials [1]. For the constitutive elements of structures working in compression, compression with bending, where as a result of external actions there is appear the phenomenon of buckling, it has been almost talked about early, starting from EULER and then by many authors and researchers in this field of engineering. However, the calculation of where buckling occurs, according to EN 1993-1-1, helps the engineer's structural designers use the "buckling diagrams according to European standards" [2]. Steel structures, like any other engineering structure, must be designed to have sufficient stability and achieve sustainability [3].

Primarily subject to axial compression forces, columns may also undergo bending, torsion, shear, and combinations of them, depending on boundary conditions and applied forces. Designing steel columns involves analyzing local and global stability, taking into account imperfection phenomena. This phenomenon has been meticulously addressed by a group of authors within the scope of the referenced work [4–7]. Currently, there are many studies available on section number [8], but the research on building up steel sections is comparatively limited. Doing the experimental testing of the samples—real cross sections—creates more accurate results compared to the theoretical based on the FME method, which is then applied in software.

Under axial compression loading, especially in cases of eccentricity (moments of bending at edges), the vertical elements, respectively, columns, are exhibiting additional bending. This phenomenon results in an increase in the initial bending moment (second-order theory), known as the P-Δ effect, which needs to be carefully assessed, particularly in the case of slender and flexible columns, where it often proves significant during the calculation phase [9]. In the context of the study, the theoretical treatment of columns has been conducted considering linear analysis, hence employing simplified calculation methods, and nonlinear analysis, utilizing software-based calculations [1, 10].

The objective of this paper is to investigate the bearing capacity values of axial load columns depending on the approved column's static scheme, mechanical material properties, and the selection of the column's cross-sectional type. Therefore, a study in this direction has been conducted as part of the work [11, 12]. Based on recent research conducted by various authors and institutes, there are not many comparisons between experimental results and those provided by calculation methods included in EN 1993-1-1. Hence, this is the main objective of the study.

The significance of the research lies in its contribution to understanding the behavior of axial load columns, particularly in relation to the approved static scheme, mechanical material properties, and the selection of the cross-sectional type. By conducting this study, researchers aim to provide valuable insights into how these factors affect the bearing capacity of columns, which is crucial for ensuring the safety and efficiency of structural designs. Additionally, the findings of the research can help improve design guidelines and standards, leading to more accurate and reliable structural analyses in engineering practice. Therefore, the research has significant implications for the field of structural engineering and contributes to advancing knowledge in this area.

To be more precise, the behavior of steel columns with hollow circular cross-sections under the influence of axial compression force is presented by testing three different static schemes. The experimental values obtained are analyzed and compared with theoretical values according to EN 1993-1-1 and through the SEISMOSOFT application software. The interpretation of the resulting outputs includes maximum force, lateral displacement at the midspan of the columns, axial displacement, as well as the values of deformations and stresses in the steel material.

2. Experimental Program

The experimental program comprises a series of tests conducted on three steel columns with hollow circular cross-sections. These columns are fabricated from identical steel materials and have the same dimensions and length. The objective of the experimental program is to investigate the behavior of these columns under axial compression force.

The experimental setup involves applying axial compression force to each column while measuring various parameters such as load, displacement, and strain. The columns are supported in different static schemes, as illustrated in Figure 1 and Table 1, to simulate various real-world conditions and loading scenarios.

Table 1. Presentation of the geometric characteristics of column samples

Sample No.	Dimension d (mm)	Dimension t (mm)	Length L (mm)	e_A(mm)	e_B(mm)
S-1	113.4	2.8	3000	0	0
S-2	113.4	2.8	3000	30	0
S-3	113.4	2.8	3000	30	30

The flowchart of the research methodology that was used to achieve the study's aims is shown in Figure 2.

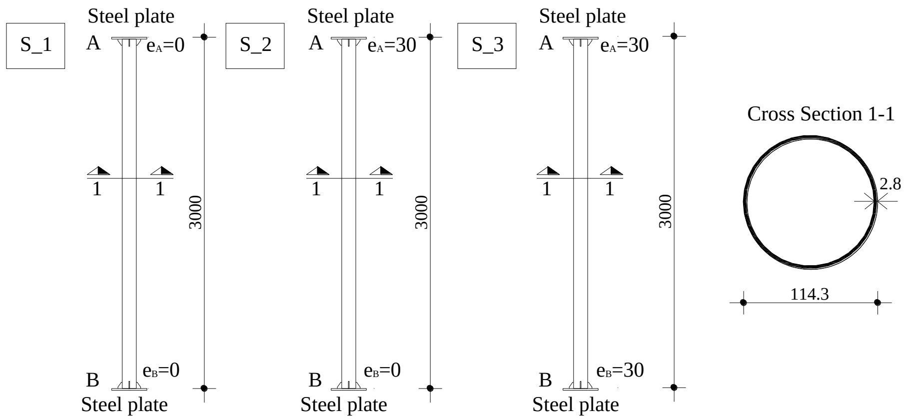

Figure 1. Details of test specimens (units: mm): S_1-centred, S_2 eccentric from above, S_3 eccentric from upper and bottom

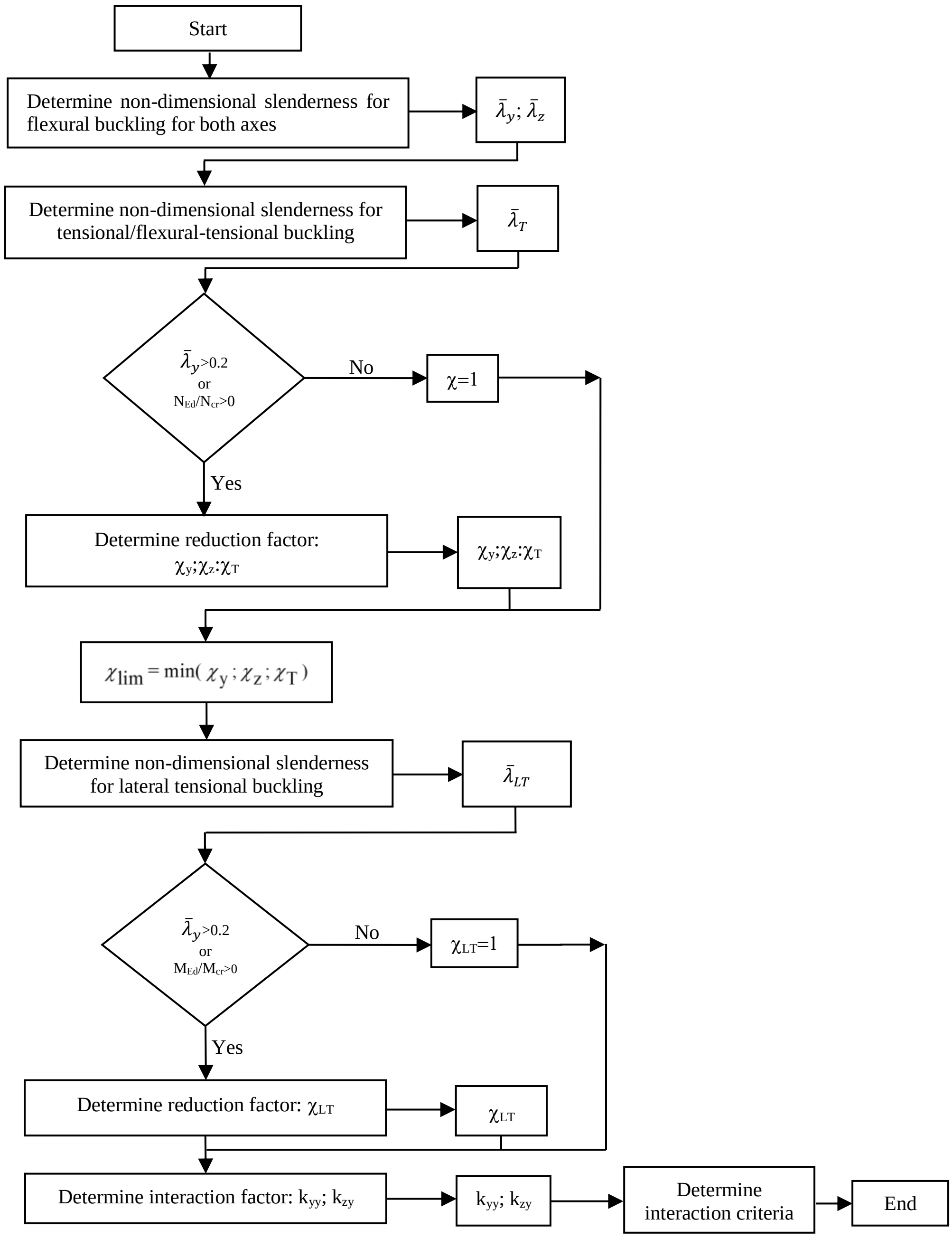

Figure 2. Flow chart for column design

Throughout the experimental program, meticulous attention is paid to controlling variables and ensuring the accuracy and repeatability of measurements. The data collected from the tests are analyzed and compared with theoretical predictions and numerical simulations obtained through EN 1993-1-1 standards and the SEISMOSOFT application software. The experimental program aims to provide valuable insights into the performance of steel columns under axial compression, validate theoretical models, and contribute to the development of design guidelines and standards in structural engineering. For the computation of the theoretical problem, the block diagram is shown, explaining the steps of the analysis in accordance with the standard EN 1993-1-1 [13].

The experimental models consist of tubes formed from flat steel plates with a thickness of 2.8 mm. These plates are shaped into cooled, rotated cylinders and welded longitudinally, as depicted in Figure 3. To ensure stability during testing, steel plates are welded to the top and bottom of each unit, allowing for secure fixation of the unit's end onto the testing equipment frame.

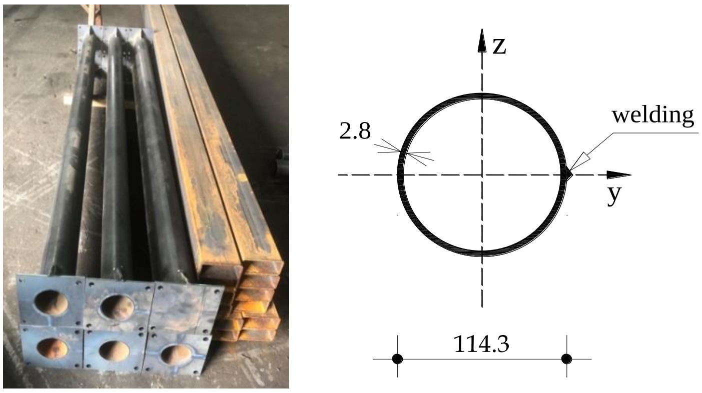

Figure 3. Examples tests units – steel columns, their cross section created in cool base shaping

According to ISO 6892-1:2019 [14] and EN 10002-1:2001 [15] standards, test tubes are prepared for experimental examinations. These test units are extracted from the sample columns, which are steel columns formed into cooled cylinders using CNC cutting, as depicted in Figure 4. The CNC cutting process was conducted by a local company, "Metal Teknika Asllani," located in Fushë Kosova, Kosovo. One of these test tubes is taken from the geometrically welded part of the tube to account for the effects of welding on behavior.

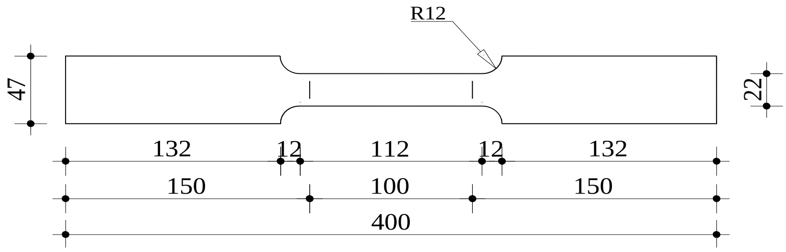

Figure 4. Tests units taken from the sample columns (units: mm)

In Figure 5, the practical attracting units for the mechanical examination properties of the used materials for the sample of columns are presented.

Figure 5. Preparation and cutting of the tests tube

The examination of the test tubes and determination of the steel strength are realized by the accredited laboratory "Proing" located in Pristina, Kosovo, and the final obtained results are shown in Figure 6 and Table 2.

(a) (b) (c)

Figure 6. Examination of the mechanical material properties – thru units taken from the sample column a). before examination, b). during the testing and c). after the final cutting of unit's tests

Table 2. Materials properties exanimated

Test tube	F_m(kN)	A(mm^2)	R_p(N/mm^2)	R_m(kN)	A_g(%)
E_1	25.96	56	344.10	463.60	15.82
E_2	27.05	57.8	335.50	468.00	16.24
E_3*	29.72	58.8	396.10	505.50	10.96

The young modulus value is use based on the literature of E=200,000 N/mm^2.

Notice *- unit taken in area of the welded unit.

For precise examination and reliable results, they were requested from the laboratory of "ProIng", Pristina, in order to calibrate the examination machine-piston, as shown in Figure 7. Calibration of the hydraulic piston was conducted using the "HBM Quantum X" equipment, ensuring the validated force of pressure fell within an accuracy range of +/- 0.50 kN [4].

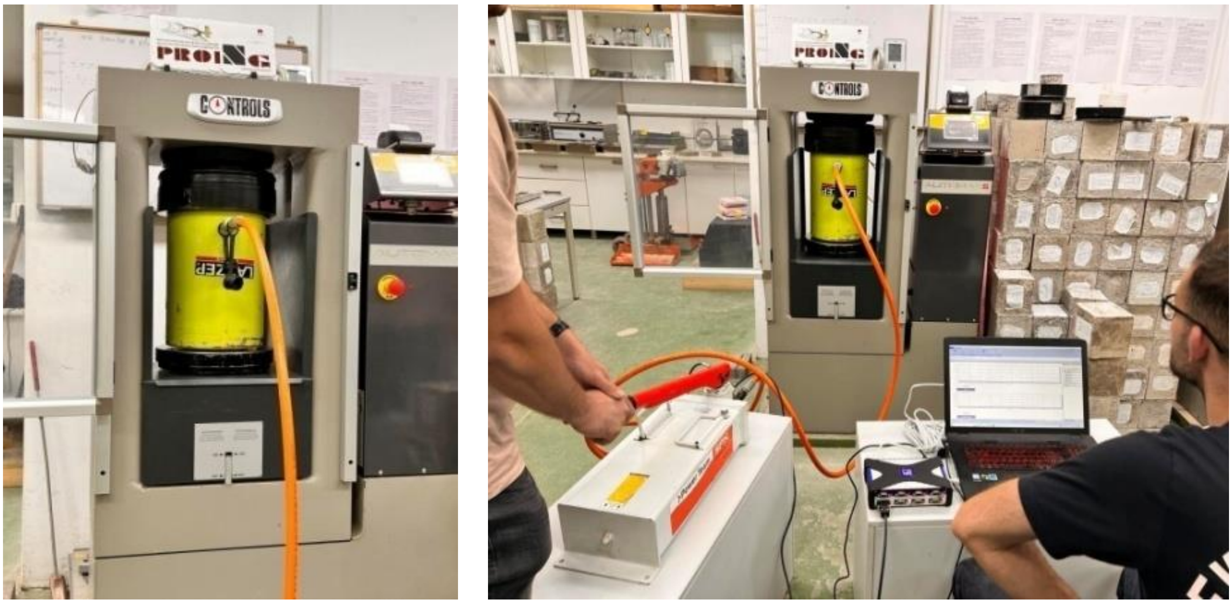

Figure 7. The piston calibration

In the erected steel samples of columns are installed the necessary measuring equipment's to obtain precise and reliable testing results. Equipment "load cell" for the monitoring of the applied load, LVDT for the measurement of the horizontal and vertical displacement of the samples, monitoring the incline deformation through an inclinometer, and four strain gauges are installed for measuring the deformation, as shown in Figure 8 [16–19].

For the determination of the stress and the strain distribution of the steel columns initiated by the axial load, the strain gauges produced by the company "HBM" are erected, as shown in Figure 9.

Figure 8. Examination – testing of the columns, axial and eccentric compression of the units, residual deformation of unit

(a) b) (c)

Figure 9. a) The strain gauges install, b) the local stability loss, c) deformation shape of column

The behavior monitoring of the column, the obtained results from the experimental computation, calibration of the equipment's, testing of the proper technical installation of the fixed unit, direction of the installation of the strain gauges, etc., the applied load on the samples is static and shall be in increments as shown in Figure 10, the incremental diagram of the applied load. All measurements during the examinations are taken digitally via LVDT and Strain Gauge on equipment from "HBM Quantum X" [4, 20–23] and transformed into numerical and graphical data by the computer (Figures 11 to 16).

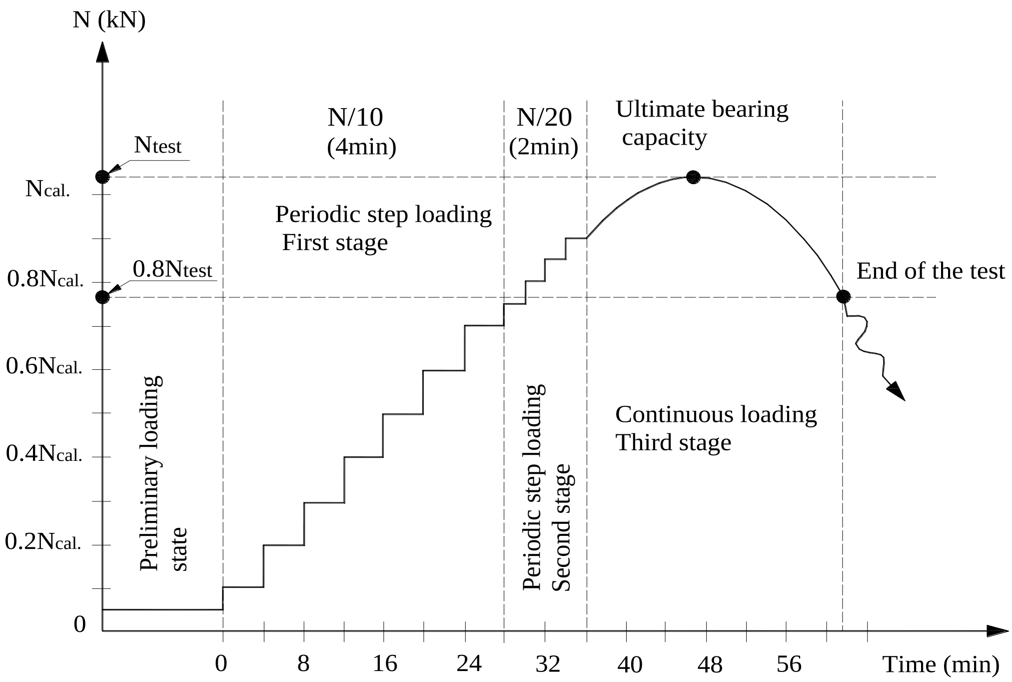

Figure 10. Incremental Load applying diagram

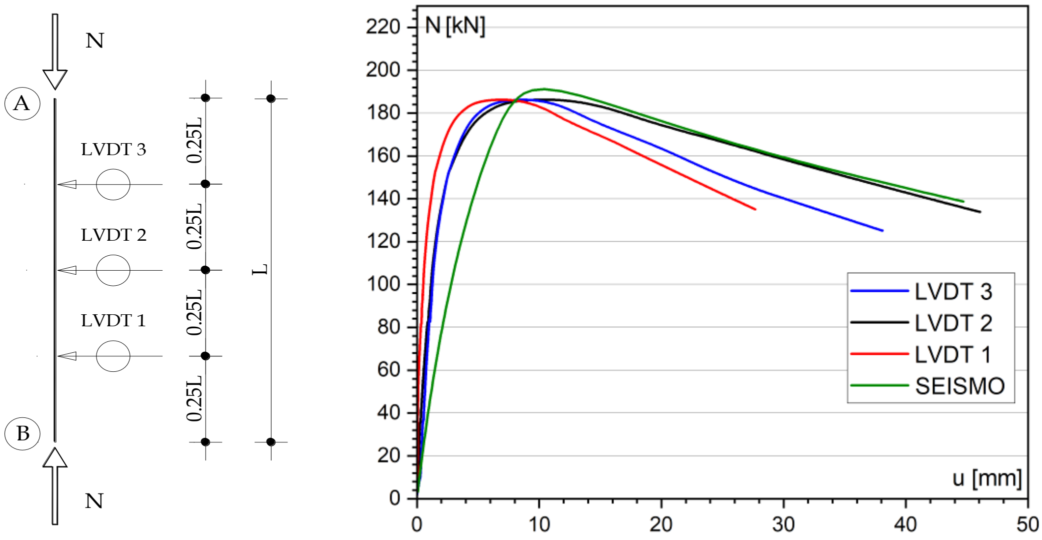

Figure 11.Graphical presentation of the diagram force–lateral displacements

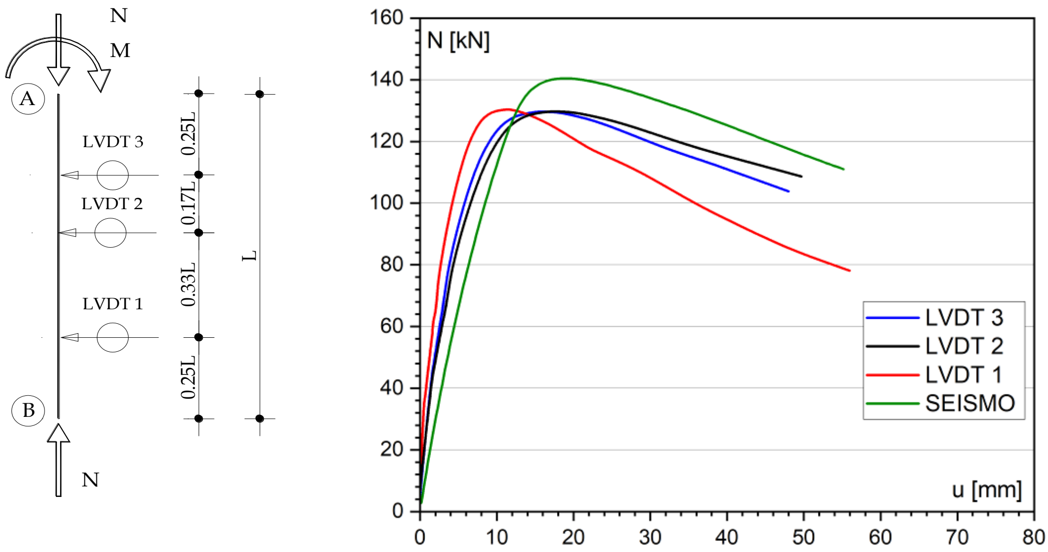

Figure 12. Graphical presentation of the diagram force–lateral displacements

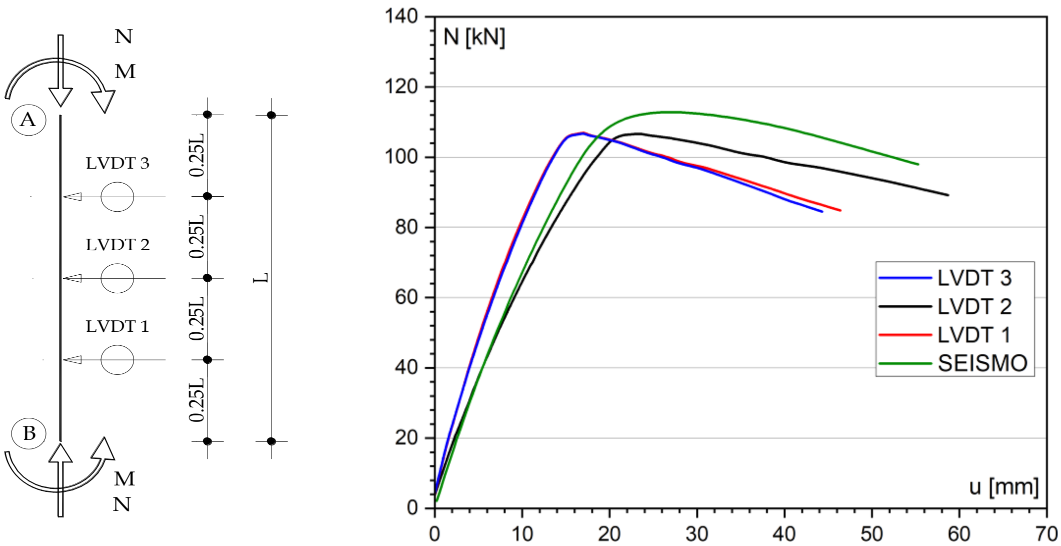

Figure 13. Graphical presentation of the diagram force–lateral displacements

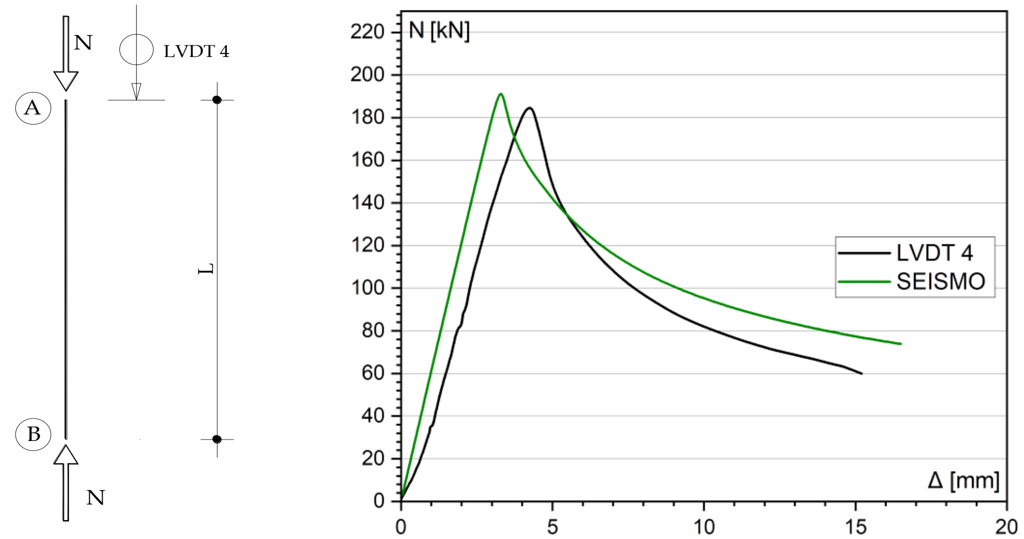

Figure 14. Graphical presentation of the diagram force-axial displacement

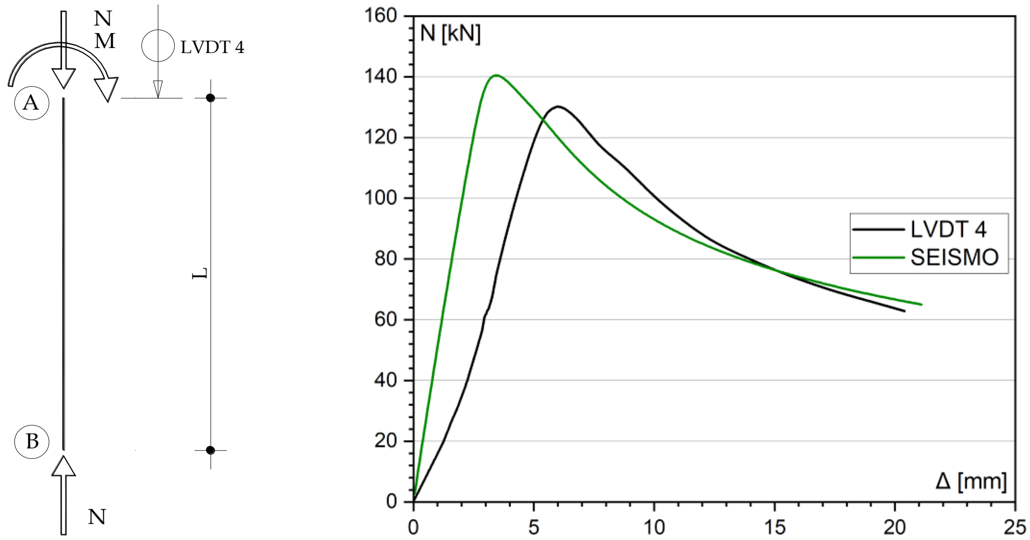

Figure 15. Graphical presentation of the diagram force-axial displacement

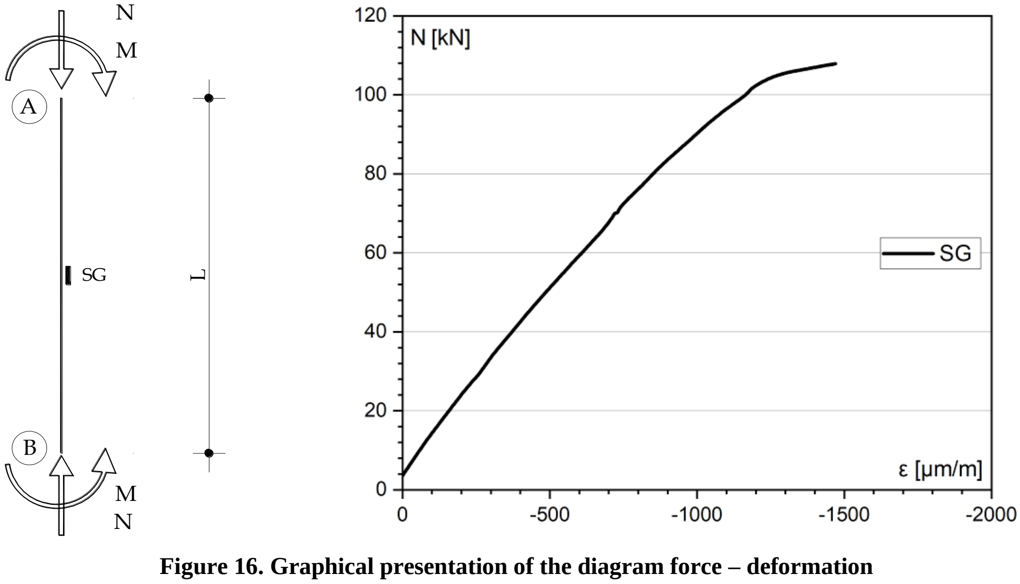

Figure 16. Graphical presentation of the diagram force – deformation

The obtained results are based on the value of the young modulus of steel, E_s=200000 N/mm^2, in compliance with EN 199-1-1 [1, 24]. The stress computation value on the basis of the strain (Hook-e Law) for the cross section of the column in the middle of the spam due to pressure contains:

σ = ε·E_{cm}= 1425·200000·10^{-6} = 285 N/mm^2

3. Theoretical Part

A compression member should verify their bearing capacity and against buckling, comparing the actions on structure to the member resistance as follows:

$$N_{Ed} \leq N_{b,Rd} \tag{1}$$

where N_{Ed} is design value of axial force and $N_{b,Rd}$ is design buckling values of the resistance of the compression members.

The design buckling resistance of a compression members should be taken as follows:

$$N_{b,Rd} = \chi \cdot \beta_A \cdot A \cdot \frac{f_y}{\gamma_M} \tag{2}$$

where A is area of cross section of members, β_A=1,0 for the cross section that belongs to curvature class of 1, 2 and 3, $\beta_A = A_{eff}/A$ for the cross section that belongs to curvature class 4, χ is non dimensional reduction factor of buckling equal to ratio of normal critical stress in bending with limited value of the yield, were depends on the relative slenderness λ, then from shape of the cross section of member, geometrical imperfection, f_y is yield strength of the material, and γ_M is particular partial safety factor for material.

The reduction factor χ shown the relation as below:

$$\chi = \frac{N_{b,Rd}}{N_{pl,Rd}} = \frac{f_{b,Rd}}{f_{y,d}} \tag{3}$$

$$f_{b,Rd} = \frac{N_{b,Rd}}{A} \tag{4}$$

$$f_{yd} = \frac{f_y}{\gamma_M} \tag{5}$$

Non dimensional slenderness $\bar{\lambda}$ should be determined in ration of effective slenderness λ with slenderness in yield limited point λ_E.

$$\bar{\lambda} = \frac{\lambda}{\lambda_E} \tag{6}$$

$$\lambda = \frac{l_o}{i} \frac{effective\ length\ of\ bending}{inertial\ redius\ representing} \tag{7}$$

$$\lambda_E = \pi \sqrt{\frac{E}{f_y}} \tag{8}$$

The reduction factor of the non-dimensional slenderness analytically it may take as follow:

$$\chi = \frac{1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2}{2\bar{\lambda}^2} - \frac{1}{2\bar{\lambda}^2}\sqrt{\left[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2\right]^2 - 4\bar{\lambda}^2} \tag{9}$$

The imperfection factor α corresponding to the appropriate buckling curve should obtained from Table 3 [25]:

Table 3. Imperfection factor for buckling curves

Buckling Curve	a₀	a	b	c	d
Imperfection factor (α)	0.13	0.21	0.34	0.49	0.76

The Equation 9 may reduce also in the straightforward way as follow:

$$\chi = \frac{1}{\Phi+\sqrt{\Phi^2-\bar{\lambda}^2}} \text{ ,where: } \Phi = 0.5\left[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2\right] \tag{10}$$

Based on EN 1993-1-1, where the member which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{Ed,y}+\Delta M_{Ed,y}}{\chi_{LT}\cdot\frac{M_{Rk,y}}{\gamma_{M1}}} + k_{yz}\cdot\frac{M_{Ed,z}+\Delta M_{Ed,z}}{\frac{M_{Rk,z}}{\gamma_{M1}}} \leq 1 \tag{11}$$

$$\frac{N_{Ed}}{\frac{\chi_z \cdot N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{Ed,y}+\Delta M_{Ed,y}}{\chi_{LT}\cdot\frac{M_{Rk,y}}{\gamma_{M1}}} + k_{zz}\cdot\frac{M_{Ed,z}+\Delta M_{Ed,z}}{\frac{M_{Rk,z}}{\gamma_{M1}}} \leq 1 \tag{12}$$

where $M_{Ed,y}$ is design maximum bending moment about the y-y axis, $M_{Ed,z}$ is design maximum bending moment about the z-z axis, $\Delta M_{Ed,y}$; $\Delta M_{Ed,z}$ are the moments due to the shift of the centroidal axis for class 4 sections, χ_y, χ_y are the reduction factors due to flexural buckling, χ_{LT} is the reduction factor due to lateral torsional buckling, and k_{yy}; k_{yz}; k_{zy} *and* k_{zz}- are the interaction factors.

Based on the used literature the capacity resistance of the columns for the axial loads:

$$N_{Rd,sec}=A_{s,neto} \cdot f_{yk} = \frac{\pi}{4}\,[D^2 - d^2] \cdot f_{yk} \tag{13}$$

As per theory of the Structure, the displacement from the eccentricity of the compressed members is expressed with equation:

$$W_{max} = \frac{40 \cdot N \cdot e_{0d} \cdot l^2}{384 \cdot EI} \cdot \delta \tag{14}$$

$$W_{max} = 0.058\,\frac{M_A \cdot l^2}{EI} \cdot \delta \tag{15}$$

$$W_{max} = \frac{3 \cdot l^2}{48 \cdot EI} \cdot (M_A + M_B) \cdot \delta \tag{16}$$

$$N_{cr} = \frac{\pi^2 \cdot EI}{l_e^2} \tag{17}$$

The effect of the axial compressive force on the displacement in the middle of the members is:

$$\delta = \left(\frac{1}{1-\frac{N}{N_{cr}}}\right) \tag{18}$$

where N is axial action, $N_{cr.}$ is Euler critical force, M_A is on top of member bending moment, and M_B is on bottom of member bending moment.

Table 4. Examinations result of bearing capacity of the units – columns

ID	e_A(mm)	e_B(mm)	f_y(N/mm^2)	$N_{Exp.}$(kN)	N_{EC3}(kN)	N_{SS}(kN)	$N_{Exp.}$/ N_{EC3}	$N_{Exp.}$/ N_{SS}
S_1	0.00	0.00	275	191.60	200.25	191.12	0.96	1.00
S_2	3.00	0.00	275	127.71	124.78	140.44	1.02	0.91
S_3	3.00	3.00	275	107.85	107.56	112.82	1.00	0.96
Mean							0.99	0.96
Sta. Dev.							0.03	0.04

Table 5. Lateral displacement of the tested columns

ID	L(mm)	D(mm)	t(mm)	$w_{Exp.}$(mm)	$w_{Teor.}$(mm)	w_{SS}(mm)	$w_{Exp.}/w_{teor.}$	$w_{Exp.}/w_{SS}$
S_1	3150	113.4	2.80	9.96	9.13/18.30*	10.42	1.09/0.54	0.96
S_2	3150	113.4	2.80	17.26	13.27	19.00	1.30	0.91
S_3	3150	113.4	2.80	21.65	22.27	26.82	0.97	0.81
Mean							1.12/0.94	0.89
Sta. Dev.							0.17/0.38	0.07

Notice * Computation made heaving the in consideration the eccentricity from the imperfection of e_o= (L/600 and L/300)

Table 6. Value and ratio of the axial forces experimental force and the theoretical values of the force given by different computation methods

Method	N(kN)	$N_{Rd,sec.}$(kN)	$N/N_{Rd,sec.}$
S_1			
Experimental	191.60		0.71
SEISMOSOFT	191.12	270	0.71
Eurocode	200.25		0.74
S_2			
Experimental	127.71		0.47
SEISMOSOFT	124.78	270	0.46
Eurocode	140.44		0.52
S_3			
Experimental	107.85		0.40
SEISMOSOFT	107.56	270	0.40
Eurocode	112.82		0.42

4. Conclusion

During the design, the calculation methods and test results reveal that the static scheme significantly influences the load-bearing capacity of structural members. This directly affects the stability of the structure, making it a crucial consideration for designers. Therefore, careful attention must be paid to this phenomenon during the design process. Theoretical, experimental, and simulation results from the software have demonstrated relatively acceptable accuracy. It's worth noting that discrepancies in the results are influenced by various factors, including simplifications and recommendations provided by design codes. However, the accuracy of recorded measurements during the experimental tests must not be overlooked.

The ratio of the obtained results for the load-bearing capacity of columns for the static scheme S_1 compared to the static scheme S_2 shows a 33% lower value, while for the static scheme S_3, it shows a 44% lower value. From here, it is concluded that the load-bearing capacities of steel columns with hollow sections serve their supporting function. In this context, it is noted that the static scheme S_1, which uses columns made of steel with hollow sections, has higher load-bearing capacities compared to the other schemes, S_2 and S_3. This result suggests that for this specific project, scheme S_1 provides better stability and performance in supporting the columns.

The theoretical computation results obtained from this study have influenced the classification criteria of cross-sections. These criteria, as applied within EN 1993-1-1 for open or closed cross sections, such as "HOP" and "I," may be considered conservative for cases involving circular cross-sections and compression with bending. Therefore, this issue remains a subject that warrants further analysis in future research endeavors.

However, the recommended value of imperfection, as stipulated by design codes, plays a significant role in the final result. In testing scenarios, it exerts notable effects, particularly in the calculation of lateral displacements. EN 1993-1-1 recommends a specific value, but various authors propose different values. Consequently, this discrepancy in recommendations could be considered an issue for further research and analysis. Conducting a larger number of tests may lead to more accurate recommendations in this regard. Aside from the static scheme, the material properties, and geometry are critical factors affecting the loss of load-bearing capacity in steel columns. Particularly, the L/d ratio, representing the geometry, holds significant importance. Therefore, the investigation of this ratio will be the primary focus of future studies conducted by the authors of this work.

5. Declarations

5.1. Author Contributions

Conceptualization, F.G., A.M., D.K, G.R.R., and N.S.H.; methodology, F.G., A.M., and D.K.; software, F.G., A.M., and D.K.; validation, F.G., A.M., and D.K.; formal analysis, F.G., A.M., D.K., and G.R.R.; investigation, F.G., A.M., D.K., and N.S.H.; resources, F.G., A.M., and D.K.; data curation, F.G., A.M., and D.K.; writing—original draft preparation, F.G., A.M., D.K., G.R.R., and N.S.H.; writing—review and editing, N.S.H.; visualization, F.G. and A.M.; supervision, F.G and E.SH.; project administration, F.G., A.M., and D.K.; funding acquisition, F.G., A.M., D.K., G.R.R., and N.S.H. All authors have read and agreed to the published version of the manuscript.

5.2. Data Availability Statement

The data presented in this study are available in the article.

5.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

5.4. Conflicts of Interest

The authors declare no conflict of interest.

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