Comparison of the Design Norms NP 005/2003 and EUROCODE 5. Elements Subjected to Compression Parallel to the Fibers

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***Abstract*—The paper aims to compare the two main design norms used in Romania for the dimensioning and verification of timber structural elements, namely NP 005 and Eurocode 5, highlighting the advantages and the disadvantages of each norm in comparison to each other, as well as their common points. An application was solved using both design norms for the case of elements subjected to compression parallel to the fibers. Since the design methods differ greatly one with respect to the other, a strict comparison of the obtained results could not be performed per element. Thus, multiple sets of data were introduced for the dimensions of the cross-section resulting in 34 distinct possible area combinations and three heights for the column were chosen, thus affecting the buckling length. For all these combinations only eight principle slenderness coefficients were found. Multiple sets of data were chosen for the forces acting on the timber element in order to see which possible combinations would yield valid results, without the need to resize the element. A matrix solution and representation were found.**

***Index Terms*—buckling, compression parallel to the fibers, structural timber.**

I. Introduction

Nowadays timber from well-managed forests represents one of the most sustainable resources available and together with stone it is one of the oldest known materials used in construction works. Examples of this fact are seen in many historic buildings all around the world.

Some important advantages of using wood as a construction material are:

- light weight (3.5 up to 15 times lighter than other construction materials [1]) with a high strength-to-weight ratio. Because of this characteristic, the seismic forces, which are proportional to the structural mass, are substantially lighter than for other types of construction materials (heavy steel and concrete structures experience greater forces);
- capability of withstanding and transferring both tension and compression forces (unlike concrete);
- inherent ductility due to numerous nailed connections and joints (unlike rigid masonry and concrete systems).
- good insulating properties to heat, sound and electricity if the material doesn't contain a high humidity percentage;
- no expansion joints required due to reduced dimensional variation along the fibers of the elements at temperature variations.

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As with all construction materials, some important disadvantages exist as well, but which, identified and taken into account from the start, can be easily surmounted.

Some limitations for wood as a construction material are:

- Limitations in the maximum cross-sectional dimensions and lengths of solid sawn timbers, due to available log sizes and natural defects. These are overcome by the recent developments in composite and engineered wood products. Finger jointing and various lamination techniques have enabled the creation of timber elements and systems of high quality in any shape, form and size to be constructed. Pre-assembled, modern, wooden elements are limited only by the manufacturing and/or transportation boundaries.
- Limitations due to environmental conditions: unlike some structural materials such as steel or concrete, the properties of timber are very sensitive to environmental conditions. For example, the moisture content in an element has a direct effect on the strength (it can be reduced by even 40%) and stiffness, swelling or shrinkage of timber. A proper understanding of the physical characteristics of timber enables the building of safe and durable timber structures.
- Varied strengths depending on the orientation of the applied forces with respect to the direction of the wooden fibers. For example, the compressive strength for pine tree essence on the parallel direction, according to NP 005, is 15 N/mm^2 and on transverse direction is 3.3 N/mm^2. This, unlike for concrete and steel, happens due to the fact that timber is a natural, organic, anisotropic, non-homogeneous material.

II. Design Norms

The legal design codes accepted in Romania for the dimensioning and verification of structures made of timber are: NP 005-2003 (revised in 2005) [2] and SR EN 1995-1-1-2004 [3] (Romanian equivalent of Eurocode 5: *EN 1995: Design of timber structures.*) The European norm *EN 1995* comprises three parts, all having the Romanian equivalent plus the national Annex. The three parts are:

- EN 1995-1-1 Design of timber structures – Part 1-1: General – Common rules and rules for buildings;
- EN 1995-1-2 Design of timber structures – Part 1-2: General – Structural fire design;
- EN 1995-2 Design of timber structures – Part 2: Bridges.

The SR EN 1995-1-1-2004 norm will be further referred to in this paper, in short, as EC5.

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Both norms stipulate the computation and design of the wooden structural elements based on the limit states method, namely ultimate limit state (ULS) and serviceability limit state (SLS), having the same definitions in both codes. ULS corresponds to a loss of strength or stability, practically forms of structural failure/collapse, and SLS is related to excessive deformations or vibration conditions which alter the normal use of the construction. For the ULS one is dealing with extreme safety conditions and for the SLS it is the level of comfort and appearance that is being addressed.

The relation for the verification of elements to ULS according to NP 005 is [2]:

$$S_d \le R_d \tag{1}$$

in which:

S_d – design value of the loadings / stresses which result from the combinations of various actions acting simultaneously on an element;

R_d – design bearing capacity corresponding to an element, depending on the strength of the material, modulus of elasticity and cross-sectional characteristics.

The relation for the verification of the elements to the ULS according to EC5 is:

$$\frac{\gamma_F \cdot S_k}{C} \le \frac{\gamma_C \cdot k_{\text{mod}} \cdot f_k}{\gamma_M} \tag{2}$$

where:

$\gamma_F \cdot S_k$ – sum of the partial safety coefficients for each type of action multiplied with the corresponding action;

C – geometrical characteristic of the cross-section, function of the action to which the element is subjected;

γ_C – partial safety coefficient of the work conditions of the element;

k_{mod} – modification factor which takes into account the type of action the element is subjected to and the humidity conditions;

f_k – characteristic strength of timber;

γ_M – partial safety coefficient for the material, which takes into account the possibility of the characteristic value of the material (strength or stiffness) being less than the specified value and the effect of the scatter of values around the mean value.

The design bearing capacity of an element, to various loadings, is determined with the following general relation (NP 005):

$$F_i = R_i^C \cdot S_i \cdot m_{Ti} \tag{3}$$

in which:

R_i^C – design strength of timber at the loading "*i*", function of the wooden material, quality class of the material and exploitation conditions;

S_i – sectional characteristic at the loading "*i*";

m_{Ti} – coefficient which takes into account the effect of the (chemical) treatment of timber, function of the exploitation or service class.

EC5 does not have recommendations for the influence of the characteristic of wood due to its treatment. It does not have a m_{Ti} coefficient.

The computations are based on the importance class of the buildings according to the possible human and economic losses, quality class of the material and lifespan of the structure (or exploitation duration).

In NP 005 it is stated from the beginning that the timber elements and constructions must be checked in the elastic domain of the material during different stages of work like execution, exploitation, maintenance or transportation.

In both norms, the computations are performed for the most unfavorable design situations, meaning reduced strengths of the elements and increased actions on the elements, to ensure a high level of structural safety.

In NP 005, the influence of the work conditions on the design of the structural elements are taken into account through two coefficients, m_{ui} and m_{di}, which consider the environmental humidity and the duration of the applied actions.

EC5 has wider areas of application and addresses extra topics in comparison to NP 005. Some examples can be: the computation and verification of elements with variable or curved cross-sections, components and assemblies (like beams with thin web joined together with glue) or the dimensioning of connections with perforated or notched metallic plates or diaphragms for roofs, floors and walls. A special chapter is related to the deformations of the wooden elements transmitted by vibrations. For more details on continuous vibrations and shocks in buildings which influence the human body (1 Hz…80 Hz), the code makes reference to the norm SR ISO 2631-2:1997.

A big disadvantage when using EC5, in the authors' opinion, is the lack of tables with values for the characteristic strengths, modulus of elasticity and densities for the various species of wood, function of the class quality. To obtain these an additional norm is required: SR EN 338-2010: Glued laminated timber – Strength classes and determination of characteristic values [4].

NP 005 states two types of timber: hardwood (hornbeam, beech, ash, birch, poplar, acacia, oak) and softwood (fir, larch, spruce, pine).

Characteristic strengths are given for the equilibrium humidity of 12% and for duration of the applied forces of maximum 3 minutes.

Three quality classes are given in the norm for which the characteristic values according to the type of load and timber species are specified. Wood of the third quality class cannot be used for structural elements.

SR EN 338:2010 classifies coniferous structural timber into 12 quality classes (C14-C50) and broad-leaved into 8 classes (D18-D70), according to the conditions imposed by SR EN 1912:2012 Structural timber [5].

The design strength for timber is computed with respect to the different wood species, at various loading types, function of the exploitation conditions of the construction elements. The relation is as follows (NP 005):

$$R_i^C = \frac{m_{ui} \cdot m_{di} \cdot R_i}{\gamma_i}, \tag{4}$$

where:

m_{ui} – work conditions coefficients which introduce the equilibrium humidity of the wooden material;

m_{di} – work conditions coefficients which introduce the duration of the loadings;

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R_i – characteristic strengths of various wooden species at various loadings;

γ_i – partial safety coefficients function of the loading type.

All of the above coefficients are given in tables, in the NP 005 norm.

For the design strength value according to EC5, the following formula is employed:

$$f_d = k_{\mathrm{mod}} \cdot \frac{f_k}{\gamma_M} \tag{5}$$

in which:

f_k – characteristic strength of timber, as in (2);

k_{mod} – modification factor as in (2);

γ_M – partial safety coefficient which takes into account the type of the cross-section and the material, unlike NP 005 which takes into account the type of loading on the element, as in (2).

III. Service Classes

Because the strength and creep characteristics of timber and wood-related products are influenced by the moisture content of the material, these properties are dependent on the temperature and relative humidity conditions the wooden structural elements will be subjected to over the design life of the building. When the moisture content is low, the strength capacity will be at its maximum and as the moisture content increases the strength is reduced and will reach a minimum value at the fiber saturation point. To take this effect into account in the design process, three service classes have been defined in both norms, covering the typical environmental conditions that timber structures will function under. These are as follows:

Service class 1 – where the average moisture content in most softwoods will not exceed 12%. This corresponds to a temperature of 20°C and a relative humidity of the surrounding air only exceeding 65% for a few weeks per year.

Service class 2 – where the average moisture content in most softwoods will not exceed 20%. This corresponds to a temperature of 20°C and a relative humidity of the surrounding air only exceeding 85% for a few weeks per year.

Service class 3 – where the average moisture content in most softwoods exceeds 20%.

IV. Elements Subjected to Compression Forces Parallel to the Fibers

A. *Formulae according to NP 005-03*

The strength capacity of the timber elements made from solid wood, having a simple cross-section, subjected to axial compression forces parallel to the fibers, C_r, is established with the following relation:

$$C_r = R_{c\|}^{c} \cdot A_{calc} \cdot \varphi_c \cdot m_T \tag{6}$$

in which:

$R_{c\|}^{c}$ – represents the design strength of the solid wood element to axial compression parallel to the fibers, function of the wood species, the quality class of the wood and the exploitation conditions of the construction elements. It is computed according to (4);

A_{calc} – the design area of the cross-section function of the gross or net area of the most loaded section;

φ_c – buckling coefficient, less than one;

m_T – wood treatment coefficient, given in tables.

B. *Formulae according to EC5*

The following relation must be satisfied:

$$\sigma_{c,0,d} \le f_{c,0,d} \tag{7}$$

in which:

$\sigma_{c,0,d}$ – represents the design value of the compression stress along the fiber;

$f_{c,0,d}$ – represents the design value of the compression strength along the fiber.

C. *Computations according to both norms*

Given the available data for a timber column, listed below, the maximum load (NP 005) which its cross-section can withstand or the strength capacity (EC5) must be computed.

C1. NP 005

Data:

The cross-section of the element is rectangular, 15 *cm* × 10 *cm*, made of softwood, quality class II;

Height of the column is, $L = 3.5\,m$;

The exploitation class is 2;

The wooden material is fireproofed;

The column is subjected the compression parallel to its fibers and thus the required formula for its strength capacity is, according to (6):

$$C_r = R_{c\|}^{c} \cdot A_{calc} \cdot \varphi_c \cdot m_T \tag{8}$$

in which the index $c\,\|$ represents the chosen type of action, namely compression parallel to the fiber.

In the above equation the terms can be computed or taken directly from tables as follows:

$R_{c\|}^{c} = 6.912\ N/mm^2$;

$A_{calc} = A_{gross}\,(\text{without weakening}) = 150\ cm^2$;

$m_T = 0.9$;

φ_c – buckling coefficient which depends on the slenderness, λ_f, of the element.

$$\lambda_f = \frac{\text{buckling length}}{\text{radius of gyration}} = \frac{l_f}{i};$$

$$i = \sqrt{\frac{I}{A}};$$

in which:

I – moment of inertia;

A – area of the cross section.

In order to determine the buckling lengths in each plane, the column must firstly be optimally positioned. Instead of working with *Ox* and *Oy* axes, which are unknown in this calculation step, axes Δ_{1C} and Δ_{2C} will be used. The indexes 1 and 2 represent the principal directions of the moments of inertia and *C* indicates that the axes pass through the centroid of the cross-section.

$I_{\Delta 1C} = 1250\ cm^4$;

$I_{\Delta 2C} = 2812.5\ cm^4$.

Radii of gyration in both planes:

$i_{\Delta 1C} = \sqrt{\frac{I_{\Delta 1C}}{A}} = 2.89\ cm$;

$i_{\Delta 2C} = \sqrt{\frac{I_{\Delta 1C}}{A}} = 4.33\ cm$.

The buckling length, with respect to the support conditions in each plane is:

$l_f^{XOY} = 2 \cdot L = 7\ m$;

$l_f^{XOZ} = 1 \cdot L = 3.5\ m$.

The optimum position of the cross-section can now be established:

$i_{\Delta 2C} > i_{\Delta 1C}$;

$l_f^{\max} = l_f^{XOY}$.

It results that $\Delta_{2C} \perp XOY$. It further results that:

$$\begin{matrix} \Delta_{2C} \parallel OZ \\ \Delta_{1C} \parallel OY \end{matrix} \rightarrow \begin{matrix} I_{\Delta_{2C}} \\ I_{\Delta_{1C}} \end{matrix} \text{ and } \begin{matrix} i_{\Delta 2C} = r_z = 4.33\ cm \\ i_{\Delta 1C} = r_y = 2.89\ cm \end{matrix}$$

The slenderness coefficients are:

$\lambda_y = \frac{l_f^{XOY}}{i_z} = 161.66$;

$\lambda_z = \frac{l_f^{XOZ}}{i_y} = 121.11$.

The maximum slenderness coefficient is chosen from the values above, $\lambda_{\max} = 162$ which leads to the minimum buckling coefficient $\varphi_{\min} = 0.118$, according to Table 4.3 from the NP 005 norm [2] .

Finally, the strength capacity for the element subjected to compression forces parallel to the fibers is:

$C_r = R_{c\parallel}^c \cdot A_{calc} \cdot \varphi_c \cdot m_T = 11.011\ kN$.

If the strength capacity is divided by the area of the cross-section, then the unit stress is 0.734 *MPa*.

C2. EC5

Data similar to the previous example but chosen according to EC5:

The cross-section of the element is the same as before: rectangular, 15 *cm* × 10 *cm*, made of softwood, quality class C14;

Height of the column is, L = 3.5 *m*;

The exploitation class is 2.

Additionally, the element is subjected to permanent axial forces: $G = 35\ kN$ and short-duration variable loads: $q = 16\ kN/m^2$.

The geometrical characteristics, A, I_y, I_z remain the same as in the previous example.

The design actions are obtained by multiplying the given loads with partial safety coefficients for the ULS, fundamental combination, as follows:

$G_d = 47.25\ kN$

$q_d = 24\ kN/m^2$

The variable action, q_d , which is a uniformly distributed load on the surface of the element must be multiplied by the cross-sectional area of the column in order to obtain a concentrated load, as G_d .

$Q_d = q_d \cdot A = 0.36\ kN$

The total effect of the actions on the element is:

$F_d = G_d + Q_d = 47.61\ kN$

The material characteristics for the quality class C14 are as follows:

$f_{c0k} = 16\ N/mm^2$

$E_{0.05} = 4.7\ N/mm^2$

For the design value, (5) is employed:

$$f_{c0d} = k_{\text{mod}} \cdot \frac{f_{c0k}}{\gamma_M} \tag{9}$$

in which the index c_0 represent the chosen type of action, namely compression parallel to the fibers (0-degree angle).

In (9), the terms are as follows:

$\gamma_M = 1.3$;

$k_{\text{mod}1} = 0.6$, for permanent actions, exploitation class II;

$k_{\text{mod}2} = 0.9$, for variable, short-duration actions, exploitation class II.

The final value for the k_{mod} coefficient is:

$$k_{\text{mod}} = \frac{k_{\text{mod}1} \cdot G_d + k_{\text{mod}2} \cdot Q_d}{F_d} = 0.602$$

$f_{c0d} = 7.413\ N/mm^2$.

The strength verification required from the norm is (7). It results that:

$\sigma_{c0d} \le 8.98\ N/mm^2$;

$\sigma_{c0d} = \frac{F_d}{A} = 3.174\ N/mm^2$.

(7) is verified. Additionally, the stability check must be done for centric compression which may lead to buckling.

The relative buckling coefficients for each axis, according to Chapter 6.3 "*Stability of the elements*" from the EC5 norm, are:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c0k}}{E_{0,05}}} = 1.501;$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi} \sqrt{\frac{f_{c0k}}{E_{0,05}}} = 2.252 .$$

If pure axial compression is considered, without bending, then the following verification must be performed (EC5):

$$\frac{\sigma_{c0d}}{k_{c,y} \cdot f_{c0d}} \le 1;$$

$$\frac{\sigma_{c0d}}{k_{c,z} \cdot f_{c0d}} \le 1;$$

in which:

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} = 0.379;$$

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$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} = 0.180\,;$$

$$k_y = 0.5\left[1 + 0.2\left(\lambda_{rel,y} - 0.3\right) + \lambda_{rel,y}^2\right] = 1.747\,;$$

$$k_z = 0.5\left[1 + 0.2\left(\lambda_{rel,z} - 0.3\right) + \lambda_{rel,z}^2\right] = 3.230\,.$$

The verification can now be performed:

$$\frac{\sigma_{c0d}}{k_{c,y} \cdot f_{c0d}} = 1.131\,;$$

$$\frac{\sigma_{c0d}}{k_{c,z} \cdot f_{c0d}} = 2.375\,.$$

The stability check is not validated. To remedy this problem a few options are available. These can be:

- increasing the quality class of the wood;
- the softwood species could be changed to hardwood,
- the cross-section of the material can be increased to make the column less slender;
- the free end of the column in plane *XOY* should at least be simply supported, thus majorly lowering the buckling length from 7 *m* to 3.5 *m*. This last option can be easily implemented and it is quite rational since in the other plane, *XOZ*, the column is already simply supported.

Unlike in the previous case of the NP 005 norm, the stability check in this case was not validated due to the high values of the loads applied on the column. In the NP 005, the exterior loads are not taken into account or required by the design formulae.

V. Multiple Data

a. For the application given in the previous chapter, *C.1*, multiple sets of data are chosen for the dimensions of the cross-section and three heights for the column, thus affecting the buckling length. The length and width of the cross-section vary from 5 to 5 *cm*, starting from 10 *cm* till 45 *cm*, resulting in 34 distinct possible area combinations. For each of these combinations the height of the column was considered separately 3 *m*, 3.5 *m* and 4 *m*.

It resulted from computations that for all these combinations of dimensions (Fig. 1), there were only eight principle slenderness coefficients (Fig. 2, Fig. 3).

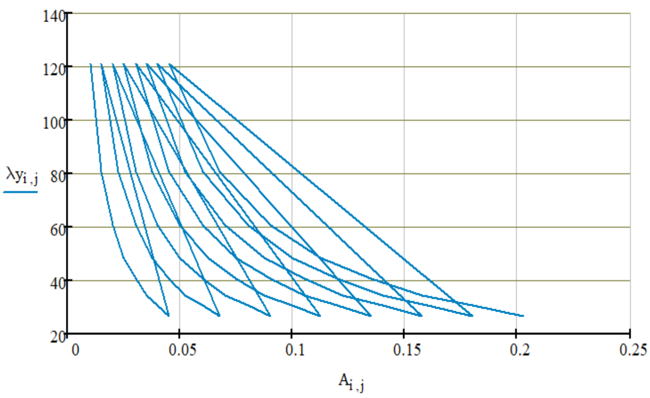


Figure 1. Representation of the slenderness coefficients on the *y* direction with respect to the area combinations, H = 3.5 *m*

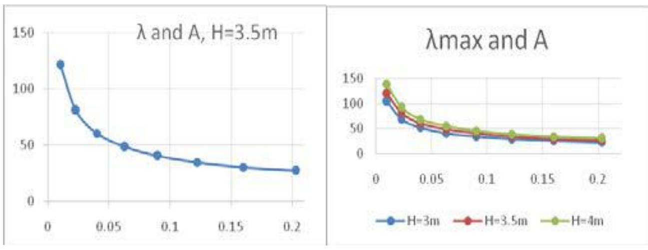
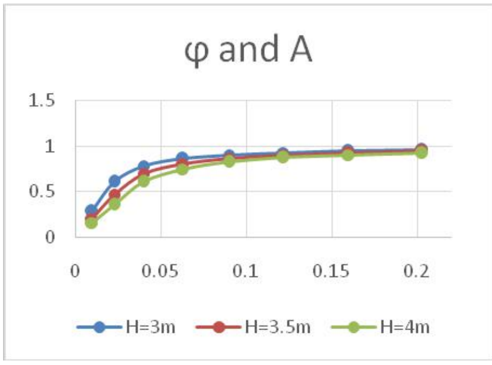
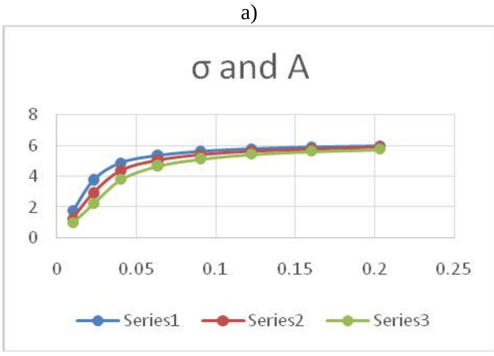


Figure 2. Representation of the principle slenderness coefficients with respect to the area, for H = 3.5 *m* and for all the considered heights

For this set of eight slenderness coefficients the buckling coefficients, strength capacities and stresses were computed.



a)



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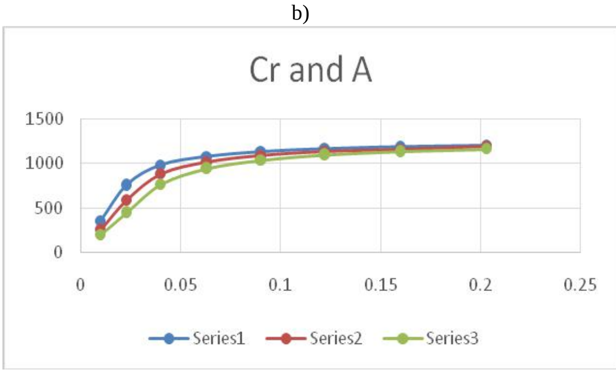

c)

Figure 3. Representation of: a) the buckling coefficients; b) the compression stresses; c) the strength capacities for all the heights

It can be observed that for values of the area of the cross-section below 50 cm^2, the slenderness coefficient is quite high and varies drastically, thus negatively influencing the stability of the element. Slender cross-sections, susceptible to buckling will also have lower strength capacities, but they will increase rapidly until an area of 50 cm^2.

The results of the computations for the three considered heights are listed below in Tables III.

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TABLE I. COMPUTATION FOR $H = 3.00$ M

A [m²]	λ	ϕ	σ [MPa]	Cr [kN]
0.01	103.92	0.287	1.785	361.54
0.023	69.282	0.608	3.782	765.91
0.04	51.962	0.784	4.877	987.61
0.063	41.569	0.859	5.344	1082
0.09	34.641	0.902	5.611	1136
0.122	29.692	0.928	5.773	1169
0.16	25.981	0.946	5.885	1192
0.203	23.094	0.958	5.96	1207

TABLE II. COMPUTATION FOR $H = 3.50$ M

A [m²]	λ	ϕ	σ [MPa]	Cr [kN]
0.01	121.24	0.208	1.294	262.02
0.023	80.829	0.472	2.936	594.58
0.04	60.622	0.702	4.367	884.32
0.063	48.497	0.808	5.026	1018
0.09	40.415	0.866	5.387	1091
0.122	34.641	0.902	5.611	1136
0.16	30.311	0.922	5.736	1161
0.203	26.943	0.947	5.891	1193

TABLE III. COMPUTATION FOR $H = 4.00$ M

A [m²]	λ	ϕ	σ [MPa]	Cr [kN]
0.01	138.56	0.16	0.995	201.55
0.023	92.376	0.358	2.227	450.98
0.04	69.282	0.608	3.782	765.91
0.063	55.426	0.749	4.659	943.52
0.09	46.188	0.823	5.12	1037
0.122	39.59	0.872	5.425	1098
0.16	34.641	0.902	5.611	1136
0.203	30.792	0.922	5.736	1161

It can be seen that for the first height difference, from 3 *m* to 3.5 *m*, the slenderness increases with 16.67% while for the second height difference, from 3 *m* to 4 *m*, it increases with 33.33%. For the buckling coefficient, in the first case, from 3 *m* to 3.5 *m* the maximum decrease is by 27.53% thus significantly reducing the design strength capacity, while the minimum decrease, for the largest area considered, is 1.15%. The buckling coefficient in the second case, from 3 *m* to 4 *m*, varies in the maximum case by 44.25% and in the minimum by 3.76%. Thus, keeping the same cross-sectional dimensions, it can be observed that for a height difference of just 0.5 *m*, the design strength capacity is reduced by more than a quarter, while for an increase in the height of the column of 1 *m*, the design strength capacity is reduced by almost a half.

b. For the application given in the previous chapter, *C.2*, multiple sets of data are chosen for the forces acting on the timber column in order to see which possible combinations will yield valid results, without the need to resize it. The chosen cross-section on which the varying actions are applied is the slenderest one, 10×15 *cm*, being the most susceptible to buckling. The column heights are kept as in the previous example to 3 *m*, 3.5 *m* and 4 *m*. The permanent loads, *G*, vary from 5 *kN* to 5 *kN*, starting from 10 *kN* till 45 *kN*. And the short-durations loads, *q*, uniformly distributed on the area, vary from 2 kN/m^2 to 2 kN/m^2, starting from 4 kN/m^2 till 18 kN/m^2. To these values the partial safety coefficients are applied, as before. For the permanent loads 1.35 and for the variable loads 1.5.

The sum of the forces required for computations is:

$$F_{d_{i,j}} = G_{d_{i,j}} + Q_{d_{i,j}}$$

in which:

the subscript *d* – represents the design value;

the subscripts *i* and *j* – represent the variation of the permanent and variable loads;

G – is the permanent load;

Q – is the uniformly distributed load, *q*, multiplied with the area of the cross-section of the column.

The resulting matrix is:

$$F_d = \begin{pmatrix} 13.59 & 13.635 & 13.68 & 13.725 & 13.77 & 13.815 & 13.86 & 13.905 \\ 20.34 & 20.385 & 20.43 & 20.475 & 20.52 & 20.565 & 20.61 & 20.655 \\ 27.09 & 27.135 & 27.18 & 27.225 & 27.27 & 27.315 & 27.36 & 27.405 \\ 33.84 & 33.885 & 33.93 & 33.975 & 34.02 & 34.065 & 34.11 & 34.155 \\ 40.59 & 40.635 & 40.68 & 40.725 & 40.77 & 40.815 & 40.86 & 40.905 \\ 47.34 & 47.385 & 47.43 & 47.475 & 47.52 & 47.565 & 47.61 & 47.655 \\ 54.09 & 54.135 & 54.18 & 54.225 & 54.27 & 54.315 & 54.36 & 54.405 \\ 60.84 & 60.885 & 60.93 & 60.975 & 61.02 & 61.065 & 61.11 & 61.155 \end{pmatrix} \text{kN}$$

Considering this force matrix, the following results were obtained:

$$k_{\text{mod,max}} = 0.609$$

$$k_{\text{mod,min}} = 0.600$$

$$f_{c0d,\text{max}} = 7.492\, N/mm^2$$

$$f_{c0d,\text{min}} = 7.390\, N/mm^2$$

TABLE IV. ANALOGY BETWEEN COMPUTATIONS

	$H = 3$ *m*	$H = 3.5$ *m*	$H = 4$ *m*	Difference from 3 *m* to 3.5 *m* [%]	Difference from 3.5 *m* to 4 *m* [%]
$k_{c,y}$	0.241	0.180	0.140	25.31	41.91
$k_{c,z}$	0.490	0.379	0.299	22.65	38.98

A representation of the buckling factors, for the three different heights considered, is shown in the Fig. 4.

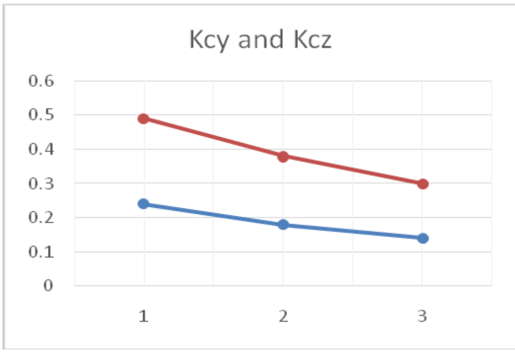

Figure 4. Representation of the buckling factors according to EC5 for the three column heights 3 *m*, 3.5 *m* and 4 *m*, on both *y* and *z* directions

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After the final verifications are performed,

$$Verif_y = \frac{\sigma_{c0d}}{k_{c,y} \cdot f_{c0d}} \le 1,$$

$$Verif_z = \frac{\sigma_{c0d}}{k_{c,z} \cdot f_{c0d}} \le 1$$

for the column height of 3 *m*, matrices like the ones below will result,

$$\mathrm{Verif_y} = \begin{pmatrix} 0.508 & 0.509 & 0.51 & 0.511 & 0.511 & 0.512 & 0.513 & 0.514 \\ 0.761 & 0.762 & 0.763 & 0.764 & 0.765 & 0.765 & 0.766 & 0.767 \\ 1.014 & 1.015 & 1.016 & 1.017 & 1.018 & 1.019 & 1.019 & 1.02 \\ 1.268 & 1.268 & 1.269 & 1.27 & 1.271 & 1.272 & 1.273 & 1.273 \\ 1.521 & 1.522 & 1.522 & 1.523 & 1.524 & 1.525 & 1.526 & 1.527 \\ 1.774 & 1.775 & 1.776 & 1.776 & 1.777 & 1.778 & 1.779 & 1.78 \\ 2.027 & 2.028 & 2.029 & 2.03 & 2.03 & 2.031 & 2.032 & 2.033 \\ 2.28 & 2.281 & 2.282 & 2.283 & 2.284 & 2.284 & 2.285 & 2.286 \end{pmatrix}$$

$$\mathrm{Verif_z} = \begin{pmatrix} 0.25 & 0.25 & 0.251 & 0.251 & 0.251 & 0.252 & 0.252 & 0.253 \\ 0.374 & 0.375 & 0.375 & 0.375 & 0.376 & 0.376 & 0.377 & 0.377 \\ 0.499 & 0.499 & 0.499 & 0.5 & 0.5 & 0.501 & 0.501 & 0.502 \\ 0.623 & 0.624 & 0.624 & 0.624 & 0.625 & 0.625 & 0.626 & 0.626 \\ 0.748 & 0.748 & 0.748 & 0.749 & 0.749 & 0.75 & 0.75 & 0.75 \\ 0.872 & 0.872 & 0.873 & 0.873 & 0.874 & 0.874 & 0.875 & 0.875 \\ 0.996 & 0.997 & 0.997 & 0.998 & 0.998 & 0.999 & 0.999 & 0.999 \\ 1.121 & 1.121 & 1.122 & 1.122 & 1.123 & 1.123 & 1.123 & 1.124 \end{pmatrix}$$

meaning that the possible values of the forces for which the element has both strength and stability are, in this case, for $G = 10\ kN$ and 45 *kN* and *q* having its entire considered range from 4 *kN/m*^{2} to 18 *kN/m*^{2}. All pairs of combinations are valid, for example 10 *kN* and 4 *kN/m*^{2} or 10 *kN* and 18 *kN/m*^{2} and so on. The same goes for the value of 45 *kN* paired with the values of *q*.

It can be seen that the loss of stability occurs faster on the *y* direction.

A representation of the above verifications is shown below (Fig. 5). The values for which the verifications are valid are located on the graph below the values of 1 on both axes.

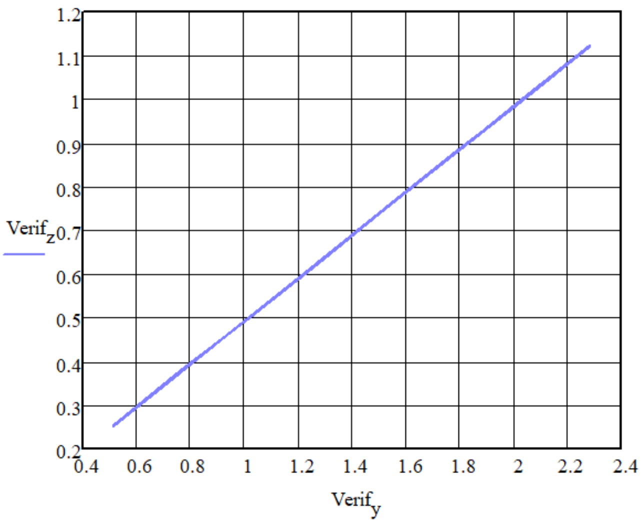

Figure 5. Verification of the timber column of height 3 *m*

Similarly, for the column heights of 3.5 *m* and 4 *m*, the pairs of forces are for $G = 10\ kN$ and *q* having its entire considered range from 4 *kN/m*^{2} to 18 *kN/m*^{2}.

VI. Conclusions

The design methods used in the above computations differ greatly one from the other in their approach to the dimensioning and verification of the timber structural elements and thus a strict comparison of the obtained results cannot be performed in this case, per element. However, an overall assessment of the strengths, stability and material consumption of the roof structure which will result from using the formulae and design methods from both codes can be performed and it will represent a future research purpose.

From the above computations the following conclusions can still be drawn:

The first method, according to NP 005, is a bit more accessible, being shorter, more straightforward and having no requirement of previous knowledge of the forces acting on the column.

The second method, according to EC5 depends on the type and duration of the actions.

Using the first method (NP 005) the strength capacity of the element is obtained while with the second method (EC5) it is verified that the element will keep its strength and stability at certain actions.

In both methods the assessment of the loss of stability through buckling is performed similarly.

In regard to the ease of access, the authors consider that NP 005 is better because it does not require the use of additional norms, like SR EN 338, for the values of the characteristic strengths. An important observation is that the characteristic values given in both norms, NP 005 and EC5 through SR EN338 vary greatly (in our example from 12 *N/mm*^{2} to 16 *N/mm*^{2}) as well as the classification of timber classes (12 classes versus 20 classes).

An important difference between the codes is the manner in which the work conditions – humidity and duration of the action – are taken into account in the design computations. For NP 005 there are 2 coefficients m_{ui} and m_{di} while for EC5 there is only one, k_{mod}, which represents a combination of m_{ui} and m_{di}. Also, according to EC5, a material has the same partial safety coefficient regardless of the loading type.

Another difference between the two codes is the application domain. EC5 is more restrictive, its stipulations being valid only for solid timber cross sections while EC5 has a wider area, including the more modern wooden materials like fiber boards, OSB or glue laminated timber.

Both codes consider the semi-probabilistic design methods of ultimate limit states. Both consider the same service classes and both limit the behavior of the material to the elastic domain with certain strengths reserves in the elasto-plastic one, given only by the various connections of the elements (for example steel nails).

Since both norms are in force, the requirements for both should be satisfied in the design of wooden structural elements. Fortunately, the provisions of both norms are not contradictory and usually if the requirements for one are satisfied the requirements for the other will be as well. Still, verification is advisable.

Since both norms have been in use for almost 15 years already, it is the authors' opinion that revised versions should be considered, especially since for the *EC5* amendments have been made during the years, the last one being done in 2014: "*SR EN 1995-1-1:2004/A2:2014/ Eurocod 5: Proiectarea structurilor de lemn. Partea 1-1: Generalităţi. Reguli comune şi reguli pentru clădiri*". This norm brings the modification of the following paragraphs of the initial code: *paragraph 1.2, 1.6, 2.2.3, 2.3.2.2, 4.2, 6.1.5, 6.1.8, 6.2.3, 6.5.2, 8.3.2, 8.4, 8.6, 8.7.1, 8.7.2, 8.8.5.1, 8.8.5.2, 8.9, 8.10, Annex A* and *Annex B.* A revised norm containing all the modifications throughout the years is advised for a better ease of work and greater efficiency.

References

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[5] *Timber. Resistance classes. Classification of classes and species by visual examination, SR EN 1912: 2012 / AC: 2013*, 2012 (in Romanian)

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