Saimaa University of Applied Sciences
Faculty of Technology, Lappeenranta
Double Degree Programme in Civil and Construction Engineering

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Comparison of basic steel structures' calculations by SP and Eurocode

Bachelor’s Thesis 2016

Abstract

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Comparison of basic steel structures' calculations by SP and Eurocode, 37 pages, 1 appendix.
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The purpose of the thesis work was to study how the basic steel structures are calculated according to Russian and European norms. All calculations were conducted according to actual norms of steel structures’ design (3-7). The thesis considers moment, shear and buckling resistance calculations, and also analysis of similarities and differences between European and Russian methods of design for steel I-beam.

The results of the study represent the step-by-step instruction for steel beam design with examples, which can be used by students of technical specialties or construction designers.

Keywords: steel structures, I-beam, Eurocode, SNiP, SP, Russian norms, basic calculations.

Content

1 Introduction

SNiP (Building Standards and Rules) were emerged in the USSR in 1955 as a set of provisions regulating design and construction processes. It was the first system of normative documents in the field of construction in the USSR. Nowadays, SNiP are divided into five sections of documents:

- Organization, management, economics
- Design standards
- Organization, production and acceptance of works
- Estimate standards
- Standards of material resources and labor costs

July 1, 2015 all of SNiP were canceled as mandatory and replaced by SP (Set of Rules).

SP 16.13330.2011 “Steel structures” applies to design of steel structures of buildings and constructions of different purposes, operating at temperatures above than 60 °C and lower than 100 °C. The rules do not apply to the design of steel structures of bridges, transport tunnels and culverts.

The Eurocodes are the set of harmonized technical standards specifying how structural design should be conducted within the European Union. The Eurocodes were developed by the European Committee for Standardization (1).

The purpose of the Eurocodes is to provide:

- a means to prove compliance with the requirements for mechanical strength and stability and safety in case of fire established by European Union law.

- a basis for construction and engineering contract specifications.

- a framework for creating harmonized technical specifications for building products.

The first Eurocodes were published in 1984, and by 2002, ten sections have been developed and published:

0. Basis of structural design (EN 1990)

1. Actions on structures (EN 1991)

2. Design of concrete structures (EN 1992)

3. Design of steel structures (EN 1993)

4. Design of composite steel and concrete structures (EN 1994)

5. Design of timber structures (EN 1995)

6. Design of masonry structures (EN 1996)

7. Geotechnical design (EN 1997)

8. Design of structures for earthquake resistance (EN 1998)

9. Design of aluminium structures (EN 1999)

In addition, each country has to have its own National Annex to the Eurocodes that takes into account the features (e.g. snow load factors) of a particular country.

EN 1993: "Design of steel structures" applies to the design of buildings and other civil engineering works in steel. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990 – Basis of structural design. EN 1993 is concerned with requirements for resistance, serviceability, durability and fire resistance of steel structures.

EN 1993 is wider in scope than most of the other design EN Eurocodes due to the diversity of steel structures, the need to cover both bolted and welded joints and the possible slenderness of construction. EN 1993 has about 20 parts covering common rules, fire design, bridges, buildings, tanks, silos, pipelined piling, crane supported structures, towers and masts, chimneys, etc (2):

EN 1993-1-1:2005: General rules and rules for buildings

EN 1993-1-2:2005: General rules - Structural fire design

EN 1993-1-3:2006: General rules - Supplementary rules for cold-formed members and sheeting

EN 1993-1-4:2006: General rules - Supplementary rules for stainless steels

EN 1993-1-5:2006: General rules - Plated structural elements

EN 1993-1-6:2007: Strength and stability of shell structures

EN 1993-1-7:2007: Strength and stability of planar plated structures subject to out of plane loading

EN 1993-1-8:2005: Design of joints

EN 1993-1-9:2005: Fatigue

EN 1993-1-10:2005: Material toughness and through-thickness properties

EN 1993-1-11:2006: Design of structures with tension components

EN 1993-1-12:2007: General - High strength steels

EN 1993-2:2006: Steel bridges

EN 1993-3-1:2006: Towers, masts and chimneys – Towers and masts

EN 1993-3-2:2006: Towers, masts and chimneys – Chimneys

EN 1993-4-1:2007: Silos

EN 1993-4-2:2007: Tanks

EN 1993-4-3:2007: Pipelines

EN 1993-5:2007: Piling

EN 1993-6:2007: Crane supporting structures

The main objective of the study is to compare two different calculation methods of basic structures in order to find similarities and differences between them, which will help to simplify the cooperation of Russian and EU engineers.

2 Calculations

2.1 Initial data

The example is the simply supported unrestrained beam in steel S275 located in typical office building.

It is assumed that F_{Ed} (design loading on the structure) = 50 kN/m and the span of the beam L = 6.0 m.

In this case, the maximum bending moment at the midspan will be:

$$M_{y,Ed} = \frac{F_{Ed}L^2}{8} = \frac{50 * 6^2}{8} = 225 \text{ kNm}$$

The maximum shear force nearby beam support:

$$V_{Ed} = \frac{F_{Ed}L}{2} = \frac{50 * 6}{2} = 150 \text{ kN}$$

Calculation scheme with moment and shear diagrams shown in Figure 1.

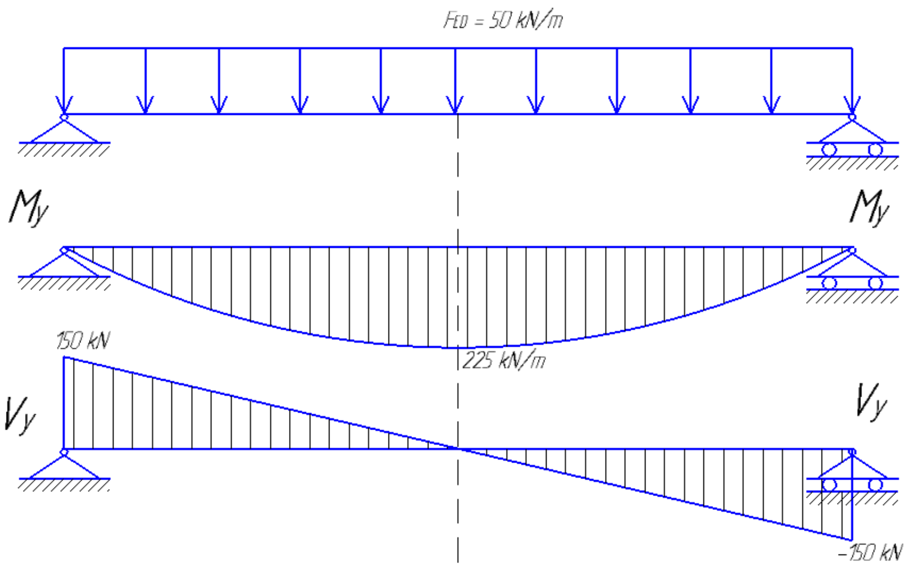


Figure 1. Moment and shear diagrams

2.2 Calculations by Eurocode

In accordance to EN 1993, the required cross-section needs to have a plastic modulus about the major axis (y-y) that is greater than:

$$W_{el,y} = \frac{M_{y,Ed}\gamma_{M0}}{f_y}$$

where γ_{M0} – partial factor.

The following numerical values for partial factors γ_{Mi} are recommended for buildings:

γ_{M0} = 1,00;

γ_{M1} = 1,00;

γ_{M2} = 1,25.

Assuming the nominal thickness of the element t≤40 mm, the yield strength is:

f_y = 275 N/mm^2

$$W_{el,y} = \frac{M_{y,Ed}\gamma_{M0}}{f_y} = \frac{225 * 10^5 * 1,0}{275 * 10^2} = 818,2\ cm^3$$

From the tables of section dimensions and properties (10) check the cross-section IPE 450, which has $W_{el,y} = 1500\ cm^3$.

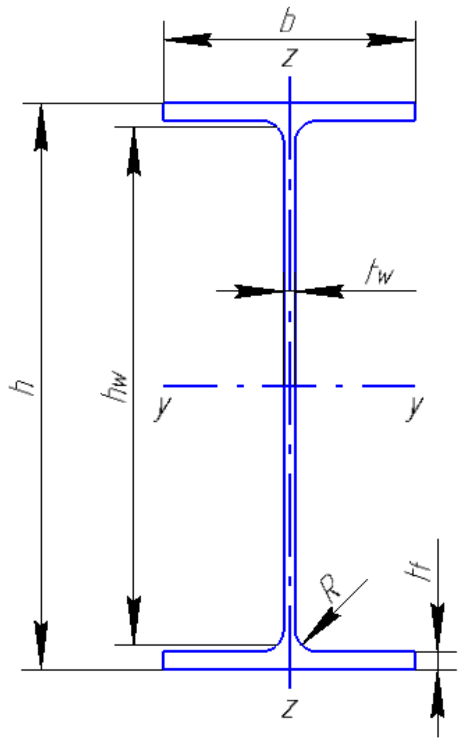


Figure 2. Dimensions of the cross-section

Dimensions							
h	**h_w**	**b**	**t_w**	**t_f**	**R**	**A**	**d**
mm	mm	mm	mm	mm	mm	mm^2	mm
450,0	420,8	190,0	9,4	14,6	21,0	9880	378,8
Properties							
G	**I_y**	**I_z**	**I_t**	**I_w**	**i_z**	**$W_{el,y}$**	**$W_{pl,y}$**
kg/m	cm^4	cm^4	cm^4	cm^4	cm	cm^3	cm^3
77,6	33740	1676	66,87	791,0	4,12	1500	1702

Table 1. Properties and dimensions of cross-section IPE 450

2.2.1 Classification of cross-sections

The role of cross section classification is to identify the extent to which the resistance and rotation capacity of cross sections is limited by its local buckling resistance (Table 2).

Assuming the section is class 1, then:

$$\frac{c_f}{t_f} \leq 9\varepsilon$$

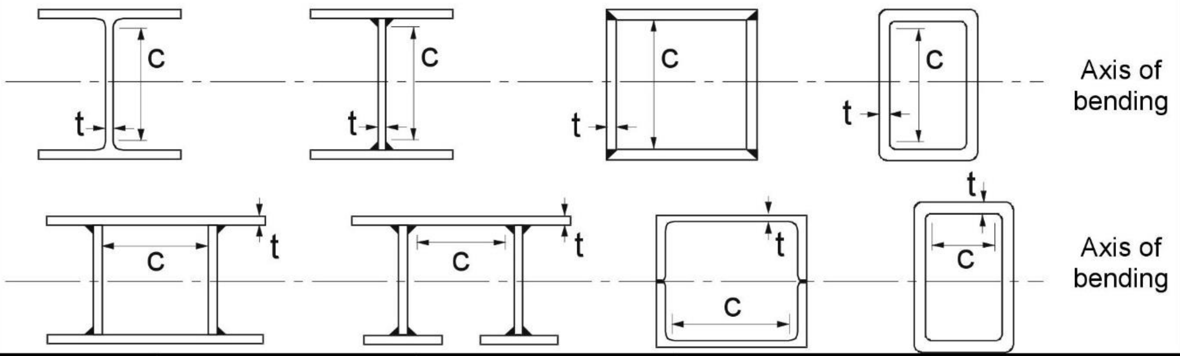
where ε is determined by the formula:

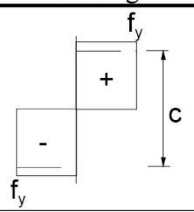
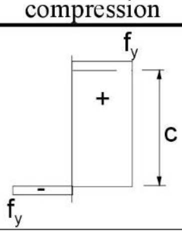
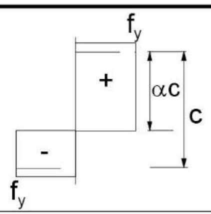
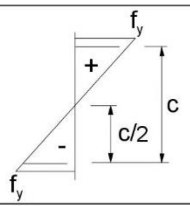
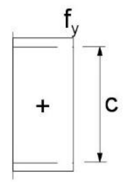
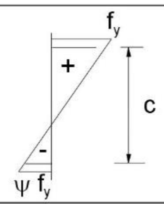
$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{275}} = 0,92$$

c_f – outstand flange (Figure 3), that is determined by the formula:

$$c_f = \frac{\mathrm{b}}{2} - \frac{t_w}{2} - R = \frac{190}{2} - \frac{9,4}{2} - 21,0 = 69,3\ \mathrm{mm}$$

Internal compression parts



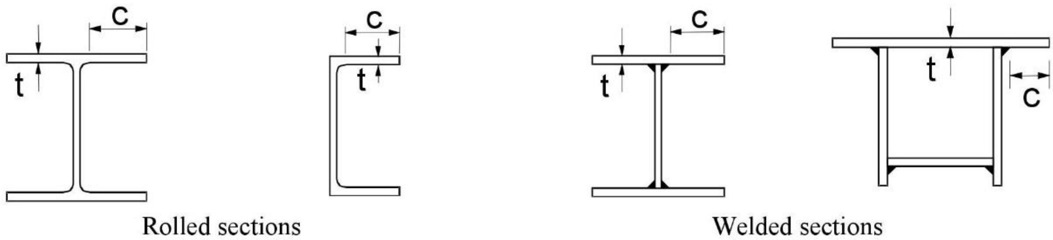
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \frac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \frac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \frac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \frac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1^{*)}$: $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

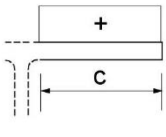
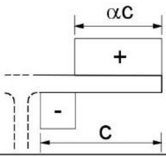
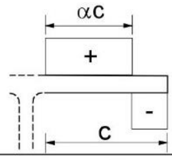
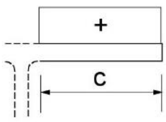
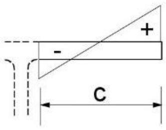
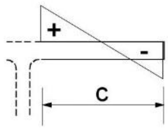
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Table 2. (sheet 1 of 3): Maximum width-to-thickness ratios for compression parts

Outstand flanges

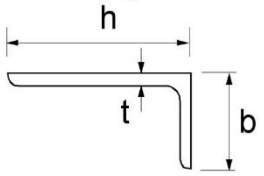


Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \dfrac{9\varepsilon}{\alpha}$	$c/t \le \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \dfrac{10\varepsilon}{\alpha}$	$c/t \le \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

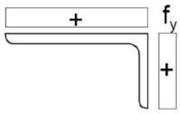
Table 2. (sheet 2 of 3): Maximum width-to-thickness ratios for compression parts

Angles

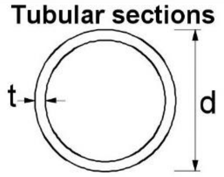


Refer also to "Outstand flanges" (see sheet 2 of 3)

Does not apply to angles in continuous contact with other components

Class	Section in compression
Stress distribution across section (compression positive)	
3	$h/t \leq 15\varepsilon : \ \dfrac{b+h}{2t} \leq 11{,}5\varepsilon$

Tubular sections



Class	Section in bending and/or compression
1	$d/t \leq 50\varepsilon^2$
2	$d/t \leq 70\varepsilon^2$
3	$d/t \leq 90\varepsilon^2$ **NOTE** For $d/t > 90\varepsilon^2$ see EN 1993-1-6.

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51

Table 2. (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

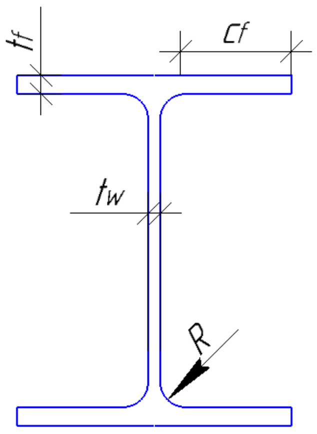


Figure 3. Outstand flange of a rolled cross-section

$$\frac{c_f}{t_f} \leq 9\varepsilon$$

$$\frac{63{,}9}{14{,}6} \leq 9 * 0{,}92$$

$$4{,}75 \leq 8{,}28$$

Therefore, the flange outstand in compression is Class 1.

In addition, it is necessary to check the internal compression part (Figure 4).

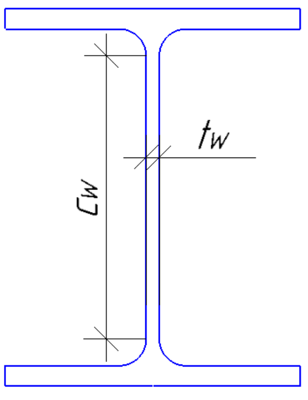


Figure 4. Internal compression part of a rolled cross-section

Class 1 section should satisfy:

$$\frac{c_w}{t_w} \leq 72\varepsilon$$

$$\frac{378{,}8}{9{,}4} \leq 72 * 0{,}92$$

$$40,30 \leq \ 66,24$$

Therefore, the section is Class 1.

2.2.2 Moment resistance

The design value of the bending moment M_{Ed} at each cross-section should satisfy:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq \ 1,0$$

The design resistance for bending is:

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,Rd} f_y}{\gamma_{M0}}$$

The $W_{el,y}$ is using as a safe solution here and after, also allowed to use $W_{pl,y}$ for class 1 cross-sections.

$$M_{c,Rd} = \frac{1500 * 10^3 * 275}{1,0} * 10^{-3}\ \mathrm{Nm} = 412500\ \mathrm{Nm} = 412,5\ \mathrm{kNm}$$

$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{225\ \mathrm{kNm}}{412,5\ \mathrm{kNm}} = 0,55 < 1,0$$

2.2.3 Shear resistance

The design value of the shear force V_{Ed} at each cross section should satisfy:

$$\frac{f_y/\sqrt{3}}{\gamma_{M0}} \geq \tau_{Ed} = \frac{V_{Ed} * S}{I * t_w}$$

where S – first moment of area, it can be approximately calculated as:

$$S = \frac{W_{pl,y}}{2} = \frac{1702}{2} = 851\ cm^3$$

$$\frac{275/\sqrt{3}}{1,0} \geq \frac{150 * 851}{33740 * 0,94}$$

$$\frac{150 * 10^3 * 851 * 1,0}{33740 * 0,94 * 10^2 * 275/\sqrt{3}} \leq 1$$

$$0{,}25 \leq 1$$

The shear resistance of the section is adequate.

Where the shear force V_{Ed} is less than half the plastic shear resistance $V_{pl,Rd}$ its effect on the moment resistance may be neglected.

2.2.4 Lateral torsional buckling resistance

A laterally unrestrained member subject to major axis bending should be verified against lateral-torsional buckling as follows:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq \ 1{,}0$$

where M_{Ed} – design value of the moment

$M_{b,Rd}$ – design buckling resistance moment.

The design buckling resistance moment of a laterally unrestrained beam should be taken as:

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}}$$

where W_y – appropriate section modulus ($W_y = W_{pl,y}$ for class 1 sections).

χ_{LT} – reduction factor for lateral-torsional buckling, which can be defined as:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta \bar{\lambda}_{LT}^2}} \text{ but } \begin{cases} \chi_{LT} \leq 1{,}0 \\ \chi_{LT} \leq \frac{1}{\bar{\lambda}_{LT}^2} \end{cases}$$

where Φ_{LT} is determined by formula:

$$\Phi_{LT} = 0{,}5 \left[1 + \alpha_{LT} \left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0} \right) + \beta \bar{\lambda}_{LT}^2 \right]$$

where α_{LT} is an imperfection factor, for buckling curve type b the imperfection factor is 0,34 (Table 3, 4);

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Table 3. Imperfection factors for buckling curves

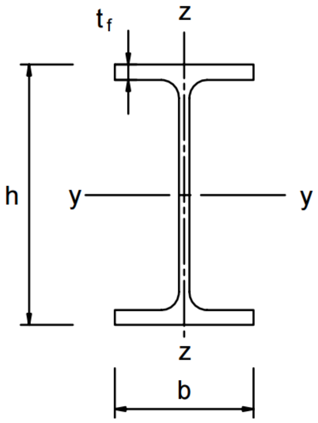
Cross section	Limits		Buckling about axis	Buckling curve	
				S 235 S 275 S 355 S 420	S 460
	h/b>1,2	$t_f < 40\ mm$	y-y	a	a_0
			z-z	b	a_0
		$t_f \leq 100\ mm$	y-y	b	a
			z-z	c	a
	h/b≤1,2	$t_f \leq 100\ mm$	y-y	b	a
			z-z	c	a
		$t_f > 100\ mm$	y-y	d	c
			z-z	d	c

Table 4. Selection of buckling curve for a rolled cross-section

$\bar{\lambda}_{LT}$ – non-dimensional slenderness.

The following values for $\bar{\lambda}_{LT,0}$ and β are recommended for rolled sections or equivalent welded sections:

$\bar{\lambda}_{LT,0} = 0{,}4$ (maximum value);

$\beta = 0{,}75$ (minimum value).

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

where M_{cr} is the elastic critical moment for lateral-torsional buckling.

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z}}$$

where G – the shear modulus (G = 80770 N/mm^2);

E – Young's modulus (E = 210000 N/mm^2).

For distributed load on a simply supported beam C_1 = 1,127.

$$C_1 \frac{\pi^2 E I_z}{L^2} = 1{,}127 * \frac{\pi^2 * 210000 * 10^6 * 1674 * 10^{-8}}{6^2} = 1086360\ N = 1086{,}36\ kN$$

$$\sqrt{\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z}} = \sqrt{\frac{791 * 10^{-12}}{1676 * 10^{-8}} + \frac{6^2 * 80770 * 10^6 * 66{,}87 * 10^{-8}}{\pi^2 * 210000 * 10^6 * 1674 * 10^{-8}}} = 0{,}237\ m$$

$$M_{cr} = 1086{,}36 * 0{,}237 = 257260\ Nm = 257{,}26\ kNm$$

Non-dimensional slenderness:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{1702 * 10^{-6} * 275 * 10^6}{257{,}26 * 10^3}} = 1{,}35$$

$$\Phi_{LT} = 0{,}5 * [1 + 0{,}34 * (1{,}35 - 0{,}4) + 0{,}75 * 1{,}35^2] = 1{,}34$$

$$\chi_{LT} = \frac{1}{1{,}34 + \sqrt{1{,}34^2 - 0{,}75 * 1{,}35^2}} = 0{,}498$$

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} = 0{,}498 * 1702 * 10^{-6} * \frac{275 * 10^6}{1} = 233165\ Nm$$
$$= 233{,}16\ kNm$$

Verification:

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{225}{233{,}16} = 0{,}96 < 1{,}0$$

Buckling resistance is adequate.

2.2.5 Calculation of deflections

The vertical deflection at the mid-span is determined by formula:

$$f = \int_L \frac{M_F \overline{M}_1}{EI} dx$$

where M_1 – moment diagram by the load P = 1 (Figure 5);

M_F – moment diagram by the design load (Figure 1).

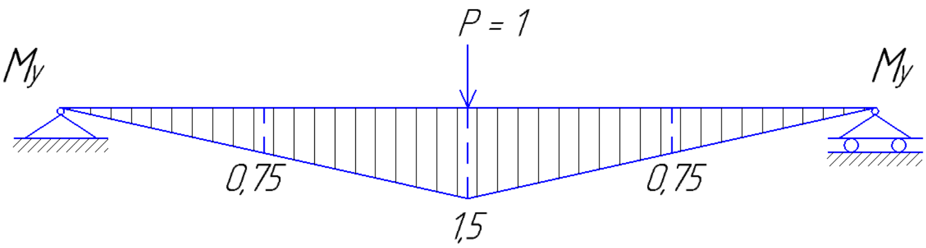


Figure 5. Moment diagram by the load P = 1

Integral can be calculated according to the Simpson's rule:

$$f = \int_L \frac{M_F \overline{M}_1}{EI} dx = \sum \frac{L}{6EI} \left[\overline{M}_1^b M_F^b + 4\overline{M}_1^m M_F^m + \overline{M}_1^e M_F^e \right]$$

where $\overline{M}_1^b, M_F^b$ – value of the moment at the beginning of section;

$\overline{M}_1^m, M_F^m$ – value of the moment at the middle of section;

$\overline{M}_1^e, M_F^e$ – value of the moment at the end of section.

Division the beam on the sections shown in Figure 6.

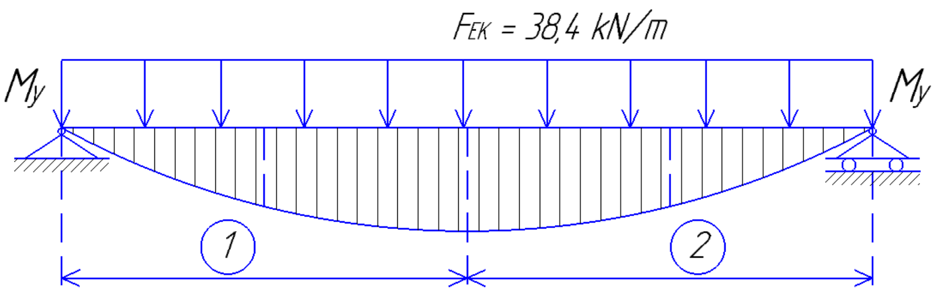


Figure 6. Division the beam on the sections

Vertical deflections should be calculated under the characteristic load combination F_{EK} due to variable loads.

$$f = \frac{3}{6EI}[0 + 4 * 130 * 0{,}75 + 173 * 1{,}5] + \frac{3}{6EI}[173 * 1{,}5 + 4 * 130 * 0{,}75 + 0] =$$

$$= \frac{3 * 1299}{6EI} = \frac{649{,}5}{EI} = \frac{649{,}5}{210000 * 33740 * 10^{-5}} = 0{,}0091\ m$$

$$= 9{,}1\ mm$$

Vertical deflection limit:

$$\frac{L}{300} = \frac{6000}{300} = 20{,}0\ mm$$

$$9{,}1\ mm < 20{,}0\ mm$$

Therefore, the vertical deflection at the mid-span of the beam is adequate.

2.3 Calculations by SP

This example uses the same profile as in the previous calculating (IPE 450 in S275)

2.3.1 Strength calculation

The beam under bending should satisfy:

$$\frac{M}{W_{n,min} R_y \gamma_c} \leq 1$$

where M – maximum bending moment ($M_{y,Ed}$ in EN 1993);

R_y – design value of steel's bending resistance by yield strength (according to GOST 27772-88 “Rolled steel for construction” $R_y = 275\ N/mm^2$);

γ_c – operating conditions factor (for an office building $\gamma_c = 1{,}0$)

$$\frac{225 * 10^3}{1500 * 275 * 1{,}0} = 0{,}55 \leq 1$$

The beam under shear force should satisfy:

$$\frac{QS}{I t_w R_s \gamma_c} \leq 1$$

where Q – shear force (V_{Ed} in EN 1993);

R_s – design value of steel's shear resistance that is determined by formula:

$$R_s = 0{,}58 * R_y = 0{,}58 * 275 = 159{,}5\ N/mm^2$$

It is important to note that factor 0,58 is approximately equal to $1/\sqrt{3}$ from European formula for shear capacity.

I – moment of inertia;

S – first moment of area (static moment), that value is usually given in Russian section dimensions and properties tables. European tables do not contain the first moment of area, so that it is necessary to calculate it for IPE 450.

S is also can be calculated using $W_{pl,y}$, as demonstrated above, but Russian section dimensions and properties tables do not contain the values of $W_{pl,y}$.

The first moment of area for the area F of any shape, and divided into n number of very small, elemental areas dF_i:

$$S_x = \sum_{i=1}^{n} y_i dF_i = \int_F y dF$$

$$S_y = \sum_{i=1}^{n} x_i dF_i = \int_F x dF$$

where x_i and y_i – distances to each elemental area measured from a given *x-y* axis.

To determine the first moment of area it is necessary to consider the half of the I-Profile, because the maximum value of shear stress τ located in the edge of profile. For simplicity cross-section is divided into areas of a simple shape (Figure 7).

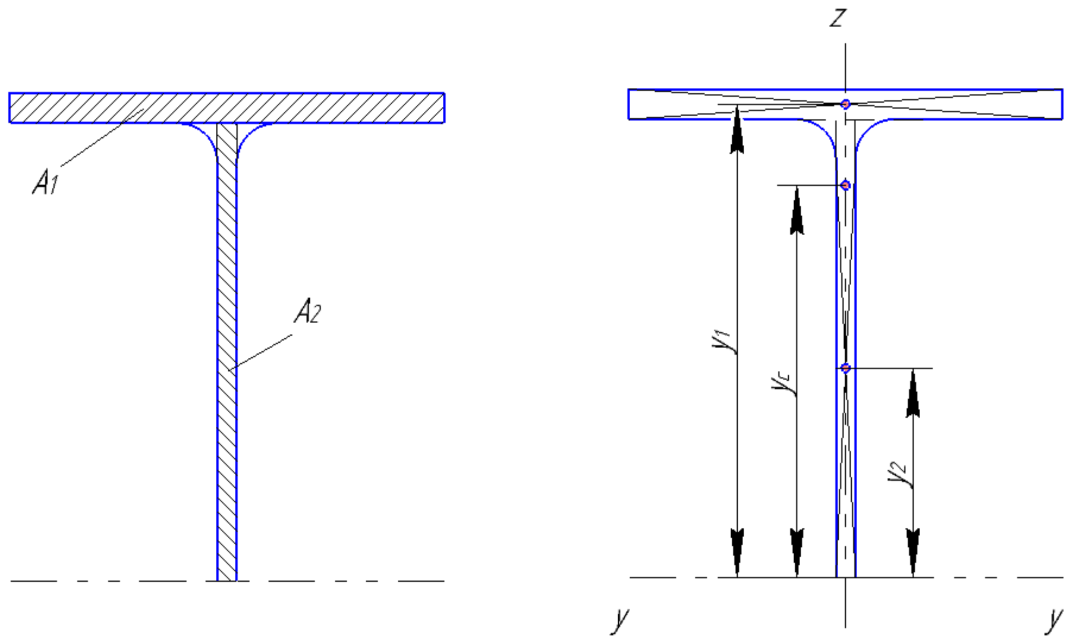


Figure 7. Half of the I-profile divided into rectangular shapes with centers of mass

The first moment of area for I-beam is determined by formula:

$$S = \frac{A}{2} y_c$$

where A – cross-sectional area;

y_c – coordinate of the center of mass for a half of I-profile (Figure 4) that is determined by formula:

$$y_c = \frac{A_1 y_1 + A_2 y_2 + \ldots + A_n y_n}{A_1 + A_2 + \ldots + A_n}$$

where A_n – n-area of a simple shape;

y_n – coordinate of the center of mass of the n-area.

$$y_c = \frac{A_1 y_1 + A_2 y_2}{A_1 + A_2} = \frac{b t_f * \frac{h_w + t_f}{2} + \frac{h_w t_w}{2} * \frac{h_w}{4}}{b t_f + \frac{h_w t_w}{2}} = \frac{b t_f \left(h_w + t_f\right) + \frac{1}{4} h_w^2 t_w}{2 b t_f + h_w t_w}$$

$$y_c = \frac{190 * 14{,}6 * (420{,}8 + 14{,}6) + \frac{1}{4} * 420{,}8^2 * 9{,}4}{2 * 190 * 14{,}6 + 420{,}8 * 9{,}4} = 170{,}87\ mm$$

$$S = \frac{A}{2}y_c = \frac{9880}{2} * 170{,}87 = 844125{,}80\ mm^3 = 844{,}12\ cm^3$$

The resulting value of the first moment of area is approximate, because of the flange to wall connection parts that are not taken into account.

$$\frac{QS}{It_w R_s \gamma_c} = \frac{150 * 10^3 * 844{,}12}{33740 * 0{,}94 * 10^2 * 159{,}5 * 1} = 0{,}25 \leq 1$$

Hence, shear resistance of the section is adequate.

2.3.2 Flexural stability calculations

$$\frac{M_y}{\varphi_b W_y R_y \gamma_c} \leq 1$$

where φ_b – flexural stability factor, for calculation scheme shown in Figure 1 it is determined by formula (for the case when part (flange) of the beam under load is compressed):

$$\left[\begin{array}{c} for\ (0{,}1 \leq \alpha \leq 40):\quad \varphi_b = 1{,}60 + 0{,}08\alpha \\ for\ (40 < \alpha \leq 400):\ \varphi_b = 3{,}15 + 0{,}04\alpha - 2{,}7 * 10^{-5}\alpha^2 \end{array}\right.$$

where α – factor that is defined by formula:

$$\alpha = 1{,}54 \frac{I_t}{I_z}\left(\frac{l_{ef}}{h}\right)^2 = 1{,}54 * \frac{66{,}87}{1676} * \left(\frac{600}{45}\right)^2 = 10{,}92$$

where l_{ef} – effective length, for the beam without the intermediate stiffeners it is assumed to be equal to the span of the beam.

Therefore, flexural stability factor is:

$$\varphi_b = 1{,}60 + 0{,}08\alpha = 1{,}60 + 0{,}08 * 10{,}92 = 2{,}47$$

$$\frac{M_y}{\varphi_b W_y R_y \gamma_c} = \frac{225 * 10^3}{2{,}47 * 1702 * 275} = 0{,}19 \leq 1$$

Conditional slenderness of the flange under compression $\bar{\lambda}_b$ should be equal or less than the limit slenderness $\bar{\lambda}_{ub}$.

$$\bar{\lambda}_b = \left(\frac{l_{ef}}{b}\right)\sqrt{\frac{R_{yf}}{E}}$$

Limit slenderness for the case when the upper flange is under load:

$$\bar{\lambda}_{ub} = 0{,}35 + 0{,}0032b/t + (0{,}76 - 0{,}02b/t)b/h \qquad \text{but} \qquad \begin{cases} 1 \le h/b \le 6 \\ 15 \le b/t \le 35 \end{cases}$$

where b and t – width and thickness of the compression flange;

h – distance (height) between the axes of the flanges.

$$1 \le h/b \le 6$$

$$1 \le (450 - 14{,}6)/190 \le 6$$

$$1 \le 2{,}3 \le 6$$

$$15 \le b/t \le 35$$

$$b/t = 13{,}0 < 15$$

When $b/t < 15$, need to take the value $b/t = 15$. Then limit slenderness is:

$$\bar{\lambda}_{ub} = 0{,}35 + 0{,}0032 * 15 + (0{,}76 - 0{,}02 * 15) * \frac{190}{450 - 14{,}6} = 0{,}60$$

Conditional slenderness:

$$\bar{\lambda}_b = \left(\frac{6000}{190}\right)\sqrt{\frac{275}{210000}} = 1{,}14$$

Flange stability is more than limit value, so it is necessary to install the intermediate stiffeners for reduction of the effective length l_{ef}.

Also it is necessary to check the slenderness of the wall $\bar{\lambda}_w$ that should be equal or less than 2,5:

$$\bar{\lambda}_w = \left(\frac{h_{ef}}{t_w}\right)\sqrt{\frac{R_{yf}}{E}} \le 2{,}5$$

where h_{ef} – effective height of the wall that is equal to d in properties and dimensions table (Table 1) and shown in Figure 8.

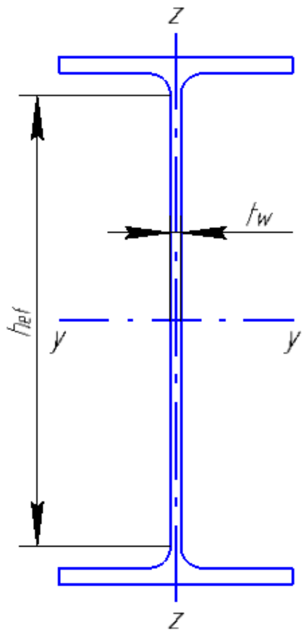


Figure 8. Effective height of the wall

$$\bar{\lambda}_w = \left(\frac{378{,}8}{9{,}4}\right)\sqrt{\frac{275}{210000}} = 1{,}46 \leq 2{,}5$$

In addition, the wall stability should be checked by the formula:

$$\sqrt{\left(\frac{\sigma}{\sigma_{cr}} + \frac{\sigma_{loc}}{\sigma_{loc,cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2} / \gamma_c \leq 1$$

where σ – maximum compressive stress in the wall;

τ – average shear stress;

σ_{loc} – local stress, in the absence of concentrated forces $\sigma_{loc} = 0$;

σ_{cr}, $\sigma_{loc,cr}$ and τ_{cr} – critical stresses.

Maximum compressive stress in the wall is determined by formula:

$$\sigma = \frac{M_y}{I_y} = \frac{225 * 10^2}{33740} = 0{,}67\ kPa$$

Average shear stress is determined by formula:

$$\tau = \frac{Q}{t_w h_w} = \frac{150 * 10^3}{9{,}4 * 420{,}8} = 37{,}92\ MPa$$

Normal critical stress is determined by formula:

$$\sigma_{cr} = \frac{c_{cr} R_y}{\overline{\lambda}_w^2}$$

where c_{cr} – factor that is depends on factor δ:

$$\delta = \beta \left(\frac{b_f}{h_{ef}}\right)\left(\frac{t_f}{t_w}\right)^3$$

where β – factor that is equal to 0,8.

$$\delta = 0{,}8 * \left(\frac{190}{378{,}8}\right) * \left(\frac{14{,}6}{9{,}4}\right)^3 = 1{,}50$$

For welded connections and $\delta = 1{,}50$, $c_{cr} = 32{,}4$, then:

$$\sigma_{cr} = \frac{32{,}4 * 275}{1{,}46^2} = 4189{,}86\ N/mm^2 = 4189{,}86\ MPa$$

Shear critical stress is determined by formula:

$$\tau_{cr} = 10{,}3 \left(1 + \frac{0{,}76}{\mu}\right) R_s / \overline{\lambda}_w^2$$

where μ – the ratio of larger side of the wall compartment (the distance between the intermediate stiffeners) to the lower, in the case of a beam without intermediate stiffeners the ratio is l_{ef}/h_{ef}.

$$\tau_{cr} = 10{,}3 * \left(1 + \frac{0{,}76 * 378{,}8}{6000}\right) * 159{,}5/1{,}46^2 = 809{,}61\ MPa$$

$$\sqrt{\left(\frac{\sigma}{\sigma_{cr}} + \frac{\sigma_{loc}}{\sigma_{loc,cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2} / \gamma_c = \sqrt{\left(\frac{0{,}67 * 10^{-3}}{4189{,}86}\right)^2 + \left(\frac{37{,}92}{809{,}61}\right)^2} / 1 = 0{,}047 \le 1$$

3 Comparison

In this chapter will be the analysis of the main differences in the beam design between SP and Eurocode norms. The differences between the typical I-beams used in Russia and Finland also will be explained.

The resistance to the bending moment, as demonstrated above, is calculated the same way for both standards:

$$M_{c,Rd} = \frac{W_y f_y}{\gamma_{M0}}$$

As can be seen, for the same values of γ_{M0}, the value of $M_{c,Rd}$ depends only on yield strength, which is characteristic of steel and W_y that depends on the cross-section shape. The values of yield strength in Finland and in Russia are the same for one type of steel. Therefore, it is advisable to consider the differences between cross-section shapes of typical I-beams.

The typical profiles of cross-sections in Russia slightly different from the Finnish profiles. In Europe, hot rolled I-beams height varies from 80 mm to 600 mm, but in Russia this value varies from 100 mm to 1013 mm. Some of profiles from Russian dimensions and properties tables have analogues in European tables (IPE 100 and 10Б1, IPE 200 and 20Б1), but most of them have not. If profile has analogue, that is mean that shape and dimensions of cross-section are the same for both European and Russian profiles. Therefore, in this case, the values of W_y are the same too.

Comparison of the resistance to bending moment $M_{c,Rd}$ for Russian and European I-beams with parallel flanges shown in Figure 9. The diagram reflects both analyses elastic and plastic for EU profiles (IPE). The same diagram for shear capacity $V_{c,Rd}$ shown in Figure 10.

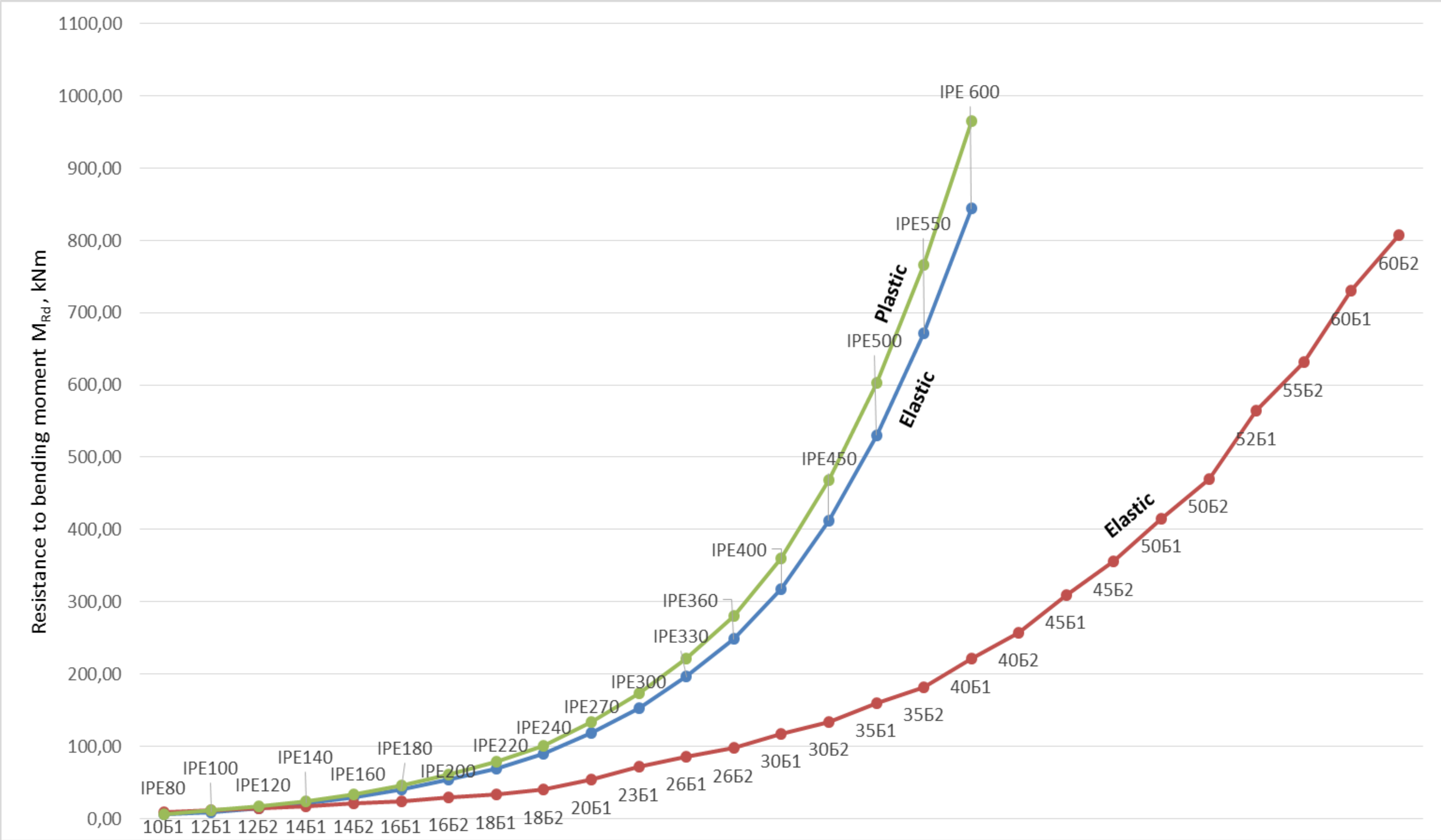


Figure 9. Value of the resistance to bending moment M_{Rd} for different profiles

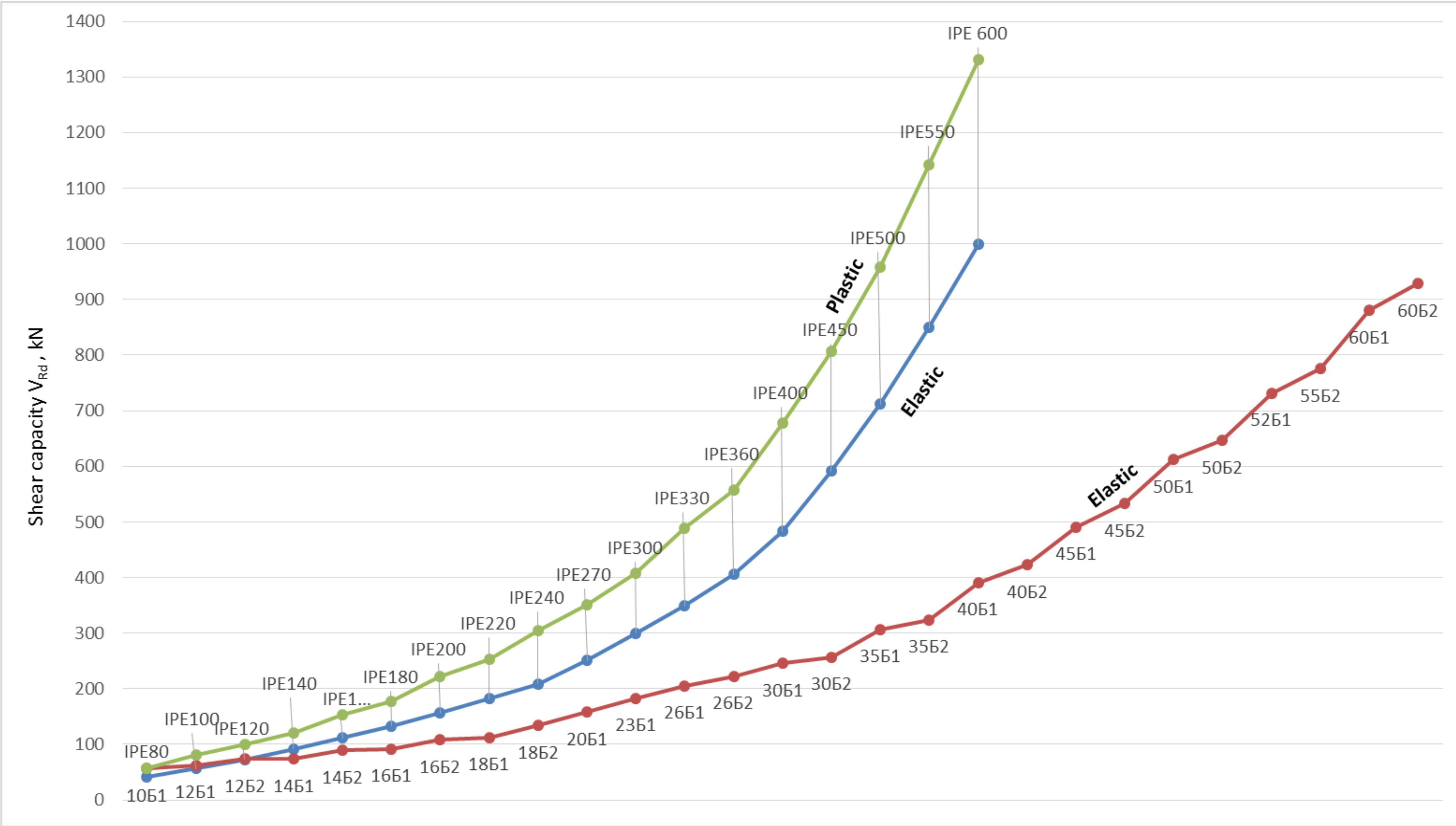


Figure 10. Value of the shear resistance V_{Rd} for different profiles

Formulas for determining of shear resistance are the same in both Russian and European norms, except for value $\frac{1}{\sqrt{3}}$, which is using in Eurocode's formula $\frac{f_y/\sqrt{3}}{\gamma_{M0}} \geq \tau_{Ed}$ that rounded to 0,58. Therefore, there is no need to compare the two calculation methods. However, it is possible to compare how to beam resists to shear in two plastic and elastic conditions. The diagram (Figure 11) demonstrates how ratio $\frac{V_{Ed}}{V_{c,Rd}}$ varies depending on the beam span.

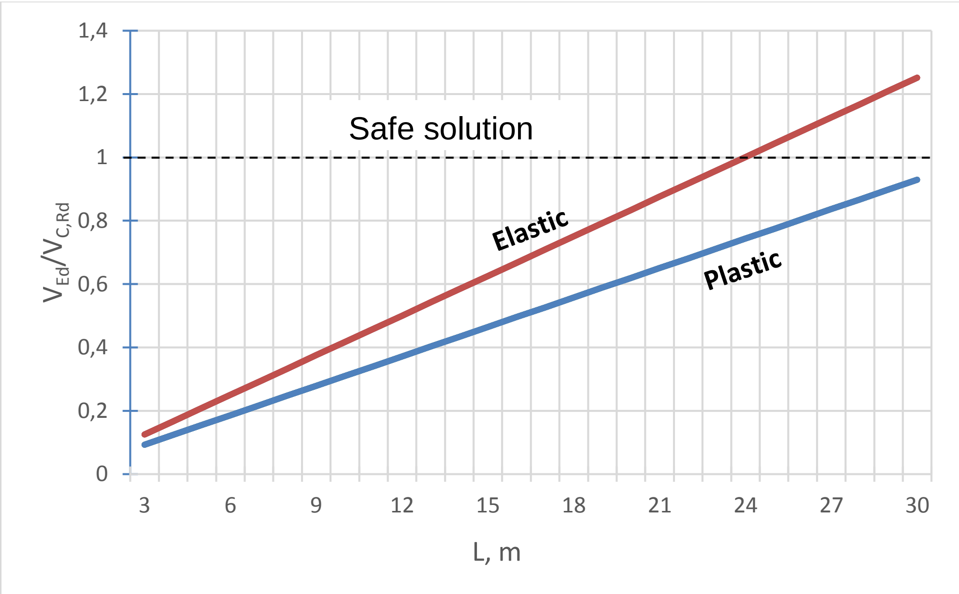


Figure 11. Function $f(L) = \frac{V_{Ed}}{V_{c,Rd}}$ in the elastic and plastic conditions

Methods of resistance to buckling calculation in two Russian and European norms are quite different. But it is possible to compare the values of slenderness $\bar{\lambda}$ ($\bar{\lambda}_b$ in SP and $\bar{\lambda}_{LT}$ in EN) for both cases. The Figure 12 demonstrates how to $\bar{\lambda}$ changes depending on the beam span for IPE 450.

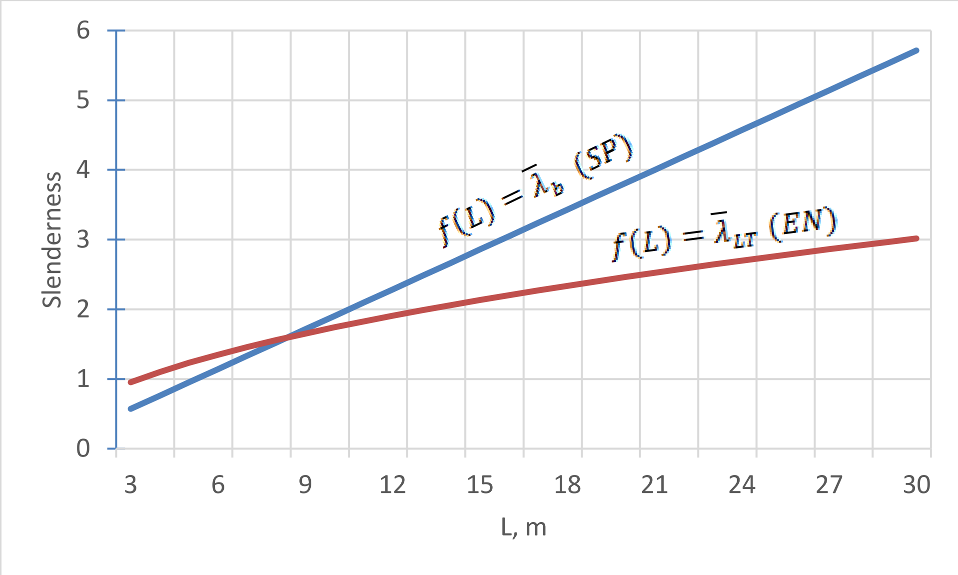


Figure 12. Function $f(L) = \bar{\lambda}$ by EN and SP $f(L) = \bar{\lambda}_{LT}\ (EN)$

4 Conclusion

Results of the work:

1. Calculation of steel I-beam, which includes cross-section classification, moment, shear and buckling resistance calculations and definition of deflections according to actual both Russian and European norms.

2. Analysis of differences and similarities between design methods and between Russian (Б) and European (IPE) I-beam profiles as well. Comparison of elastic and plastic conditions is also include.

3. Step-by-step instruction for steel beam design that includes formulas and references to chapters of Eurocode and SP.

The comparison of calculation methods shows that there is no any significant difference between strength design (moment and shear capacity) for EN and SP, except for various factors (partial, load, etc.). However, buckling calculation methods are quite different and should be considered in more detail.

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APPENDICES

APPENDIX 1 Methods of calculating the beam under bending according EN and SP (sheet 1 of 5)

All calculations appropriate for class 1 I-beam with parallel flanges under bending. Calculation scheme shown in Figure 1.

	EN	SP	
	Design load		
NA to EN 1990 "Basis of structural design"	Definition of design value of load $\mathrm{F_{Ed}}$		SP 20.13330.2011 "Loads and actions"
	Maximum bending moment and shear force		
	$\mathrm{M_{y,Ed}} = \frac{\mathrm{F_{Ed}L^2}}{8}$ $\mathrm{V_{Ed}} = \frac{\mathrm{F_{Ed}L}}{2}$	$M = \frac{q\mathrm{L}^2}{8}$ $\mathrm{V_{Ed}} = \frac{q\mathrm{L}}{2}$	

APPENDIX 1 Methods of calculating the beam under bending according EN and SP (sheet 2 of 5)

	Moment and shear resistance		
EN 1993 “Design of steel structures” - 6.2.5 EN 1993 - 6.2.6	$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1{,}0$$ $$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,Rd} f_y}{\gamma_{M0}}$$ Plastic: $$\frac{V_{Ed}}{V_{c,Rd}} \leq 1{,}0$$ $$V_{c,Rd} = V_{pl,Rd} = \frac{A_v (f_y/\sqrt{3})}{\gamma_{M0}}$$ $$A_v = A - 2bt_f + (t_w + 2r)t_f$$ $(A_v \geq \eta h_w t_w;\ \eta = 1{,}0)$ Elastic: $$\frac{f_y/\sqrt{3}}{\gamma_{M0}} \geq \tau_{Ed} = \frac{V_{Ed} * S}{I * t_w}$$ $$(S = \frac{W_{pl,y}}{2})$$	$$\frac{M}{W_{n,min} R_y \gamma_c} \leq 1$$ $$\frac{QS}{I t_w R_s \gamma_c} \leq 1$$ $$(R_s = 0{,}58 * R_y)$$ $$S = \frac{A}{2} y_c$$ $$y_c = \frac{A_1 y_1 + A_2 y_2 + \ldots + A_n y_n}{A_1 + A_2 + \ldots + A_n}$$	SP 16.13330.2011 “Steel structures” - 8.2.1

APPENDIX 1 Methods of calculating the beam under bending according EN and SP (sheet 3 of 5)

	Buckling calculations		
EN 1993 - 6.3.2.1	$\frac{M_{Ed}}{M_{b,Rd}} \leq 1{,}0$ $M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}}$	$\frac{M_y}{\varphi_b W_y R_y \gamma_c} \leq 1$ $\left[\begin{array}{l} for\ (0{,}1 \leq \alpha \leq 40): \quad \varphi_b = 1{,}60 + 0{,}08\alpha \\ for\ (40 < \alpha \leq 400): \ \varphi_b = 3{,}15 + 0{,}04\alpha - 2{,}7 * 10^{-5}\alpha \end{array} \right.$ $\alpha = 1{,}54 \frac{I_t}{I_z} \left(\frac{l_{ef}}{h} \right)^2$	SP 16.13330.2011 - 8.4.1 Annex Ж (Table Ж.1)
EN 1993 - 6.3.2.3 (Tables 6.3, 6,5)	$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta \overline{\lambda}_{LT}^2}}$ but $\begin{cases} \chi_{LT} \leq 1{,}0 \\ \chi_{LT} \leq \frac{1}{\overline{\lambda}_{LT}^2} \end{cases}$ $\Phi_{LT} = 0{,}5 \left[1 + \alpha_{LT} \left(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0} \right) + \beta \overline{\lambda}_{LT}^2 \right]$	$\overline{\lambda}_b \leq \overline{\lambda}_{ub}$ $\overline{\lambda}_b = \left(\frac{l_{ef}}{b} \right) \sqrt{\frac{R_{yf}}{E}}$ $\overline{\lambda}_{ub} = 0{,}35 + 0{,}0032 b/t + (0{,}76 - 0{,}02 b/t) b/h$ but $\begin{cases} 1 \leq h/b \leq 6 \\ 15 \leq b/t \leq 35 \end{cases}$	SP 16.13330.2011 - 8.4.4 (Table 11)

APPENDIX 1 Methods of calculating the beam under bending according EN and SP (sheet 4 of 5)

	Buckling calculations		
	$\bar{\lambda}_{LT,0} = 0{,}4$ (maximum value); $\beta = 0{,}75$ (minimum value). $$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$ $$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z}}$$	$$\bar{\lambda}_w = \left(\frac{h_{ef}}{t_w}\right) \sqrt{\frac{R_{yf}}{E}} \leq 2{,}5$$	SP 16.13330.2011 - 8.5.1
		$$\sqrt{\left(\frac{\sigma}{\sigma_{cr}} + \frac{\sigma_{loc}}{\sigma_{loc,cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2} / \gamma_c \leq 1$$ $$\sigma_{cr} = \frac{c_{cr} R_y}{\bar{\lambda}_w^2}$$	SP 16.13330.2011 - 8.5.3 (Table 12)
		$$\sigma = \frac{M_y}{I_y}$$ $$\tau = \frac{Q}{t_w h_w}$$	SP 16.13330.2011 - 8.5.2
		$$\delta = \beta \left(\frac{b_f}{h_{ef}}\right) \left(\frac{t_f}{t_w}\right)^3$$	SP 16.13330.2011 - 8.5.4
		$$\tau_{cr} = 10{,}3 \left(1 + \frac{0{,}76}{\mu}\right) R_s / \bar{\lambda}_w^2$$	SP 16.13330.2011 - 8.5.3

APPENDIX 1 Methods of calculating the beam under bending according EN and SP (sheet 5 of 5)

	Calculation of deflections	
NA to EN 1990	General formula: $$f = \int_L \frac{M_F \overline{M}_1}{EI} dx$$ Simplified formula: $$f = \frac{5l^4 F_{EK}}{384 E I_y}$$ Deflection limit definition	SP 20.13330.2011 Annex E.2.1 (Table E.1)