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NECESSARY HEIGHT OF THE RING BEAMS IN THE STEEL SILOS ON DISCRETE SUPPORTS. ANOTHER APPROACH FOR ITS DETERMINATION

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Keywords: *steel silos, meridional stresses, critical height, vertical stiffening element, loss of stability, material and geometrical nonlinearity, imperfections*

ABSTRACT

The steel silos are interesting complex facilities. For assuring their complete emptying by gravity, they are often placed on a carrying frame structure above ground. In the joint between the thin-walled shell and the carrying frame elements, the values of the stresses are extremely high. It can cause local loss of stability in the shell. For avoiding shell buckling, many designers put stiffening elements above the discrete supports. They are parts of the ring beams under the cylindrical body. The question is how high the mentioned stiffening elements, respectively the girder should be? The reasonable approach is they should rise to the level where the values of the meridional normal stresses above the discrete supports and in the middle between them are equal. But where is this level? Many researchers worked on the values and on the way of distribution of the meridional normal stresses. As a result of their efforts, the critical height H_{cr} of the shell and the ideal position H_I of the intermediate stiffening ring are determined. But those heights are very different and are determined for smooth steel shells, without vertical stiffening elements in them. The conclusions of the author, based on his previous researches, are that vertical stiffening elements considerably modify the picture and they must reach level situated between H_{cr} and H_I. The author tried to determine the necessary height of the stiffening elements using a new, alternative way, taking into consideration non-linear behaviour of the steel, the effects of the changes in the geometry during loading caused by welding operations imperfections and vertical stiffening elements.

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1. Introduction

Often steel silos are elevated facilities, placed above ground on a supporting structure. The purpose is to assure easy and complete emptying by gravity. The supporting structure is different for every project, depending on the real exploitation conditions. The most popular are two types – made by the horizontal girders and columns or only by columns. Both types of frame structures cause concentrated meridional forces in the cylindrical body of the silo. As a result, the thin-walled shell could lose stability.

The simplest way to design steel silo is hypothetically to divide the cylindrical shell into two parts – a discretely supported ring beam and a fully supported shell above it. This conception is accepted by the European standard EN 1993-4-1 [1], see Fig. 1. To ensure continuous support of the shell, the bending stiffness of the ring beam should be high. Unfortunately, EN 1993-4-1 does not mention the recommended stiffness of the ring beam.

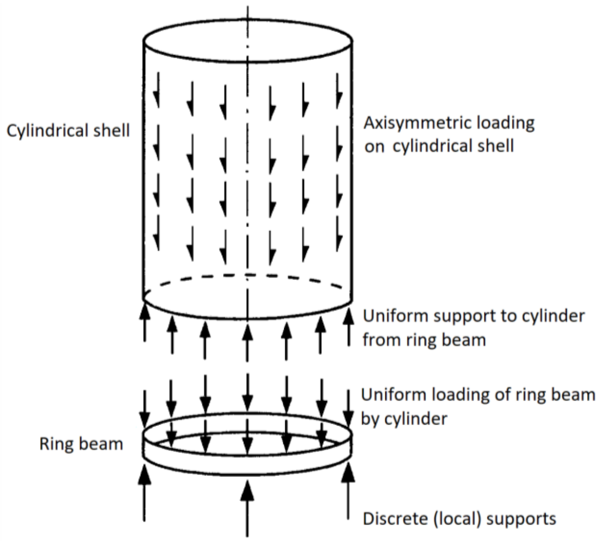


Figure 1. Traditional design model for silos on discrete supports according to EN 1993-4-1

In 1985 *Rotter* [2] offered the ratio ψ = 0,25 as applicable for design, where:

$$\psi = \frac{K_{shell}}{K_{ring}}, \tag{1}$$

in which K_{shell} is the stiffness of the cylindrical shell;

K_{ring} – the stiffness of the ring beam.

Based on the English translation of the study of *Vlasov* [3] for curved beams, the stiffness of the ring beam K_{ring} is calculated as:

$$K_{ring} = \frac{\left(n^2 - 1\right)^2 EI_r}{R^4} \frac{1}{f_r}, \tag{2}$$

where n is the number of uniformly spaced supports;

E – modulus of elasticity;

I_r – the moment of inertia about a radial axis;

R – radius of ring beam centroid.

$$f_r = 1 + \frac{EI_r}{n^2 K_T}, \tag{3}$$

in which:

$$K_T = GJ + n^2 \frac{EC_w}{R^2}, \tag{4}$$

where *G* is the shear modulus;

J – torsional constant;

C_w – warping constant for open sections.

The semimembrane theory of shells, proposed by *Vlasov* [4] gives the following expression for the stiffness of cylindrical shell:

$$K_{shell} = n\sqrt{\left(n^2-1\right)}\frac{E}{\sqrt[4]{3}}\left(\frac{t}{R}\right)^{3/2}\frac{1}{f_s}, \tag{5}$$

where *t* is the thickness of the cylindrical shell.

$$f_s = \frac{\left(e^{\eta}\right)^2 - 2e^{\eta}\sin\left(\eta\right) - 1}{\left(e^{\eta}\right)^2 - 2e^{\eta}\cos\left(\eta\right) + 1}, \tag{6}$$

in which:

$$\eta = \frac{2\pi H}{\mu}, \tag{7}$$

where *H* is the height of the cylindrical shell;

μ – expressed by *Calladine* [5] longwave bending half-wavelength:

$$\mu = \frac{2\pi\sqrt[4]{3}}{n\sqrt{\left(n^2-1\right)}}\sqrt{\frac{R}{t}}R. \tag{8}$$

Based on Eqs. (2) and (5), the stiffness ratio ψ will look as follows:

$$\psi = \frac{K_{shell}}{K_{ring}} = \frac{0,76\left(Rt\right)^2}{I_r}\sqrt{\frac{R}{t}}\sqrt{\frac{n^2}{\left(n^2-1\right)^3}}\frac{f_r}{f_s}. \tag{9}$$

In their researches, published in 2011, *Topkaya* and *Rotter* [6, 7], conducted extensive finite element analyses (FEA) for verification of the criteria of *Rotter* [2] about stiffness of ring beam. With 1 280 numerical models, having two different types of ring sections, various heights and radii of the cylindrical shells, the authors verified the validity of suggested by

Rotter in 1985 ratio $\psi = 0{,}25$. On the base of conducted FEA, researchers have concluded that when a stiffness ratio $\psi \leq 0{,}1$, the axial stresses will not deviate more than 25% from the uniform support assumption.

The research of *Zeybek*, *Topkaya* and *Rotter* [8] shows that the equations based on the theory of *Vlasov* [3] for a curved beam provide results with sufficient accuracy when the girder is separated from the cylindrical body. When the ring beam and the cylindrical shell are joined, the values received through finite elements analysis are considerably different from the analytical results in closed form. The differences become significant with the increase of the thickness of the cylindrical shell.

In 2014 *Topkaya* and *Rotter* [9] determined the ideal position of the intermediate stiffening rings on the shell. They expect that a ring placed at this ideal position can effectively remove all circumferential nonuniformity in the axial membrane stress above it. The simple expression of the ideal location H_I is:

$$H_I = \sqrt{12(1+\nu)}\,\frac{R}{n}, \tag{10}$$

where ν is *Poisson's* ratio.

It should be noted that all above-mentioned researches are conducted on the smooth steel shells without vertical stiffeners on them. On the other hand, the routine practice in the design of steel structures is to place stiffening elements on the point where the concentrated loads are applied. In the case of steel silos, the vertical stiffeners should be placed above the discrete supports, see Fig. 2.

Figure 2. Stiffening elements above discrete supports of the cylindrical shell

In his researches, *Zdravkov* [10, 11] shows that vertical stiffening elements increase the height of the critical zone, where the vertical reactions of discrete supports are redistributed, i.e. the longer the stiffeners are, the higher the critical zone will be. In that case, the question is how high the stiffeners should be? In another research, conducted with ideal cylindrical shells, but accounting their geometrical non-linearity (GNA), *Zdravkov* [12] tried to resolve this issue. Based on the idea of necessity of equalization of meridional normal stresses above the stiffeners and between them, he recommends the limits of the height H_S of the stiffeners to be $H_I \leq H_S \leq H_L$, where H_I represents the ideal height to place intermediate stiffening ring, calculated according to formula (10). H_L is the distance between discrete supports, calculated according to equation (13). In the latest research of *Zdravkov* [13], done with cylindrical shells with imperfections in them, accounting their geometrical and material nonlinearity (GMNIA) it is recommended that the height H_S of the stiffeners should be within the limits $H_{30} \leq H_S \leq H_L$. In this study, the author again followed the idea of equalizing the values of the meridional stresses.

In the current research, the author will try, using a new, alternative way, to determine the necessary height H_S of the ring beam, respectively of the vertical stiffeners, above which we can accept that the cylindrical shell is continuously supported on its bellows contour.

2. Analysis

For the purpose of the research, eight steel cylindrical shells are modelled, using software ANSYS [14]. Their parameters are as follows:

a) dimensions:

- shell 1 – diameter D = 1 m, height H = 2,2 m;
- shell 2 – diameter D = 2 m, height H = 4,4 m;
- shell 3 – diameter D = 3 m, height H = 6,6 m;
- shell 4 – diameter D = 4 m, height H = 8,8 m;
- shell 5 – diameter D = 5 m, height H = 11,0 m;
- shell 6 – diameter D = 6 m, height H = 13,2 m;
- shell 7 – diameter D = 7 m, height H = 15,4 m;
- shell 8 – diameter D = 8 m, height H = 17,6 m,

where D is the diameter of the cylindrical body;
H – the distance between stiffening rings, see Fig. 3.

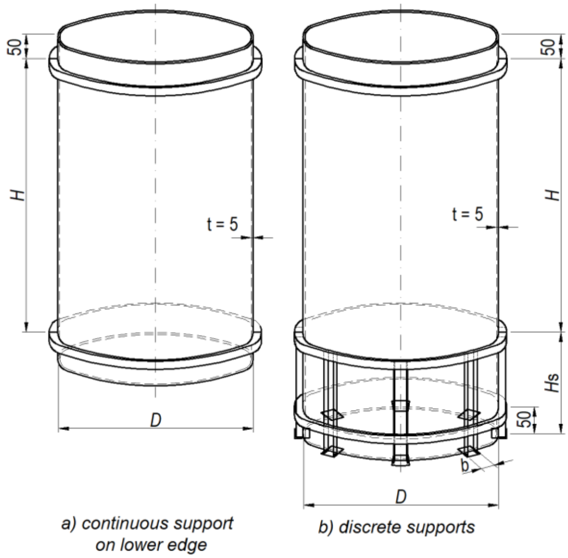


Figure 3. Numerical models – dimensions and way of supporting

b) shell 1 ÷ 8 are supported as follows:

- continuously supporting on the lower edge of the cylindrical shell, see Fig. 3*a*;
- discrete supports below the ring beam with height H_s, see Fig. 3*b*.

c) discrete supported shells have eight supports with dimensions in plan $b \times b$, which are calculated by the expression:

$$b = 0{,}0375D; \tag{11}$$

d) above every discrete support, outside on the shell, two steel plates with a section 8×100 mm are placed, reaching different height. On their upper edge, there is an intermediate ring with a section L100×8 mm.

e) the levels H_s, reached by vertical ribs, i.e. the heights of the ring beams, see fig. 3*b*, are calculated as follows:

- the length of stiffeners $H_{s,I}$ is equal to the height of the ideal position of the intermediate stiffening ring on the shell, calculated by (10);
- using an average value of the distribution of discrete forces F_R from supports $\alpha = 30°$, see Fig. 4. The height $H_{s,30}$ should be determined by the formula:

$$H_{s,30} = \frac{\pi R}{n} \tan\left(90° - \alpha°\right); \tag{12}$$

- the length of the stiffeners $H_{s,L}$ is equal to the distance between the supports. It is calculated by the expression:

$$H_{s,L} = \frac{2\pi R}{n}, \tag{13}$$

where $R = 0{,}5D$ is the radii of bending of the ring beam;
n is the number of the regularly placed supports;

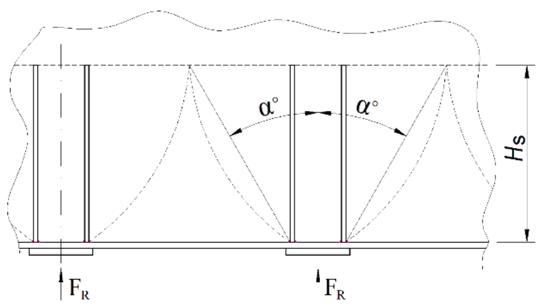


Figure 4. Average angle α of a distribution of the compressive forces on height

f) all shells are with constant thickness $t = 5$ mm;

g) to strengthen the shells in a radial direction, on 50 mm above the lower edge and 50 mm below the upper edge, rings with a section L100×8 mm are placed, welded as shown in Fig. 8;

h) the product stored in the facilities causes a vertical P_{wf} loading, due to the friction between the product and the wall of the cylindrical bodies, see Fig. 5. The research accepted that the vertical loading is constant, with a value $P_{wf} = 0{,}5$ kPa, applied on the internal surface of the shell;

i) the steel used in the models is S235. Its mechanical characteristics are according to standards EN 10025-2:2004 [15], and precisely:

- yield strength $f_y = 235$ MPa;
- modulus of elasticity – $E = 210\,000$ MPa;
- *Poisson's* ratio – $\nu = 0{,}3$.

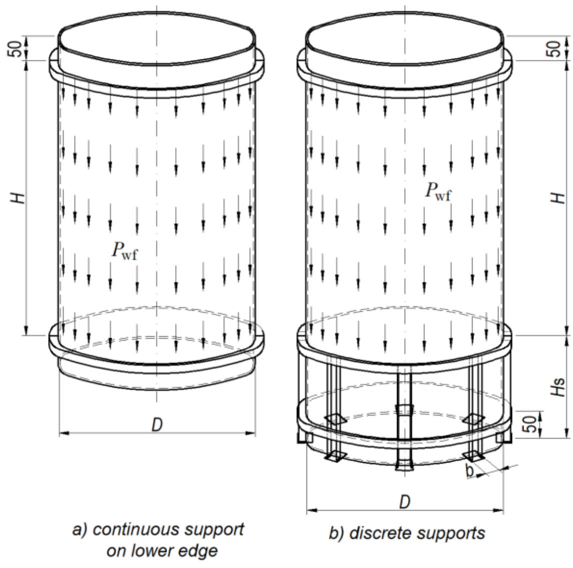


Figure 5. Numerical models – dimensions and loading

The material nonlinearity of the steel is accounted in the research. The relation between stresses σ and strain ε is bi-linear. An idealised diagram of *Prandtl* is used, see Fig. 6. The tangent modulus after the point of yielding has a value E_t = 1 400 MPa.

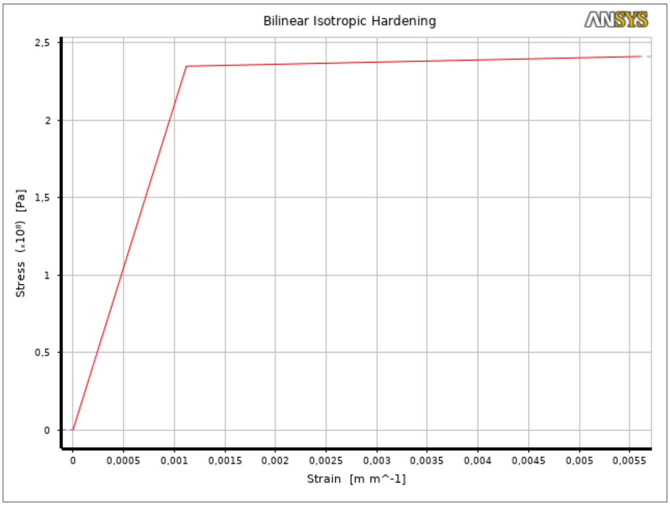


Figure 6. Idealized diagram stress-strain

j) the thin-walled shells are sensitive to the effects of change of the geometry during loading. For that reason, all the models are researched considering the geometrical nonlinearity (GNA).

k) all shells have initial geometrical imperfections. They are symmetrically positioned towards the vertical axis on the entire circumference of the shells. They simulate radial deformations, due to welding operations in horizontal joints. Their shape is shown in Fig. 7.

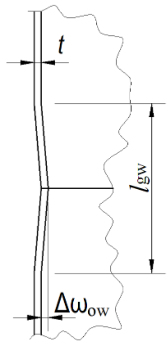


Figure 7. Geometrical imperfection of the shells. Dimensions

l) the shown in the Fig. 7 length l_{wg} of the imperfection is calculated according to the standard EN 1993-1-6, by the formula:

$$l_{gw} = 25t = 25.5 = 125 \text{ mm.} \tag{14}$$

Like the researches of *Doerich* and *Rotter* [16], *Vanlaere* et al. [17], the author accepted that the radial deflection $\Delta\omega_{0w} = t = 5$ mm. It is bigger than the calculated according to EN 1993-1-6:2007:

$$\Delta\omega_{0w} = U_{0w} l_{gw} = 0{,}016.125 = 2 \text{ mm,} \tag{15}$$

where U_{0w} = 0,016 is the dimple tolerance parameter for the fabrication tolerance quality class C.

On this way, the author wants to underline the effect of the geometrical imperfections.

m) the cylindrical shells are constructed by courses with a height of 2 200 mm. It means that the initial geometrical imperfections will be several and will be positioned at a height of 2 200 mm.

The angular section L100×8 and part of the cylindrical shell form an intermediate stiffening ring with a shape as shown in Fig. 8.

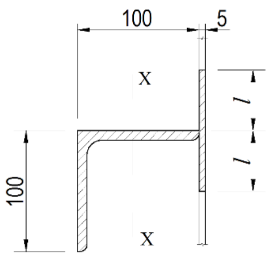


Figure 8. Shape of intermediate stiffening ring

Effective width *l*, in mm, of the steel sheets over and below the joint is calculated according to the standard API 650 [18], by the expression:

$$l \le 13{,}4\sqrt{Dt}\ , \tag{16}$$

where *D* is the diameter of the cylindrical shell, m;

t – thickness of the cylindrical shell, mm.

The cross-section A and moment of inertia I_x about vertical axis *"x-x"* of the formed stiffening rings are as follows:

- shell 1 – A = 18,9 cm^2 and I_x = 302,2 cm^4;
- shell 2 – A = 20,1 cm^2 and I_x = 345 cm^4;
- shell 3 – A = 21,1 cm^2 and I_x = 377 cm^4;
- shell 4 – A = 21,9 cm^2 and I_x = 400,5 cm^4;
- shell 5 – A = 22,6 cm^2 and I_x = 420 cm^4;
- shell 6 – A = 23,2 cm^2 and I_x = 435,5 cm^4;
- shell 7 – A = 23,8 cm^2 and I_x = 450,2 cm^4;
- shell 8 – A = 24,3 cm^2 and I_x = 462 cm^4.

The necessary stiffness of the intermediate stiffening ring is calculated by *Zeybek et al.* [19]. The stiffness ratio χ could be expressed by:

$$\chi = \frac{K_{shell}}{K_{stiffener}} = \frac{Rt\left(AR^2 + I_x n^2\left(n^2-1\right)\right)}{12\sqrt{3}\left(1+\nu\right)^{3/2} AI_x n\left(n^2-1\right)^2}, \tag{17}$$

where K_{shell} is the circumferential stiffness of the shell;
$K_{stiffener}$ – the circumferential stiffness of the circular ring.

The results of the research of *Zeybek et al.* [19] indicate that the ratios below about $\chi <$ 0,2 provide a satisfactorily uniform axial membrane stress distribution above the intermediate ring stiffener, so this limit is recommended for practical design. In his later research *Zeybek et al.* [20] prove that correlations smaller than $\chi <$ 0,2 are sufficient in the rings placed below the ideal position.

For the various shells, the ratio χ, calculated according to formula (17), has the values as follows:

- shell 1 – χ = 0,00566;
- shell 2 – χ = 0,01173;
- shell 3 – χ = 0,01923;
- shell 4 – χ = 0,029;
- shell 5 – χ = 0,0418;
- shell 6 – χ = 0,0585;
- shell 7 – χ = 0,079;
- shell 8 – χ = 0,1047.

The maximum value of the ratio χ = 0,1047 < 0,2, so it could be expected that the stiffness of the intermediate ring will be sufficient to equalize the meridional stresses in the shell above it.

All shells are modelled by 2D quad element shell281. The method of its creation is the "Quadrilaterals". The finite elements are quadrilateral with 8 joints – in the edges and in the middle of the side. The maximum dimension of the elements is 50 mm.

Option "symmetry" is activated in ANSYS. The purpose is to decrease the necessary time for calculation. Only a quarter of the silo is used in the analysis, see Fig. 9.

Option "Buckling Analysis" is activated in the used program ANSYS [14]. With this option it is possible to calculate the reserve of the carrying capacity K of the cylindrical shell before losing stability partially or wholly. When the shells are discretely supported, the reserve

K_b gives the quantitative evaluation of the height H_S of placing of the intermediate stiffening ring, respectively the length of the vertical stiffening elements. Those values K_b are compared with the accounted for a continuously supported on lower edge shell reserves K_a. The idea is that the closer the values for K_a and K_b are, the more effective the stiffening in the base of the discretely supported shells is.

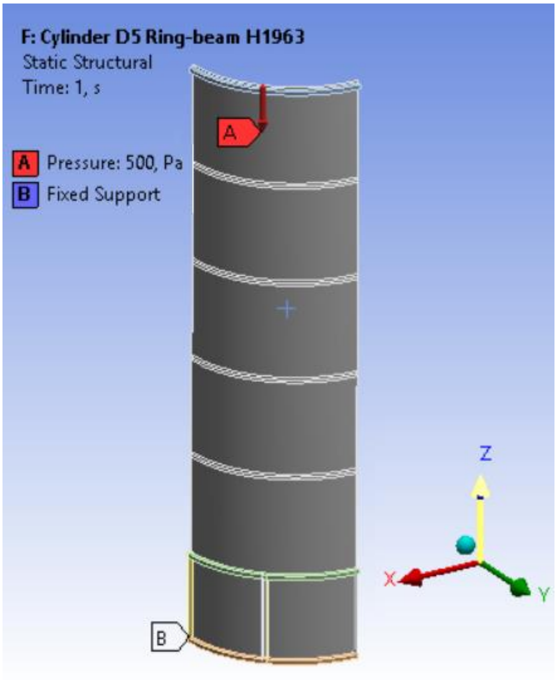


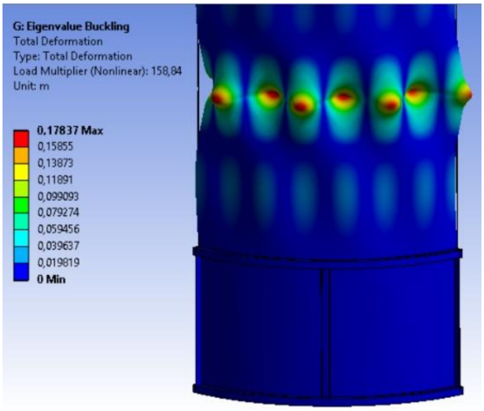
Figure 9. A quarter of the silos used in the numerical analysis

3. Results

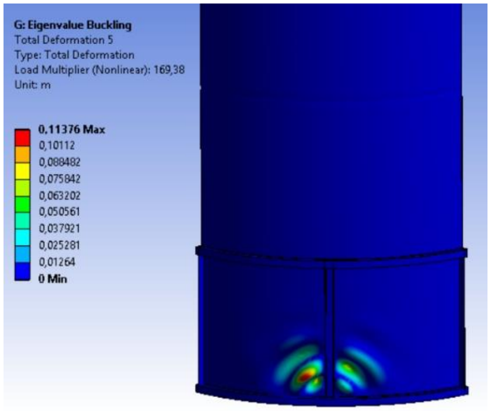
During the research of the cylindrical shells the following forms for local loss of stability are observed:

- due to the compressive forces in the zone of the geometrical imperfection (weld defects), see Fig. 10*a*;
- due to the shearing forces on the sideways of the vertical stiffening elements, see Fig. 10*b*;
- due to the compressive forces above the vertical stiffening elements and the intermediate stiffening ring, see Fig. 10*c*.

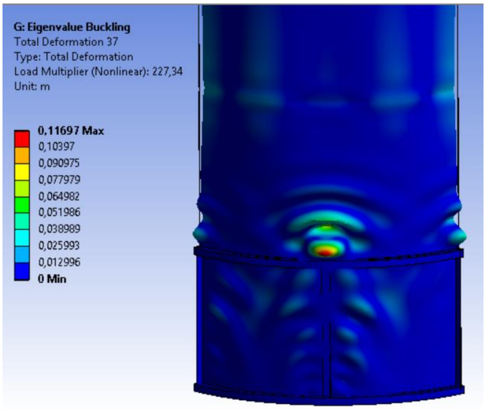
First, the cylindrical shells continuously supported in their lower edge are researched. By approximately 60 buckling modes per shell the values for the reserve of carrying capacity K_a are accounted. This value is correct when the shell buckles into the corresponding geometrical zones, see Fig. 10. These values are shown in Table 1.



a) loss of stability due to pressure forces in the zone of geometrical imperfection



b) loss of stability due to shearing forces to the side of the vertical stiffening elements



c) loss of stability due to compressive forces in the zone above the vertical stiffeners and rings

Figure 10. Shapes of local loss of stability

Table 1. Reserve of carrying capacity K_a when the shells are continuously supported

shell	dimensions, m		Reserve K_a for buckling:	
	D	*H*	in weld imperf.	above stiffeners
shell 1a	1	2,2		6342
shell 2a	2	4,4	1273	1547
shell 3a	3	6,6	468,4	681,3
shell 4a	4	8,8	250,3	382,5
shell 5a	5	11	158,8	242,1
shell 6a	6	13,2	109,5	167,2
shell 7a	7	15,4	80,4	121,7
shell 8a	8	17,6	61,9	93,1

After that, the next research for discretely supported shells having a width of the supports b is conducted. The accounted values of carrying capacity $K_{b,I}$, $K_{b,30}$ and $K_{b,L}$ are shown in Table 2, Table 3 and Table 4.

Table 2. Reserve of carrying capacity $K_{b,I}$ in discretely supported shells and ring beams with height $H_{s,I}$

shell	dimensions, m		b	$H_{s,I}$	Reserve $K_{b,I}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in imperfection	above stiffeners	in imperfection	above stiffeners
shell $1b_I$	1	2,2	0,0375	0,247		6153		3,07
shell $2b_I$	2	4,4	0,075	0,494	1273	1441	0,00	7,36
shell $3b_I$	3	6,6	0,1125	0,74	468,2	616,4	0,04	10,53
shell $4b_I$	4	8,8	0,15	0,987	250,2	335,7	0,04	13,94
shell $5b_I$	5	11	0,1875	1,234	158,8	208,5	0,00	16,12
shell $6b_I$	6	13,2	0,225	1,481	109,2	140,9	0,27	18,67
shell $7b_I$	7	15,4	0,2625	1,728	79,7	101,8	0,88	19,55
shell $8b_I$	8	17,6	0,3	1,975	60,8	77,4	1,81	20,28

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta \leq 1{,}81\%$;
- when the buckling is above the stiffeners – $\Delta \leq 20{,}3\%$.

Table 3. Reserve of carrying capacity $K_{b,30}$ in discretely supported shells and ring beams with height $H_{s,30}$

shell	dimensions, m		b	$H_{s,30}$	Reserve $K_{b,30}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in imperfection	above stiffeners	in imperfection	above stiffeners
shell $1b_{30}$	1	2,2	0,0375	0,34		6153		3,07
shell $2b_{30}$	2	4,4	0,075	0,68	1273	1492	0,00	3,69
shell $3b_{30}$	3	6,6	0,1125	1,02	468,4	645,6	0,00	5,53
shell $4b_{30}$	4	8,8	0,15	1,36	250,3	356,1	0,00	7,41
shell $5b_{30}$	5	11	0,1875	1,7	158,8	222,9	0,00	8,61
shell $6b_{30}$	6	13,2	0,225	2,04	109,5	152,3	0,00	9,78
shell $7b_{30}$	7	15,4	0,2625	2,381	80,3	110,8	0,12	9,84
shell $8b_{30}$	8	17,6	0,3	2,721	61,8	84,8	0,16	9,79

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta \leq 0{,}16\%$;
- when the buckling is above the stiffeners – $\Delta \leq 9{,}8\%$.

Table 4. Reserve of carrying capacity $K_{b,L}$ in discretely supported shells and ring beams with height $H_{s,L}$

shell	dimensions, m		b	$H_{s,L}$	Reserve $K_{b,L}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in imperfection	above stiffeners	in imperfection	above stiffeners
shell $1b_L$	1	2,2	0,0375	0,393		6175		2,70
shell $2b_L$	2	4,4	0,075	0,785	1273	1499	0,00	3,20
shell $3b_L$	3	6,6	0,1125	1,178	468,4	651,7	0,00	4,54
shell $4b_L$	4	8,8	0,15	1,57	250,3	361,7	0,00	5,75
shell $5b_L$	5	11	0,1875	1,963	158,8	228,2	0,00	6,09
shell $6b_L$	6	13,2	0,225	2,356	109,5	156,8	0,00	6,63
shell $7b_L$	7	15,4	0,2625	2,749	80,4	114,4	0,00	6,38
shell $8b_L$	8	17,6	0,3	3,142	61,9	90,2	0,00	3,22

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta = 0\%$;
- when the buckling is above the stiffeners – $\Delta \leq 6{,}6\%$.

In facilities with a bigger diameter *D*, the height H_s of the ring beam is high. When the unloading hopper is attached to the upper edge of the ring beam, a considerable part of the useful volume of the silo will be lost. The possible solution to this problem is the hopper to be joined in a lower position, somewhere at the height of the ring beam. In the current research, the author places it at 200 mm above the lower edge of the cylindrical body. The effect of its joint is simulated through adding another intermediate ring L100×8.

The accounted values of carrying capacity $K_{b,I}$, $K_{b,30}$ and $K_{b,L}$ are shown in Table 5, Table 6 and Table 7.

Table 5. Reserve of carrying capacity $K_{b,I}$ in discretely supported shells, second intermediate ring and ring beams with height $H_{s,I}$

shell	dimensions, m		*b*	$H_{s,I}$	Reserve $K_{b,I}$ for buckling:		Difference with K_a, %	
	D	*H*	m	m	in imperfection	above stiffeners	in imperfection	above stiffeners
shell $1c_I$	1	2,2	0,0375	0,247		6056		4,72
shell $2c_I$	2	4,4	0,075	0,494	1273	1405	0,00	10,11
shell $3c_I$	3	6,6	0,1125	0,74	468,2	610,7	0,04	11,56
shell $4c_I$	4	8,8	0,15	0,987	250,2	336,1	0,04	13,81
shell $5c_I$	5	11	0,1875	1,234	158,8	207,6	0,00	16,62
shell $6c_I$	6	13,2	0,225	1,481	109,2	141,6	0,27	18,08
shell $7c_I$	7	15,4	0,2625	1,728	79,8	101,7	0,75	19,67
shell $8c_I$	8	17,6	0,3	1,975	60,9	77,9	1,64	19,51

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta \leq 1,64\%$;
- when the buckling is above the stiffeners – $\Delta \leq 19,7\%$.

Table 6. Reserve of carrying capacity $K_{b,30}$ in discretely supported shells, second intermediate ring and ring beams with height $H_{s,30}$

shell	dimensions, m		*b*	$H_{s,30}$	Reserve $K_{b,30}$ for buckling:		Difference with K_a, %	
	D	*H*	m	m	in imperfection	above stiffeners	in imperfection	above stiffeners
shell $1c_{30}$	1	2,2	0,0375	0,34		6119		3,64
shell $2c_{30}$	2	4,4	0,075	0,68	1273	1485	0,00	4,18
shell $3c_{30}$	3	6,6	0,1125	1,02	468,4	643,2	0,00	5,92
shell $4c_{30}$	4	8,8	0,15	1,36	250,3	356,7	0,00	7,23
shell $5c_{30}$	5	11	0,1875	1,7	158,8	223,2	0,00	8,47
shell $6c_{30}$	6	13,2	0,225	2,04	109,5	153,2	0,00	9,14
shell $7c_{30}$	7	15,4	0,2625	2,381	80,4	111,1	0,00	9,54
shell $8c_{30}$	8	17,6	0,3	2,721	61,8	85,5	0,16	8,89

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta \leq 0,16\%$;
- when the buckling is above the stiffeners – $\Delta \leq 9,54\%$.

Table 7. Reserve of carrying capacity $K_{b,L}$ in discretely supported shells, second intermediate ring and ring beams with height $H_{s,L}$

shell	dimensions, m		b	$H_{s,L}$	Reserve $K_{b,L}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in imperfection	above stiffeners	in imperfection	above stiffeners
shell $1c_L$	1	2,2	0,0375	0,393		6143		3,24
shell $2c_L$	2	4,4	0,075	0,785	1273	1496	0,00	3,41
shell $3c_L$	3	6,6	0,1125	1,178	468,4	651,6	0,00	4,56
shell $4c_L$	4	8,8	0,15	1,57	250,3	363,3	0,00	5,28
shell $5c_L$	5	11	0,1875	1,963	158,8	229,5	0,00	5,49
shell $6c_L$	6	13,2	0,225	2,356	109,5	158,2	0,00	5,69
shell $7c_L$	7	15,4	0,2625	2,749	80,4	115,4	0,00	5,46
shell $8c_L$	8	17,6	0,3	3,142	61,9	89,5	0,00	4,02

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta = 0\%$;
- when the buckling is above the stiffeners – $\Delta \leq 5{,}69\%$.

Several conclusions can be established from the research of the above-described shells:

a) higher ring beams assure more complete equalization of the meridional normal stresses;

b) even when the ring beams have height H_s, equal to the distance between the supporters, there is no complete equalization of the values of axial normal stresses in the cylindrical shells;

c) in the first mode of loss of stability, the shell is buckling in the zone of imperfection, independently of the way of the girder support and the accepted heights $H_{s,I}$, $H_{s,30}$ or $H_{s,L}$ of the ring beam. In this mode, the reserve of bearing capacity K has the smallest value.

Having in mind the previous conclusion, we could say that the influence of the imperfections is more considerable from the difference in the meridional stresses above the ring beams, whether it is height $H_{s,I}$, $H_{s,30}$ or $H_{s,L}$.

To verify that the accepted in the study geometric imperfections are not excessively large, the characteristic values of the bearing capacity $\sigma_{x,Rk}$ at axial pressure determined according to the methodology of EN 1993-1-6 [21] and numerically, by the models above, were compared. The first approach uses the analytical expressions described in the latest version of the standard. In the second approach, knowing the values of the axial normal stresses $\sigma_{x,Ed}$ in the imperfections and the reserve of the carrying capacity K, through the expression

$$\sigma_{x,Rk,FEA} = K\sigma_{x,Ed} \tag{18}$$

we could calculate the bearing capacity of the shell $\sigma_{x,Rk,FEA}$ before buckling. The values are shown in Table 8 below:

Table 8. Characteristic values of the carrying capacity by normal stresses σx,Rk during axial pressure

shell		EN1993-1-6				FEA		
D	H	λ_x	λ_{xp}	χ_x	$\sigma_{x,Rk,EN}$ kN/cm^2	$K_a = K_{bL}$ in imperf.	$\sigma_{x,Ed}$ kN/cm^2	$\sigma_{x,Rk,FEA}$ kN/cm^2
1	2,2	0,430	0,789	0,877	20,60			
2	4,4	0,608	0,778	0,628	14,76	1273	0,022	28,01
3	6,6	0,745	0,773	0,440	10,33	468,4	0,044	20,61
4	8,8	0,860	0,771	0,305	7,17	250,3	0,066	16,52
5	11	0,962	0,769	0,227	5,33	158,8	0,088	13,97
6	13,2	1,053	0,768	0,178	4,18	109,5	0,11	12,05
7	15,4	1,138	0,768	0,145	3,40	80,4	0,132	10,61
8	17,6	1,216	0,767	0,121	2,85	61,9	0,154	9,53

It can be noted that the differences are significant. This could be due to one or all reasons mentioned below:

a) the geometrical imperfections of the type of weld depressions used in the numerical models, see Fig. 7, are not the most unfavourable;

b) the analytical methodology of EN 1993-1-6 [21] is very conservative. For instance, the current formulas for analytical calculation are obtained empirically, after conducting a lot of laboratory experiments, by which is determined the lowest bearing capacity of the shell before buckling;

c) even if it is very unlikely, it is possible that the numerical models are created incorrectly or/and program ANSYS does not return accurate results.

To verify which of the above reasons is leading, the author has modelled additional geometric imperfections in the meridional direction of the cylindrical shells. They have the shape shown on Fig. 11.

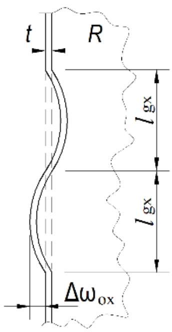


Figure 11. Geometrical imperfections in meridional direction of the shells. Dimensions

Lengths l_{gx} and dimples $\Delta\omega_{0x}$ of imperfections are calculated according to EN 1993-1-6:2007 by the formulae:

$$l_{gx} = 4\sqrt{Rt}\ , \quad (19)$$

$$\Delta\omega_{0x} = U_{0x} l_{gx}\ , \quad (20)$$

where U_{0x} = 0,016 is the dimple tolerance parameter for the fabrication tolerance quality class C.

Geometrical imperfections in meridional direction are modelled in the lower course of the cylindrical shell. They start at a distance l_R above the intermediate ring.

$$l_R = 0{,}1L, \tag{21}$$

where $L = H$ is the distance between stiffening ring, see Fig. 3.

Additional shells are included in this research. Their ring-beam has a height $H_{s,45}$, calculated by formula (22). It is accepted that the average value of the distribution of discrete forces F_R from supports has a value $\alpha = 45°$, see Fig. 4.

$$H_{s,45} = \frac{\pi R}{n}. \tag{22}$$

Again, cylindrical shells, continuously supported in their lower edge, are researched first. The accounted values for the reserve of carrying capacity K_a are shown in Table 9.

Table 9. Reserve of carrying capacity K_a when the shells are continuously supported

shell	dimensions, m		Reserve K_a for buckling:	
	D	H	in weld imperf.	above stiffeners
shell 1a	1	2,2		2863,9
shell 2a	2	4,4	1274	482,95
shell 3a	3	6,6	469,82	175,36
shell 4a	4	8,8	252,97	87,56
shell 5a	5	11	160,4	52,42
shell 6a	6	13,2	111,38	34,24
shell 7a	7	15,4	81,86	25,07
shell 8a	8	17,6	62,702	18,701

There is a little increase in the values of K_a for buckling in circular welds and sharp decrease for loss of stability over the ring.

After that, the research for discretely supported shells having a width of the supports b is conducted. The accounted values of carrying capacity $K_{b,45}$, $K_{b,I}$, $K_{b,30}$ and $K_{b,L}$ are shown in Tables 10 ÷ 13.

Table 10. Reserve of carrying capacity $K_{b,45}$ in discretely supported shells and ring beams with height $H_{s,45}$

shell	dimensions, m		b	$H_{s,I}$	Reserve $K_{b,45}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in weld imperf.	above stiffeners	in imperfection	above stiffeners
shell $1b_{45}$	1	2,2	0,0375	0,196		2864		0,00
shell $2b_{45}$	2	4,4	0,075	0,393	1274	483,6	0,00	-0,13
shell $3b_{45}$	3	6,6	0,1125	0,589	470,22	175,32	-0,09	0,02
shell $4b_{45}$	4	8,8	0,15	0,785	252,14	87,69	0,33	-0,15
shell $5b_{45}$	5	11	0,1875	0,982	160,18	52,6	0,14	-0,34
shell $6b_{45}$	6	13,2	0,225	1,178	111,73	34,56	-0,31	-0,93
shell $7b_{45}$	7	15,4	0,2625	1,374	82,46	25,37	-0,73	-1,18
shell $8b_{45}$	8	17,6	0,3	1,571	61,28	18,775	2,32	-0,39

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta \leq 2{,}3\%$;
- when the buckling is above the stiffeners – $\Delta \leq 1{,}2\%$.

Table 11. Reserve of carrying capacity $K_{b,I}$ in discretely supported shells and ring beams with height $H_{s,I}$

shell	dimensions, m		b	$H_{s,I}$	Reserve $K_{b,I}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in weld imperf.	above stiffeners	in imperfection	above stiffeners
shell $1b_I$	1	2,2	0,0375	0,247		2864,1		-0,01
shell $2b_I$	2	4,4	0,075	0,494	1274	483,9	0,00	-0,20
shell $3b_I$	3	6,6	0,1125	0,74	470,57	175,24	-0,16	0,07
shell $4b_I$	4	8,8	0,15	0,987	252,56	87,61	0,16	-0,06
shell $5b_I$	5	11	0,1875	1,234	160,3	52,612	0,06	-0,36
shell $6b_I$	6	13,2	0,225	1,481	111,54	34,57	-0,14	-0,95
shell $7b_I$	7	15,4	0,2625	1,728	81,62	25,383	0,29	-1,23
shell $8b_I$	8	17,6	0,3	1,975	62,95	18,783	-0,39	-0,44

The differences with the coefficient K_a are as follows:
- when the buckling is in the weld imperfections – $\Delta \leq 0{,}4\%$;
- when the buckling is above the stiffeners – $\Delta \leq 1{,}3\%$.

Table 12. Reserve of carrying capacity $K_{b,30}$ in discretely supported shells and ring beams with height $H_{s,30}$

shell	dimensions, m		b	$H_{s,30}$	Reserve $K_{b,30}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in weld imperf.	above stiffeners	in imperfection	above stiffeners
shell $1b_{30}$	1	2,2	0,0375	0,34		2864,1		-0,01
shell $2b_{30}$	2	4,4	0,075	0,68	1274	480,9	0,00	0,43
shell $3b_{30}$	3	6,6	0,1125	1,02	470,75	174,99	-0,20	0,21
shell $4b_{30}$	4	8,8	0,15	1,36	252,82	87,38	0,06	0,21
shell $5b_{30}$	5	11	0,1875	1,7	160,41	52,52	-0,01	-0,19
shell $6b_{30}$	6	13,2	0,225	2,04	111,38	34,41	0,00	-0,49
shell $7b_{30}$	7	15,4	0,2625	2,381	81,89	25,17	-0,04	-0,40
shell $8b_{30}$	8	17,6	0,3	2,721	62,771	18,706	-0,11	-0,03

The differences with the coefficient K_a are as follows:
- when the buckling is in the weld imperfections – $\Delta \leq 0{,}2\%$;
- when the buckling is above the stiffeners – $\Delta \leq 0{,}5\%$.

Table 13. Reserve of carrying capacity $K_{b,L}$ in discretely supported shells and ring beams with height $H_{s,L}$

shell	dimensions, m		b	$H_{s,L}$	Reserve $K_{b,L}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in weld imperf.	above stiffeners	in imperfection	above stiffeners
shell $1b_L$	1	2,2	0,0375	0,393		2864,1		-0,01
shell $2b_L$	2	4,4	0,075	0,785	1274	480,1	0,00	0,59
shell $3b_L$	3	6,6	0,1125	1,178	470,79	174,88	-0,21	0,27
shell $4b_L$	4	8,8	0,15	1,57	252,86	87,29	0,04	0,31
shell $5b_L$	5	11	0,1875	1,963	160,2	52,46	0,12	-0,08
shell $6b_L$	6	13,2	0,225	2,356	111,4	34,3	-0,02	-0,17
shell $7b_L$	7	15,4	0,2625	2,749	81,9	25,022	-0,05	0,19
shell $8b_L$	8	17,6	0,3	3,142	62,726	18,661	-0,04	0,21

The differences with the coefficient K_a are as follows:
- when the buckling is in the weld imperfections – $\Delta = 0{,}21\%$;
- when the buckling is above the stiffeners – $\Delta \leq 0{,}6\%$.

For joint shell-discharging hopper at a height of 200 mm above the lower edge of the cylindrical shell, the accounted values of reserve of carrying capacity $K_{b,I}$, $K_{b,30}$ and $K_{b,L}$ are shown in Table 14 ÷ 17.

Table 14. Reserve of carrying capacity $K_{b,45}$ in discretely supported shells, second intermediate ring and ring beams with height $H_{s,45}$

shell	dimensions, m		b	$H_{s,I}$	Reserve $K_{b,45}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in weld imperf.	above stiffeners	in imperfection	above stiffeners
shell $1c_{45}$	1	2,2	0,0375	0,196		2860,4		0,12
shell $2c_{45}$	2	4,4	0,075	0,393	1273,9	482,2	0,01	0,16
shell $3c_{45}$	3	6,6	0,1125	0,589	470,17	175,59	-0,07	-0,13
shell $4c_{45}$	4	8,8	0,15	0,785	252,08	87,88	0,35	-0,36
shell $5c_{45}$	5	11	0,1875	0,982	160,15	52,69	0,16	-0,51
shell $6c_{45}$	6	13,2	0,225	1,178	111,72	34,66	-0,30	-1,21
shell $7c_{45}$	7	15,4	0,2625	1,374	80,93	25,48	1,15	-1,61
shell $8c_{45}$	8	17,6	0,3	1,571	61,395	18,84	2,13	-0,74

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta = 2{,}13\%$;
- when the buckling is above the stiffeners – $\Delta \leq 1{,}61\%$.

Table 15. Reserve of carrying capacity $K_{b,I}$ in discretely supported shells, second intermediate ring and ring beams with height $H_{s,I}$

shell	dimensions, m		b	$H_{s,I}$	Reserve $K_{b,I}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in imperfection	above stiffeners	in imperfection	above stiffeners
shell $1c_I$	1	2,2	0,0375	0,247		2861		0,10
shell $2c_I$	2	4,4	0,075	0,494	1274	482,74	0,00	0,04
shell $3c_I$	3	6,6	0,1125	0,74	470,53	175,48	-0,15	-0,07
shell $4c_I$	4	8,8	0,15	0,987	252,52	87,766	0,18	-0,23
shell $5c_I$	5	11	0,1875	1,234	160,27	52,68	0,08	-0,49
shell $6c_I$	6	13,2	0,225	1,481	111,64	34,65	-0,23	-1,18
shell $7c_I$	7	15,4	0,2625	1,728	81,65	25,47	0,26	-1,57
shell $8c_I$	8	17,6	0,3	1,975	62,926	18,827	-0,36	-0,67

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta \leq 0{,}4\%$;
- when the buckling is above the stiffeners – $\Delta \leq 1{,}6\%$.

Table 16. Reserve of carrying capacity $K_{b,30}$ in discretely supported shells, second intermediate ring and ring beams with height $H_{s,30}$

shell	dimensions, m		b	$H_{s,30}$	Reserve $K_{b,30}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in imperfection	above stiffeners	in imperfection	above stiffeners
shell $1c_{30}$	1	2,2	0,0375	0,34		2861,8		0,07
shell $2c_{30}$	2	4,4	0,075	0,68	1273,8	483,72	0,02	-0,16
shell $3c_{30}$	3	6,6	0,1125	1,02	470,74	175,13	-0,20	0,13
shell $4c_{30}$	4	8,8	0,15	1,36	252,8	87,484	0,07	0,09
shell $5c_{30}$	5	11	0,1875	1,7	160,44	52,57	-0,02	-0,29
shell $6c_{30}$	6	13,2	0,225	2,04	111,38	34,472	0,00	-0,67
shell $7c_{30}$	7	15,4	0,2625	2,381	81,9	25,24	-0,05	-0,67
shell $8c_{30}$	8	17,6	0,3	2,721	62,759	18,728	-0,09	-0,14

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta \leq 0{,}2\%$;
- when the buckling is above the stiffeners – $\Delta \leq 0{,}7\%$.

Table 17. Reserve of carrying capacity $K_{b,L}$ in discretely supported shells, second intermediate ring and ring beams with height $H_{s,L}$

shell	dimensions, m		b	$H_{s,L}$	Reserve $K_{b,L}$ for buckling:		Difference with K_a, %	
	D	H	m	m	in imperfection	above stiffeners	in imperfection	above stiffeners
shell 1c$_L$	1	2,2	0,0375	0,393		2860,3		0,13
shell 2c$_L$	2	4,4	0,075	0,785	1274,1	483,16	-0,01	-0,04
shell 3c$_L$	3	6,6	0,1125	1,178	470,79	174,99	-0,21	0,21
shell 4c$_L$	4	8,8	0,15	1,57	252,86	87,372	0,04	0,22
shell 5c$_L$	5	11	0,1875	1,963	160,31	52,5	0,06	-0,15
shell 6c$_L$	6	13,2	0,225	2,356	111,39	34,37	-0,01	-0,38
shell 7c$_L$	7	15,4	0,2625	2,749	81,9	25,1	-0,05	-0,12
shell 8c$_L$	8	17,6	0,3	3,142	62,72	18,676	-0,03	0,13

The differences with the coefficient K_a are as follows:

- when the buckling is in the weld imperfections – $\Delta \leq 0{,}21\%$;
- when the buckling is above the stiffeners – $\Delta \leq 0{,}4\%$.

Two points are interesting there:

a) differences between the reserve of carrying capacity when the buckling is above the ring, i.e. in the zone of meridional geometric imperfections, are reduced significantly;

b) often values of $K_{b,45}$, $K_{b,I}$, $K_{b,30}$ and $K_{b,L}$ are greater than K_a.

Again, a comparison was made with the characteristic values of the carrying capacity $\sigma_{x,Rk}$ at axial pressure, determined by the methodology of EN 1993-1-6:2007, and numerically, using the models above. The results are shown in Table 18.

Table 18. Characteristic values of the carrying capacity by normal stresses $\sigma_{x,Rk}$ during axial pressure

shell		EN1993-1-6				FEA			
D	H	λ_x	λ_{xp}	χ_x	$\sigma_{x,Rk,EN}$ kN/cm^2	$K_a \approx K_{bL}$ in imperf.	l_{gx} m	$\sigma_{x,Ed}$ kN/cm^2	$\sigma_{x,Rk,FEA}$ kN/cm^2
1	2,2	0,430	0,789	0,877	20,60		0,200		
2	4,4	0,608	0,778	0,628	14,76	482,95	0,283	0,038972	18,82
3	6,6	0,745	0,773	0,440	10,33	175,36	0,346	0,060336	10,58
4	8,8	0,860	0,771	0,305	7,17	87,56	0,400	0,0818	7,16
5	11	0,962	0,769	0,227	5,33	52,42	0,447	0,103328	5,42
6	13,2	1,053	0,768	0,178	4,18	34,24	0,490	0,124901	4,28
7	15,4	1,138	0,768	0,145	3,40	25,07	0,529	0,146508	3,67
8	17,6	1,216	0,767	0,121	2,85	18,701	0,566	0,168143	3,14

It can be seen that the meridional imperfections of the type shown in Fig. 11 significantly reduce the bearing capacity of the cylindrical shells accounted by FEA. Hence, the differences between bearing capacities accounted analytically according to the standard EN 1993-1-6:2007, and numerically, are much smaller. It can be concluded that the significant differences observed in Table 8, are due to inappropriately selected initial geometrical imperfections.

4. Conclusions

It is often practice in the design of real steel silos to use stiffening elements in the point of application of concentrated forces. In our case stiffening elements are placed above the discrete supports. Their length, determined in previous researches of the author, where equalization of the meridional normal stresses is wanted, should be in limits $H_{S,30} \leq H_S \leq H_{S,L}$. From the new, alternative by its approach, research done herewith, the following conclusions could be achieved:

a) even when the ring beams have height H_s, equal to the distance between the supports, there is no complete equalization of the values of meridional normal stresses in the cylindrical shells. As a result, the carrying capacity of the perfect cylindrical shell, loaded axially by compressive forces, becomes smaller when buckling is just above the ring beam. When the height of the ring beam is $H_{S,L}$ and the width of the discrete supports is b = 0,0375D, this decrease is of the order of 7%.

b) when buckling is in the zone of modelled imperfections, the resistance of the shell is the smallest.

When the quality of fabrication is class C (normal) it does not depend on the way of supporting and on the accepted height $H_{s,45}$, $H_{s,I}$, $H_{s,30}$ or $H_{s,L}$ of the ring beam. The influence of the above-described imperfections is more considerable from the difference in the meridional stresses above the ring beams whatever the height of the girder is: $H_{s,45}$, $H_{s,I}$, $H_{s,30}$ or $H_{s,L}$. Therefore, the ring beams of the real steel silos class C, could have height $H_s = H_{s,45}$ and it does not decrease their carrying capacity during axial compression with more than 2%.

c) in real steel silos whose shells are different from the ideal cylindrical shells, it is unnecessary to obtain a complete equalization of the values of the axial normal stresses above the ring beams. Its height should be calculated with the purpose to assure equal carrying capacity when buckling above it or in the imperfections.

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НЕОБХОДИМА ВИСОЧИНА НА ПРЪСТЕНОВИДНАТА ГРЕДА В СТОМАНЕНИ СИЛОЗИ НА ОТДЕЛНИ ОПОРИ. РАЗЛИЧЕН ПОДХОД ЗА НЕЙНОТО ОПРЕДЕЛЯНЕ

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Ключови думи: *стоманен силоз, меридианно напрежение, критична височина, вертикален вкоравител, загуба на устойчивост, материална и геометрична нелинейност, несъвършенства*

РЕЗЮМЕ

Стоманените силози са интересни, комплексни съоръжения. За да се осигури цялостното им изпразване по гравитачен път, често те са поставени на носеща прътова конструкция над земята. В мястото на снаждане на тънкостенната черупка и носещите прътови елементи стойностите на напреженията са много големи. Това може да доведе до местна загуба на устойчивост в черупката. За да се предотврати нейното изкорубване, много проектанти поставят закоравяващи елементи над опорите. Те са част от пръстеновидната греда под цилиндричното тяло. Въпросът е колко високи да бъдат въпросните вкоравители, респективно гредата. Разумният подход е те да стигат до там, докъдето има изравняване на стойностите на меридианните нормални напрежения над опорите и в средата между тях. Но къде е разположено това ниво? Много изследователи са работили върху стойностите и начина на разпространение на меридианните нормални напрежения над опорите на цилиндричните черупки. В резултат на усилията им са определени критичната височина H_{cr} на черупката и идеалното положение H_I на междинния закоравяващ пръстен. Тези височини доста се различават помежду си, освен това са определени при гладки стоманени черупки, без вертикални закоравяващи ребра в тях, а те съществено променят картината. Направените изследвания показват, че те трябва да достигат до ниво, намиращо се между H_{cr} и H_I. При отчитане на нелинейното поведение на стоманата ефектите от промяна в геометрията по време на натоварване, породените от заваръчните операции несъвършенства и вертикалните вкоравители е определена необходимата височина на вкоравителите по нов, различен начин.

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