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FABRICATION COST CALCULATION AND OPTIMUM DESIGN OF WELDED STEEL SILOS

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ABSTRACT

It is shown, how to incorporate the cost calculation into the design procedure to find the optimal, most economic structural version. The design procedure and cost calculation of welded steel silos are treated. In a numerical example the material and fabrication costs are calculated for roof, cylindrical bin, ringbeam, hopper and supporting columns as a function of the height/radius = *H/R* ratio for a given constant storage capacity. It is found that the self weight and the cost of the bin increases, the self weight and cost of the other parts decreases when the *H/R* ratio increases. The minimum weight and the minimum cost is reached at the practical upper limit of *H/R*.

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1. INTRODUCTION

Silos are used for many engineering purposes. Several structural versions exist for ground or elevated silos, for storage and transportation of different materials such as coal, sand, cement, grains (wheat, peas etc.). Silos may be constructed from steel, aluminium or reinforced concrete. Steel silos can be welded or bolted. Corrugated plate elements are also used (Martens 1988).

A transit silo constructed from steel plate elements is investigated in the present paper (Fig.1). This type consists of the following main structural parts: roof, circular cylindrical bin, transition ringbeam, conical hopper and supporting columns. Our aim is to show the design procedure and fabrication cost calculations of these parts and to give designers aspects for the minimum weight and cost design.

Many articles can be found in the literature dealing with the stress and strength analysis of such silos (e.g. Gaylord 1984, Trahair et al. 1983), but the minimum cost design is not treated till now.

The main structural dimensions of a silo shown in Fig.1 are the height H and radius R of the cylindrical bin welded from horizontal courses of thin plates. For a given stored material, storage capacity of the bin and hopper, for a H/R ratio and H the R value can be calculated and the structural dimensions of the silo parts can be designed on the basis of stress and buckling strength constraints.

The question of the optimum design is to determine the optimal H/R ratio for which the self weight of the structure and the cost is minimal. To illustrate the behaviour of theses objective functions a numerical example is selected and self weight and cost calculations are performed for various H/R ratios.

The slope angle of the hopper is determined by the friction angle of stored material, so it is not varied. The number of columns can be varied, but the maximal number is determined by the required minimal distance between columns to allow the emptying into lorries, and the minimal number is 6, required for the stability against horizontal action of wind.

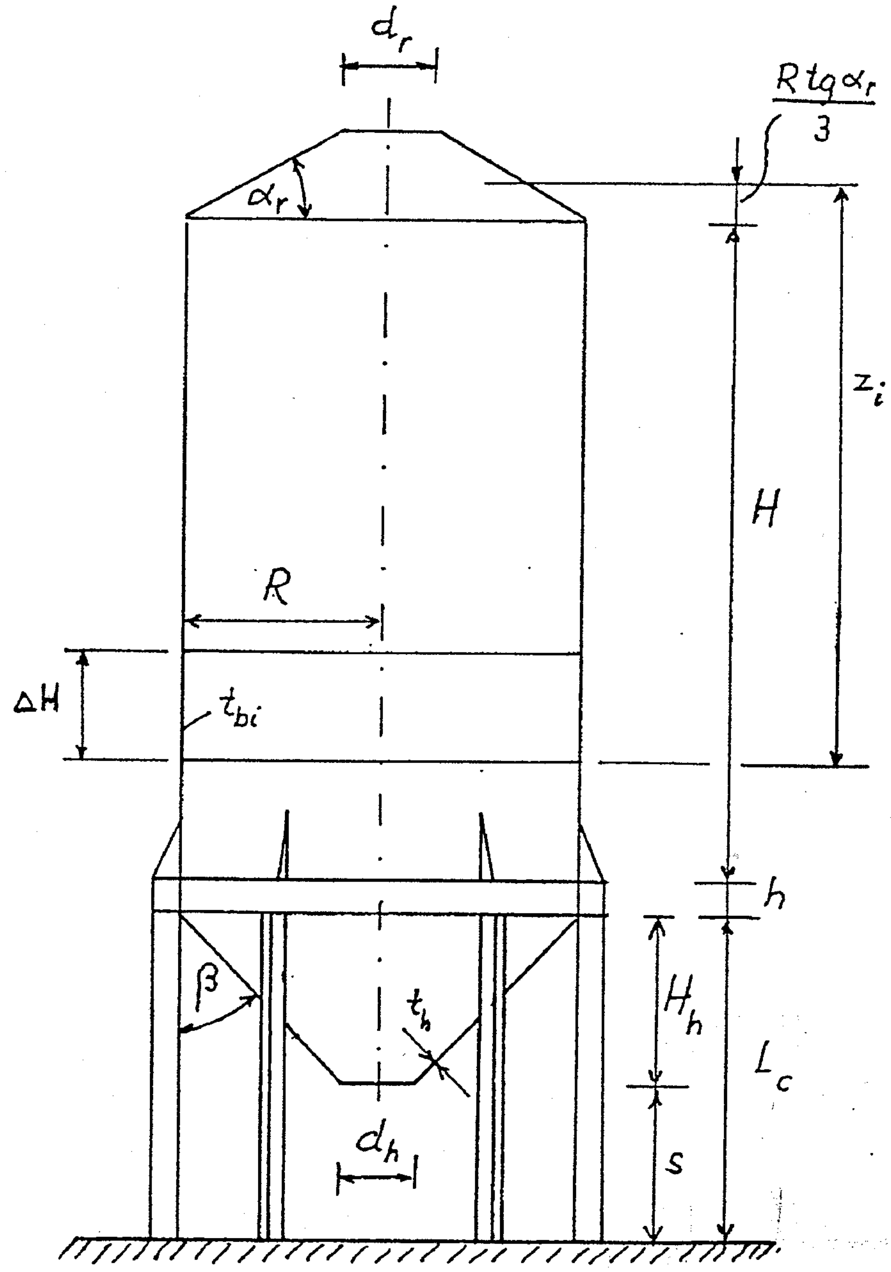

Fig. 1. Main dimensions of a welded steel silo

The design procedure and cost calculations are treated considering the practical ranges of dimensions regarding the numerical example. It should be noted that it is impossible to give general optimum design rules valid for all types of silos and all ranges of dimensions. For other types and dimensions a similar analysis and optimum design should be carried out.

2. THE COST FUNCTION

The objective function is defined as the sum of material and fabrication costs

$$K = K_m + K_f = k_m \rho_s V + k_f \sum_i T_i \tag{1}$$

where $k_m(\$/\mathrm{kg})$ and $k_f(\$/\min)$ are the material and fabrication cost factors, $\rho_s(\mathrm{kg/m^3})$ is the steel density, $V(\mathrm{m}^3)$ is the volume of structure, $T_i(\min)$ are times required for fabrication. Eq.(1) can be written in other form as follows

$$\frac{K}{k_m} = \rho_s V + \frac{k_f}{k_m} \sum_i T_i \tag{2}$$

As described previously (Farkas and Jármai 1994 a,b) the fabrication procedure can be divided in three main parts as proposed by Pahl and Beelich (1982):

a) Time of preparation, assembly and tacking

$$T_1 = C_1 \delta \sqrt{\kappa G};\ G = \rho_s V;\ C_1 = 1.0\ \min/\mathrm{kg}^{0.5} \tag{3}$$

where δ is the difficulty factor considering the structural form of a welded part (planar or spatial, simple plate elements or rolled sections, tubes etc.) κ is the number of elements to be assembled.

b) Time of welding

$$T_2 = \sum_i C_{2i} a_{wi}^n L_{wi} \tag{4}$$

where a_w (mm) is the weld dimension, L_w (mm) is the weld length, C_2 ($\min/\mathrm{mm}^2$) and n are constants characterizing the applied welding technology.

c) Time of changes of electrode, deslagging and chipping

$$T_3 = \sum_i C_{3i} a_{wi}^n L_{wi} \tag{5}$$

Ott and Hubka (1985) proposed that $C_3 = (0.2-0.4)C_2$, in average $C_3 = 0.3C_2$, so

$$T_2 + T_3 = 1.3\sum_i C_{2i} a_{wi}^n L_{wi} \quad (6)$$

Table 1. Welding times T_2 (min/mm) in function of weld size a_w (mm) for longitudinal welds, downhand position

Weld type	Welding method	a_w (mm)	$10^3 T_2 = 10^3 C_2 a_w^n$
fillet	SMAW	2-5 5-15	$4.0\ a_w$ $0.8 a_w^2$
fillet	GMAW-C	2-5 5-15	$1.70\ a_w$ $0.34 a_w^2$
fillet	SAW	2-5 5-15	$1.190\ a_w$ $0.238 a_w^2$
1/2 V butt	SMAW	4-15	$0.600 a_w^2$
1/2 V butt	GMAW-C	4-15	$0.257 a_w^2$
1/2 V butt	SAW	4-15	$0.181 a_w^2$
I-butt double-sided	GMAW-C	2-8	$2(0.567+0.417 a_w)$

The values of $C_2 a_w^n$ are given on the basis of the COSTCOMP (1990) software worked out by the Welding Institute of Netherlands (Bodt 1990) for some weld types and welding methods in Table 1.

It should be noted that the above described calculation method is an approximate one, since many parameters are neglected in it. In spite of this the method is suitable for comparisons and it can give designers and manufacturers good aspects for the selection of the best structural version. We have used this calculation method for the evaluation of some welded structural parts as box beams (Farkas 1991) or stiffened and cellular plates (Farkas 1992, Farkas and Jármai 1994 a,b). In the present study this method is used for structural parts of a welded silo.

To give internationally usable results, the k_f / k_m ratio is varied in a wide range. For steel

$$k_m = 0.5 - 1.2\$/\mathrm{kg},$$

for fabrication including overheads

$$k_f = 0 - 45\$ / \text{manhour} = 0 - 0.75\$ / \text{min},$$

thus, the ratio may vary in the range of 0-1.5, the value of $k_f / k_m = 0$ corresponds to the minimum weight design.

3. DESIGN AND COST CALCULATION OF STRUCTURAL PARTS

3.1 ROOF

In our numerical example treated in Section 4 the radius R is varied in the range of R= 2.9 - 4.25 m. For these radii a relatively simple roof structure can be used consisting of radial rafters of rolled I-section and trapezoidal plate segments welded to rafters and to inner and outer ring by fillet welds.

The snow load $p_s (\text{kN/m}^2)$ acts on roof and the plate elements should be checked for bending. The number of rafters or plate elements can be determined from the restriction that the maximal plate width should not exceed 2.3 m to be transportable. Thus, the number of rafters is $n_r = 2R\pi / 2.3$ rounded up to the next even number.

In an approximate calculation a plate strip can be designed for bending as a simply supported beam with a span length $L_p = 2R\pi / n_r$ and of thickness t_r:

$$\frac{M_{max}}{W_x} = \frac{6\gamma p_s L_p^2}{8t_r^2} \le f_y \qquad (7)$$

where $\gamma = 1.5$ is the safety factor, $f_y = 355\,\text{MPa}$ is the steel yield stress (for all parts of silo this yield stress is used).Calculating with $p_s = 1\,\text{kN/m}^2 = 10^{-3}\,\text{N/mm}^2$ the required plate thickness from Eq.(7) is

$$t_r = L_p \sqrt{\frac{3\gamma p_s}{4 f_y}} = L_p \sqrt{\frac{0.75 * 1.5 * 10^{-3}}{355}} = 1.78 * 10^{-3} L_p \qquad (8)$$

With the maximal value of $L_p = 2300$ mm, $t_r = 4$ mm. This thickness is used for all radii.

A rafter can be calculated approximately as a simply supported beam having span length of $L_r = R - d_r/2$, where $d_r = 800$mm is the diameter of inner ring. The required section modulus is

$$W_x = \gamma M_{max} / f_y = \gamma p_s L_p L_r^2 / (8 f_y) \qquad (9)$$

where $\gamma = 1.5$, $f_y = 355$MPa, $p_s = 10^{-3}$MPa, $L_p = 2300$mm.

The self weight of n_r plate elements can be calculated taking an element as a triangle with a width of $2R\pi/n_r$ and length $(R - d_r/2)/\cos\alpha_r$ ($\alpha_r = 30°$ is the roof slope angle):

$$G_p = \rho_s t_r n_r \pi R (R - d_r/2) / 0.866 \qquad (10)$$

where $\rho_s = 7850 \text{kg/m}^3$ is the density of steel.

The self weight of n_r rafters with length $L_r / \cos 30°$ and cross-section area $A\,(\text{mm}^2)$ is

$$G_{raf} = \rho_s n_r A L_r / \cos 30° \qquad (11)$$

The total weight of a roof structure is

$$G_r = G_p + G_{raf} \qquad (12)$$

The fabrication time of the roof, according to Eqs (3) and (6) is

$$\sum T_i = C_1 \delta_r \sqrt{\kappa_r G_r} + 1.3 \sum C_{2ri} a_{wri}^n L_{wri} \qquad (13)$$

where $C_1 = 1.0\,\text{min/kg}^{0.5}$, $\delta_r = 3$, $\kappa_r = 2n_r$; according to COSTCOMP data, for $a_{wr} = 3\text{mm}$ and for GMAW-C welding method $C_{2r} = 1.7x10^{-3}\,\text{min/mm}^2$, $n = 1$. The weld length is the sum of perimeters of the inner and outer ring as well as the length of longitudinal welds, approximately

$$L_{wr} = d_r\pi + 2R\pi + 2L_r n_r / \cos 30^o \tag{14}$$

3.2 BIN

The circular cylindrical bin is loaded by the horizontal pressure of the stored material calculated with the Janssen's formula

$$p_h = p_o\zeta;\quad p_o = \frac{\rho R}{2\mu};\quad \zeta = 1 - e^{-z/z_o};\quad z_o = \frac{R}{2\mu k} \tag{15}$$

where ρ is the density of stored material, μ is the friction coefficient of the material on the wall, z is the depth of stored material above the investigated section, k is the pressure coefficient. Note that the distance $Rtg\alpha_r/3$ in Fig. 1 is the possible height of the stored materrial in roof, this distance is in the following calculation neglected. With these coefficients the frictional stress on wall is

$$q = \mu p_h \tag{16}$$

and the vertical pressure in bin is

$$p_v = p_h / k \tag{17}$$

The circumferential membrane force in bin is

$$n_{\varphi b} = p_h R \tag{18}$$

and the meridional membrane force is

$$n_{zb} = \int_o^z p_v dz = \mu p_o z_o \left(\frac{z}{z_o} - \zeta \right) \tag{19}$$

The required bin thickness t_b can be calculated from the stress constraint

$$\sigma = \gamma_{red} n_{red} / t_b \le f_y; \quad n_{red} = \sqrt{n_{\varphi b}^2 + n_{\varphi b}^2 + \left| n_{\varphi b} n_{zb} \right|}$$

$$t_{b\max} = \frac{\gamma_{red} p_\circ R}{f_y} \sqrt{\zeta_H^2 + \frac{1}{4k^2}\left(\frac{H}{z_\circ} - \zeta_H\right)^2 + \frac{\zeta_H}{2k}\left(\frac{H}{z_\circ} - \zeta_H\right)} \tag{20}$$

where $\zeta_H = 1 - e^{-H/z_\circ}$ and γ_{red} is the safety factor considering the approximate self weight $(0.1*1.35)$, the dynamic effects of filling and emptying (1.2*1.5) as well as the sudden change of temperature (0.2)

$$\gamma_{red} = 0.1*1.35 + 1.2*1.5 + 0.2 = 2.135 \tag{21}$$

The bin thickness is limited to $t_{b\min} = 4\text{mm}$ by the fabrication requirements. When the t_b calculated from Eq. (20) exceeds 4 mm, the thickness can be decreased step by step to 4 mm, since the bin is welded from shell courses. The width of courses is determined by the available plate width (e.g. 1500 mm).

The whole bin should be checked for local buckling due to wind acting on the empty silo. For a cylindrical shell with variable thickness the German standard for steel structures DIN 18800 Part 4 (1990) gives a complicated method. Instead of this method the API 650 formula can be used (Gaylord 1984 p. 270)

$$\frac{H}{t_b} \le 7200\left(\frac{600 t_b}{R}\right)^{2/3} \tag{22}$$

where t_b is the average bin thickness.

The constraint on local buckling of bin courses due to the vertical pressure according to DIN 18800 Part 4 is expressed by

$$\sigma_z = 1.1 n_{zb} / t_b \le \kappa_1 f_y / \gamma_M \tag{23}$$

where $\gamma_M = 1.$ is a safety factor, κ_1 is the buckling coefficient

$$\kappa_1 = 1 \qquad \text{for } \bar{\lambda}_s \le 0.4$$

$$\kappa_1 = 1.274 - 0.686\bar{\lambda}_s \qquad \text{for } 0.4 < \bar{\lambda}_s < 1.2 \tag{24}$$

$$\kappa_1 = 0.65 / \bar{\lambda}_s^{2} \qquad \text{for } 1.2 \le \bar{\lambda}_s$$

$$\bar{\lambda}_s = \sqrt{f_y / \sigma_{z.\mathrm{id}}}; \quad \sigma_{z.\mathrm{id}} = 0.605 E t_b / R$$

With the value of the elastic modulus $E = 2.1 * 10^5 \mathrm{MPa}$ and for $f_y = 355\mathrm{MPa}$ it is

$$\bar{\lambda}_s = \sqrt{\frac{R}{t_b}} \sqrt{\frac{f_y}{0.605E}} = \frac{1}{18.9179} \sqrt{\frac{R}{t_b}} \tag{25}$$

In our numerical example treated in Section 4, the required maximal bin thickness is $t_b = 4\mathrm{mm}$, so all bin plate elements have the same thickness, 4 mm. Thus, the self weight of the bin is

$$G_b = 2R\pi H t_b \tag{26}$$

The difficulty factor in Eq. (3) is taken as $\delta_b = 4,$ since the bin is a spatial structure and needs a special erection method. The bin courses are welded from plate units having dimensions of 6000*1500 mm with horizontal and vertical double-sided I-welds with GMAW-C welding method. Number of courses is $n_{co} = H^{(\mathrm{m})} / 1.5,$ the length of circumferential welds is $2R\pi n_{co},$ number of vertical welds is $n_v = 2R^{(\mathrm{m})}\pi / 6.0$ rounded up to the next integer value, length of a vertical weld is 1.5 m. Number of assembled elements is $\kappa_b = 2R\pi n_{co} / 6.$

Thus, the total fabrication time for the bin is given by

$$\sum T_{bi} = \delta_b \sqrt{\kappa_b G_b} + 1.3 C_{2b} a_{wb} L_{wb} \tag{27}$$

where $a_{wb} = 4\mathrm{mm},$ $L_{wb} = 2R\pi n_{co} + n_v H.$ According to COSTCOMP (Table 1.)

$C_{2b} = 1.1 * 10^{-3}\ \mathrm{min/\,mm^2}$ for I-welds welded from both sides.

3.3 HOPPER

The load component perpendicular to the conical hopper wall can be calculated according to DIN 1055 Part 6 (1987)

$$p' = \left(p_v c_b \sin^2\beta + p_h \cos^2\beta\right)\left[1 + \frac{\sin 2(90-\beta)}{4\mu}\right] \tag{28}$$

where $c_b = 1.5$, β is the slope angle of hopper (Fig.1).

The hoop (circumferential) membrane tension force is

$$n_h = p' R / \cos\beta \tag{29}$$

The meridional tension is given by Gaylord (1984, p. 271)

$$n_m = \frac{p_v R}{2\cos\beta} + \frac{Q_m}{2R\pi\cos\beta} \tag{30}$$

where Q_m is the weight of stored material below the junction of hopper and ringbeam

$$Q_m = \frac{\rho\pi}{24}\left(8R^3 - d_h^3\right)\mathrm{ctg}\beta$$

d_h is the diameter of the bottom ring of the hopper.

The required hopper wall thickness t_h can be obtained from the stress constraint

$$\sigma_h = \frac{\gamma_h n_{\mathrm{red}.h}}{t_h} \le f_y; \quad n_{\mathrm{red}.h} = \sqrt{n_h^2 + n_m^2 - n_h n_m} \tag{31}$$

$\gamma_h = 1.5.$

The self weight of the hopper is expressed by

$$G_h = \frac{\rho_s \pi}{\sin\beta}\left[R^2 - \left(d_h/2\right)^2\right] t_h \tag{32}$$

The number of plate elements is the same as in the case of roof $\kappa_h = 2R\pi/2.3$. The difficulty factor is taken as $\delta_h = 4$, since the hopper is fabricated from shell elements. The hopper thickness is for all silos treated in Section 4 $t_h = 4\text{mm}$, so for I-welds between shell elements $C_{2h1} = 1.1 * 10^{-3}$ min/ mm^2. The length of a weld is $L_{wh1} = \left(R - d_h/2\right)/\sin\beta$, the total length is $L_{wh1} n_h$. The hopper is welded to the ringbeam by two circumferential fillet welds of size $a_w = 4\text{mm}$, for which $C_{2h2} = 1.7 * 10^{-3}$ min/ mm^2, and the length of welds is $L_{wh2} = 4R\pi$. Thus, the total fabrication time for hopper is given by

$$\sum T_{hi} = \delta_h \sqrt{\kappa_h G_h} + 1.3 * 4\left(C_{2h1} L_{wh1} + C_{2h2} L_{wh2}\right) \tag{33}$$

3.4 COLUMNS

Columns are loaded by self weight of roof, bin, hopper and ringbeam as well as by snow, wind and weight of the stored material. In the case of more variable actions Eurocode 3 prescribes two combinations as follows: a) considering only the most unfavourable variable action (Q) adding to the permanent actions (G): $\sum_j G_j + Q_1$; b) considering all unfavourable variable actions multiplied by 0.9: $\sum_j G_j + 0.9 \sum_j Q_i$. The safety factor is for $\gamma = 1.35$ for G and $\gamma = 1.5$ for Q.

The number of columns (n_{col}), as mentioned in the introduction, is limited by the transit function of the silo and the minimal number of columns is 6.

The snow load, according to Section 3.1 is $Q_{snow} = \pi R^2 p_s$. The weight of the stored material is $Q_{stor} = \rho V_{stor}$, the volume is given by

$$V_{stor} = \pi R^2 H + \pi H_h \left(\frac{d_h}{2}\right)^2 + \frac{\pi}{3} H_h \left(R - \frac{d_h}{2}\right)^2 ; \quad H_h = \left(R - \frac{d_h}{2}\right) \mathrm{ctg}\beta \qquad (34)$$

In our numerical example the weight of stored material gives the leading combination, so the effect of snow and wind can be neglected.

The compressive force of a column is

$$N_c = \frac{1}{n_{col}} \left(1.35 \sum_j G_j + 1.5 Q_{stor} \right) \qquad (35)$$

The overall buckling constraint is expressed by

$$N_c / A_c \le \chi f_y \qquad (36)$$

In the case of columns of square hollow section of width b_c and thickness t_c, $A_c = 4 b_c t_c$. The length of a column with pinned ends is $L_c = H_h + 2\mathrm{m}$.

The buckling coefficient can be calculated on the basis of the JRA (Japanese Road Association) column curve which gives values similar to Eurocode 3 curve "b":

$$\begin{aligned} \chi &= 1 && \text{for} \quad \bar{\lambda}_c \le 0.2 \\ \chi &= 1.109 - 0.545 \bar{\lambda}_c && \text{for} \quad 0.2 < \bar{\lambda}_c < 1 \\ \chi &= \frac{1}{0.773 + \bar{\lambda}_c^2} && \text{for} \quad \bar{\lambda}_c \ge 1 \end{aligned} \qquad (37)$$

The reduced slenderness is defined by

$$\bar{\lambda}_c = \frac{L_c \sqrt{6}}{b_c \lambda_E} \quad ; \qquad \lambda_E = \pi \sqrt{\frac{E}{f_y}} = 76.7$$

The local buckling constraint according to Eurocode 3 is

$$\delta_c = b_c / t_c \le \delta_{cL} = 42 \varepsilon = 34; \quad \varepsilon = \sqrt{235 / f_y}, \quad (f_y \text{ in MPa}) \qquad (38)$$

Treating Eq. (38) as active, the cross-sectional area can be expressed as

$$A_c = 4b_c t_c = 4b_c^2 / \delta_{cL} \tag{39}$$

Assuming that $0.2 < \bar{\lambda}_C < 1$, Eq.(36) can be written as

$$\frac{N_c}{f_y} = \chi A_c = \left(1.109 - 0.545 \frac{L_c \sqrt{6}}{b_c \lambda_E}\right) \frac{4b_c^2}{\delta_{cL}} \tag{40}$$

which is a quadratic equation for b_c..

The self weight of columns is

$$G_{col} = \rho_s n_{col} A_c L_c \tag{41}$$

Since the sections are not welded, the fabrication cost of columns may be neglected.

It should be noted that, for the spatial stability of a silo, wind braces are needed between columns. The cost of these braces is neglected.

3.5 RINGBEAM

The loads acting on the transition ringbeam cause compression, bending, shear and torsion. Since the open section beams have very small torsional stiffness, it is advantageous to use a welded box ringbeam (Farkas 1985).

The dimensions of the ringbeam can be calculated using the constraints on stress and local buckling of component plates. To construct a suitable connection between columns and ringbeam the distance between the two webs of the box beam should be equal to column width b_c, thus, the flange width will be $b_r = b_c + 40\text{mm}$. Note that this is the reason why we treat the design of the ringbeam after the design of columns.

Since the horizontal component of the tensile force acting from the hopper causes compression in the ringbeam, it is necessary to use in the local buckling constraint of webs the same limiting plate slenderness $\delta_L = 42\varepsilon = 34$ as for flanges. Thus, the active local buckling constraints are as follows:

$$\text{for webs} \quad 2h_r / t_{wr} \le \delta_L \tag{42}$$

$$\text{for flanges} \quad b_r / t_{rb} \le \delta_L \tag{43}$$

The vertical component of the tensile membrane force acting from the hopper is

$$y_r = \frac{\gamma Q_{stor}}{2R\pi} \tag{44}$$

This load causes bending and shear in vertical plane of the ringbeam

$$M_{r.\max} = y_r L_r^2 / 12; \quad L_r = 2R\pi / n_{col} \tag{45}$$

The horizontal component

$$x_r = y_r \mathrm{tg}\beta \tag{46}$$

causes compression

$$n_r = x_r R \tag{47}$$

The cross-sectional area of the ringbeam is

$$A_r = h_r t_{wr} + 2b_r t_{rb} = 2\left(h_r^2 + b_r^2\right) / \delta_L \tag{48}$$

and the section modulus is expressed by

$$W_{xr} = h_r^2 t_{wr} / 6 + h_r b_r t_{rb} = \left(h_r^3 + 3h_r b_r^2\right) / \left(3\delta_L\right) \tag{49}$$

In Eq. (48) and (49) the only unknown is h_r which can be calculated from the stress constraint

$$\frac{M_{r.\max}}{W_{xr}} + \frac{n_r}{A_r} \le f_y \tag{50}$$

Unfortunately, a closed formula cannot be derived from Eq. (50).

The self weight of the ringbeam is

$$G_{rb} = \rho_s 2\pi\left(R + b_c / 2\right) A_r \qquad (51)$$

The ringbeam is welded using four fillet welds of dimension $a_w = 4\text{mm}$. The difficulty factor is $\delta_{rb} = 4$ since a curved beam should be fabricated.

$\kappa_{rb} = 4$, $C_{2rb} = 1.7 * 10^{-3}\ \text{min/mm}^2$. The whole weld length is

$$L_{wrb} = 4R\pi + 4\left(R + b_c\right) \qquad (52)$$

The total fabrication time of a ringbeam is

$$\sum T_{rbi} = \delta_{rb}\sqrt{\kappa_{rb} G_{rb}} + 1.3 * 4 C_{2rb} L_{wrb} \qquad (53)$$

4. NUMERICAL EXAMPLE

We select the extreme heights $H = 7.5$ and 18.0 m for integer numbers of courses n_{co}.. For a given constant storage capacity $V_{stor} = 500\text{m}^3$ the required radius R can be calculated from Eq. (34). The dimensions, self weights and costs are determined for four silos, i.e. for H = 7.5, 12.0, 15.0 and 18.0 m for storage of cement with a density of $\rho = 1600\text{kg/m}^3$. For cement it is $\mu = 0.4$ and $k = 0.6$. The density of steel is $\rho_s = 7850\text{kg/m}^3$. The data for the cost calculation are summarized in Table 2.

The results of the calculations are summarized in Table 3. Fig.2. and 3. show the results as a function of *H/R* for $k_f / k_m = 0$ and 1.5.

Table 2. Calculation of fabrication times T_1 and $T_2 + T_3$ according to Eq.(3) and (6) for structural parts of a welded silo. For all parts C_1=1.0 min/kg$^{0.5}$, n=1

Structure	roof	bin	ringbeam	hopper	
δ	3	4	4	4	
κ	$4R\pi/2300$	$n_{co}n_v$	4	$2R\pi/2300$	
G	Eq. (12)	Eq. (26)	Eq. (51)	Eq. (32)	
welds a_w (mm)	fillet, 3	I-weld, 4	fillet, 3	I-weld,4	fillet,4
C_2(min/ mm^2)	$1.7*10^{-3}$	$1.1*10^{-3}$	$1.7*10^{-3}$	$1.1*10^{-3}$	$1.7*10^{-3}$
L_w	Eq.(14)	Eq.(27)	Eq.(52)	$\frac{R-d_h/2}{\sin\beta}$	$4R\pi$

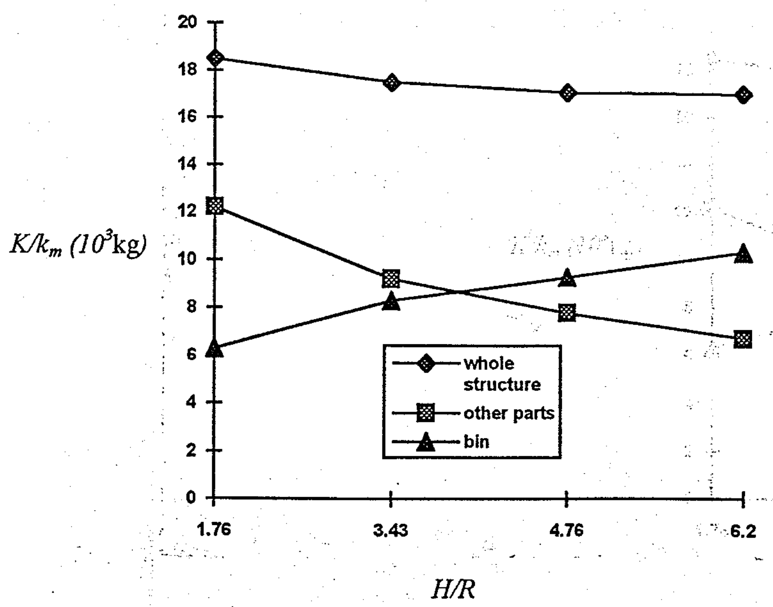

Fig. 2. Cost of silo parts in the function of H/R for k_f/k_m=0.

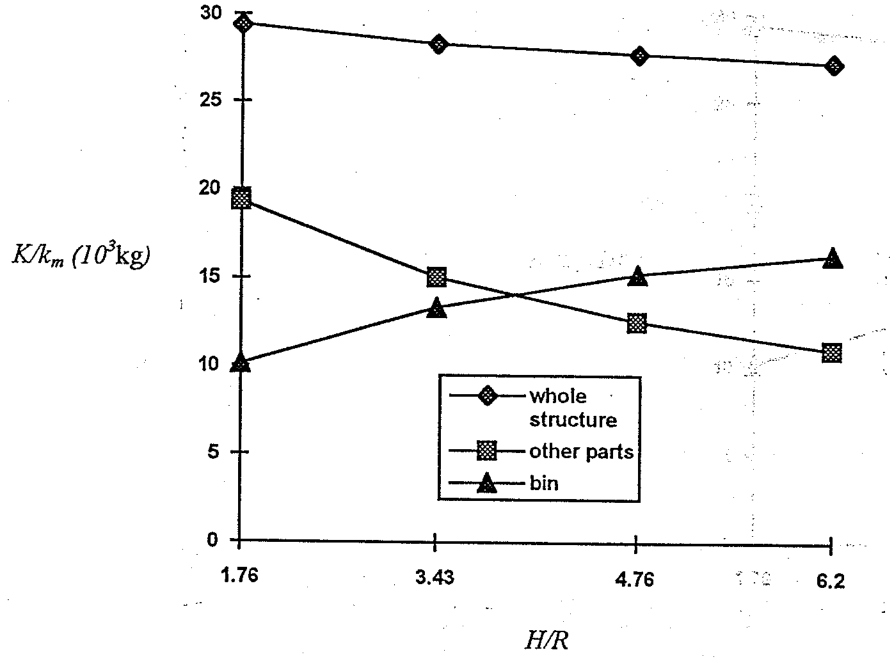

Fig. 3. Cost of silo parts in the function of H/R for k_f/k_m=1.5

Table 3. K/k_m (kg) values for four silos of equal storage capacity of 500 m^3

$\frac{k_f}{k_m}\left(\frac{kg}{min}\right)$	R(m)	4.25	3.50	3.15	2.90
	H(m)	7.50	12.00	15.00	18.00
	H/R	1.76	3.43	4.76	6.20
0	roof	2181	1449	1176	963
	bin	6289	8286	9322	10299
	ringbeam	4585	3653	3003	2521
	hopper	2747	1855	1498	1266
	columns	2681	2231	2068	1952
	total	18483	17474	17067	17001
1.0	roof	3769	2597	2073	1779
	bin	8853	11627	13240	14295
	ringbeam	6101	4943	4170	3597
	hopper	4356	3065	2583	2169
	columns	2681	2231	2068	1952
	total	25760	24463	24134	23792
1.5	roof	4563	3171	2589	2188
	bin	10135	13297	15199	16293
	ringbeam	6859	5888	4754	4135
	hopper	5160	3670	3125	2620
	columns	2681	2231	2068	1952
	total	29398	28257	27735	27188

5. CONCLUSIONS

For a given storage capacity, including the hopper volume, and for given bin height *H*, the radius *R* can be determined. The aim was to find the optimal *H/R* ratio for a given storage capacity in the practical range of *H/R*=1.76-6.20.

The calculations of an illustrative numerical example show the following.

1) When the *H/R* ratio increases *the self weight* of the bin increases but the self weight of the other parts of silo (roof, ringbeam, hopper and columns) decreases. The self weight of the whole structure has a minimum at the practical upper limit of *H/R* = 6.20. The difference between the self weights of the best and worst solution is 100(18483-17001)/17001 = 9 %.

2) *The material and fabrication cost* of the whole structure calculated with formulae (2), (3) and (6) decreases when *H/R* increases and reaches the minimum at the practical upper limit of *H/R*. Thus, designers have to choose the maximal *H/R* to achieve minimal costs. The cost difference between the best and worst solution is 100(29398-27188)/27188 = 8 %.

3) The number of columns should be minimal, the practical minimal number is 6. The slope angle of the hopper should be chosen in accordance with the friction angle of the stored material.

4) In the design of bin thickness the constraints on reduced stress and local buckling should be considered. The effect of a sudden temperature change as well as the dynamic filling and emptying effects are taken into account by multiplying factors. The thickness of the hopper is determined on the basis of the constraint on reduced stress.

5) The optimal dimensions of the ringbeam can be calculated from the stress and local buckling constraints. In the stress constraint the effect of compression and bending should be considered.

6) In the design of columns the effect of snow and wind can be neglected, the leading action is the weight of the stored material. Simple closed formulae can be derived for the optimal dimensions of columns of square hollow section.

REFERENCES

Bodt,H.J.M.: The global approach to welding costs. The Netherlands Institute of Welding, The Hague, 1990.

COSTCOMP, Programm zur Berechnung der Schweisskosten. Deutscher Verlag für Schweisstechnik, Düsseldorf, 1990.

Farkas,J.: Discussion to "Elastic behaviour of isolated column-supported ringbeams" by Rotter, J.M.- J. Constructional Steel Research **4** (1984) pp.235-252. J.C.S.R. **5**(1985) 239-242.

Farkas,J.: Techno-economic considerations in the optimum design of welded structures. Welding in the World **29**(1991) pp. 295-300.

Farkas,J.: Cost comparisons of plates stiffened on one side and cellular plates. Welding in the World **30**(1992) pp. 132-137.

Farkas,J.,Jármai,K.(a): Minimum cost design of laterally loaded welded rectangular cellular plates. Structural Optimization **8**(1994) pp.262-267.

Farkas,J.,Jármai,K.(b): The effect of the low cost technologies on the design of welded structures. Welding in the World **34**(1994) pp.123-130.

Gaylord,E.H.jr. and Gaylord,Ch.N.: Design of steel bins for storage of bulk solids. Prentice Hall,Inc. Englewood Cliffs, New Jersey, 1984.

Martens,P. ed.: Silo-Handbuch. Berlin, Ernst & Sohn, 1988.

Ott,H.H.,Hubka,V.: Vorausberechnung der Herstellkosten von Schweisskonstruktionen (Fabrication cost calculation of welded structures). Proc.Int.Conference on Engineering Design ICED, 1985. Hamburg, Ed. Heurista, Zürich, pp. 478-487. 1985.

Pahl,G.,Beelich,K.H.: Kostenwachstumsgesetze nach Ähnlichkeitsbeziehungen für Schweissverbindungen. VDI-Bericht Nr. 457. pp.129-141. Düsseldorf, 1982.

Trahair,N.S.,Abel,A. et al.: Structural design of steel bins for bulk solids. Australian Institute of Steel Construction, 1983. Sydney.

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