MINIMUM COST DESIGN OF A BELT-CONVEYOR BRIDGE CONSTRUCTED AS A BOX BEAM WITH WELDED CELLULAR WALLS

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ABSTRACT

The aim of the study is to show that a simply supported bent box beam with cellular walls can be a realistic structural version in the case when a strict deflection constraint is prescribed. An unstiffened box beam cannot be realized, since the necessary big plate thicknesses are not suitable for welding. The cellular box beam is welded from thin plates of 4 mm thickness and has much less height and cross-section area than the unstiffened one. Some design and fabrication constraints should be fulfilled, which determine the unknown beam sizes. The cost function to be minimized contents the cost of material, assembly, welding and painting, The optimum structural version is found using a MathCAD program.

KEYWORDS: welded box beams, cellular plates, fabrication cost calculation, structural optimization, deflection constraint, belt-conveyor bridges

1 INTRODUCTION

Belt-conveyor bridges are important structures in heavy industry. Their welded steel structures can be designed as cylindrical circular shells or box beams. The authors have treated the optimum design of belt-conveyor bridges in structural version of ring- or stringer- stiffened cylindrical shell (Farkas and Jármai 2008).

Consider a numerical problem of a simply supported box beam of span length L = 132 m. In order to ensure a sufficient rigidity a vertical deflection constraint of w_{max} = L/1000 = 132 mm is prescribed.

The preliminary calculations show as follows:

(1) To satisfy the deflection constraint, a simple unstiffened welded box beam should have thick (over 50 mm) flanges and webs, which would be unsuitable for welding.
(2) The stress constraint is inactive.
(3) It is unnecessary to consider the horizontal displacement, since the effect of wind load can be neglected.

Thus, it can be concluded that the box beam should have stiffened plate elements. We use cellular plates, since they have some advantages over the plates stiffened on one side.

2 OPTIMUM DESIGN OF THE SIMPLE UNSTIFFENED WELDED BOX BEAM

2.1 Loads

Dead load (belts, rollers, service-walkway) 4.09 N/mm, factored 4.5 N/mm
Live load (for two belts) 8.0 N/mm, factored 12.0 N/mm
Snow load according to EN1991-1-3 (2005)

$s = \mu C_e C_t s_k$, $C_e = C_t = 1$, $\mu = 0.8, s_k = 4.0$ kN/m^2, s = 3.2 kN/m^2 , $sb = 3.2x10^{-3} b$ (*b* in mm)
Factors for simultaneous loads are given in EN 1990 (2005).

Simultaneous snow load 0.5*sb* = 1.6x10^{-3}*b*, factored 1.5x1.6x10^{-3}*b*

Wind load according to EN1991-1-4 (2007)
Wind force $F_W = c_s c_d c_f q_p A_{ref}$
where $c_e c_d = 1, c_f = c_{f0} \psi_f \psi_\lambda$, $c_{f0} = 2.4, \psi_f = 1, \psi_\lambda = 0.88$, $c_f = 2.11$

$q_p = c_e q_b$, $c_e = 1 + 7 I_V$, $I_V = \frac{1}{\ln\left(\frac{z}{z_0}\right)}$, with z = 10, z_0 = 0.05 $I_V = 0.2, c_e = 2.4$

$$q_b = \frac{1}{2} \rho_W v_b^2 = \frac{1}{2} 1.25x23.6^2 = 348 \frac{kg}{ms^2} = 348 \frac{N}{m^2}$$
since

$$v_b = c_{dir} c_{season} v_{b.0} = v_{b.0} = 23.6 \frac{m}{s}$$

$$F_W = 2.11x2.4x348 = 1762Lh$$

Intensity of the uniformly distributed wind load

$$p_W = \frac{F_W}{L} = 1762h = 1.762x10^{-3} h$$, *h* in mm

Simultaneous wind 0.6p_W = 1.057x10^{-3}*h,* factored 1.5x1.057x10^{-3}*h*

Preliminary calculations show that the stress constraint is inactive, thus, the deflection constraint is considered.

2.2 Loads for deflection constraint

Unfactored vertical load

$$p_{VD} = 12.09 + \rho_1 A_D + 1.6x10^{-3} b \tag{1}$$

Unfactored horizontal load

$$p_{HD} = 1.057x10^{-3} h \tag{2}$$

Cross-sectional area for deflection constraint (Fig.1)

$$A_D = ht_W + 2bt_f \tag{3}$$

The active local buckling constraints according to EN 1993-1-1 (2009)

$$\frac{t_W}{2} = \beta_D h, t_f = \delta_D b, \frac{1}{\beta_D} = 69\varepsilon_D, \frac{1}{\delta_D} = 42\varepsilon_D, \varepsilon_D = \sqrt{\frac{235}{\sigma_D}} \tag{4}$$

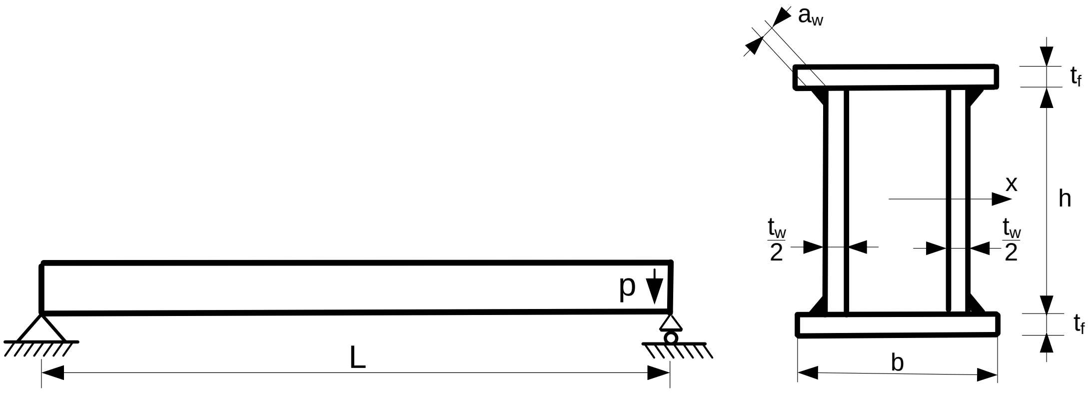

Figure 1. A simply supported welded box beam

Using Eq(4), Eq(3) can be written as

$$A_D = 2\beta_D h^2 + 2\delta_D b^2 \tag{5}$$

Maximal stress for deflection constraint, supposing that the beam is simply supported in both directions

$$\sigma_D = \sigma_{DV} + \sigma_{DH} = \frac{p_{VD} L^2}{8W_{xD}} + \frac{p_{HD} L^2}{8W_{yD}} \tag{6}$$

2.3 Geometric characteristics for deflection constraint

Moments of inertia

$$I_{xD} = \frac{h^3 t_W}{12} + 2bt_f \left(\frac{h}{2}\right)^2 = \frac{\beta_D h^4}{6} + \delta_D b^2 \frac{h^2}{2} \tag{7}$$

$$I_{yD} = \frac{\beta_D h^4}{6} + \delta_D h^2 \frac{b^2}{2} \quad (8)$$

Section moduli

$$W_{xD} = \frac{2I_{xD}}{h} = \frac{\beta_D h^3}{3} + \delta_D b^2 h \quad (9)$$

$$W_{yD} = \frac{\beta_D b^3}{3} + \delta_D h^2 b \quad (10)$$

It can be seen that, in Eqs(4, 6) σ_D depends on itself, thus, an iteration should be performed.

2.4 Deflection constraints

Vertical deflection

$$w_V = \frac{5 p_{VD} L^4}{384 E I_{xD}} \le \frac{L}{1000} \quad (11)$$

Horizontal deflection

$$w_H = \frac{5 p_{HD} L^4}{384 E I_{yD}} \le \frac{L}{1000} \quad (12)$$

Calculation shows that the constraint on horizontal deflection is inactive. Thus, the optimal box beam dimensions (*h,b*) can be determined taking into account the vertical deflection constraint only.

2.5 Optimization results

Table 1 shows the results of the systematic search.

Table 1. Results for unstiffened box beam. The optima are marked by bolt letters. In each case the deflection is 132 mm, a bit smaller than the allowed 132 mm

h mm	*b* mm	A_Dx10^{-6} mm^2	σ_D MPa
9050	6000	2.286	73
9330	5500	2.260	76
9630	**5000**	**2.260**	**80**
9950	4500	2.287	83

It can be seen that the optimum box beam dimensions are *h* = 9630 and *b* = 5000 mm. For these values the thicknesses are according to Eq(4) $t_w/2$ = 81 and t_f = 69 mm. These thicknesses are not suitable for welding, thus, it can be concluded that, in this numerical problem a stiffened box beam should be used.

3 MINIMUM COST DESIGN OF A BOX BEAM WITH CELLULAR WALLS

3.1 Introduction

Cellular plates consist of two face plates and a stiffener grid welded between them. The advantages of cellular plates over those stiffened on one side are as follows: they need smaller stiffener height and should be painted on two outer surfaces only.

For stiffeners halved rolled I-sections or welded T-profiles can be used. In the case when the deflection constraint is active and the normal stresses are small, the welded T-profiles are more economic than the halved rolled I-sections, since their webs can be thinner.

In the case when the deflection constraint is active, the ratio width/height of an optimized beam should be small. Thus, the width of 3600 mm cannot be used as in the previous section. We prescribe a minimum inner width of 2000 mm.

The corners of the box beam are designed using curved circular shell panels (Fig.2).

The unknowns are as follows: height H_0 and width B_0 of the box beam, plate thickness t, height of the cellular plate h, another dimensions of the welded T-stiffeners: web thickness t_w, flange width b and thickness t_f, number of spacings in cellular beam flanges and webs: n_x and n_y.

The constraints are as follows: beam deflection, local buckling of stiffener webs, of cellular plate elements, of circular shell sections, minimum distances of stiffeners for easy welding of connecting fillet welds.

The cost function consists of material cost, cost of assembly welding and painting and is detailed taking into account the fabrication sequence. The cost is calculated for a beam element of length 12 m. Each beam element is welded at the panel ends to connecting plates and the whole beam is constructed by beam elements bolted together. The cost of bolted connections is not treated.

3.2 Data

Span length of the whole beam L = 132 m (11 beam elements of length L_0 = 12 m). Uniformly distributed vertical load of intensity

$$p_{vd} = 12.09 + \rho_1 A + 1.6x10^{-6} B_0, \rho_1 = 7.85x10^{-5} \text{ kg/mm}^3 \tag{13}$$

Yield stress f_y = 355 MPa, elastic modulus E = 2.1x10^5 MPa, Poisson ratio v = 0.3.

3.3 Geometric characteristics

Moment of inertia of a welded T-stiffener (Fig.2)

$$A_S = h_0 t_w + bt_f, \; h_0 = h - t - t_f, h_{01} = h_0 - t_f \tag{14}$$

$$y_G = \frac{h_0 t_w}{A_S}\left(\frac{h_0 + t_f}{2}\right) \tag{15}$$

$$I_S = \frac{h_0^3 t_w}{12} + h_0 t_w \left(\frac{h_0 + t_f}{2} - y_G\right)^2 + \frac{b t_f^3}{12} + b t_f y_G^2 \tag{16}$$

Radii of the circular shell corners (Fig.2) are as follows: outer shell segment 2*h*, inner segment *h*. Thus, the widths of the box beam flange and heights of webs are

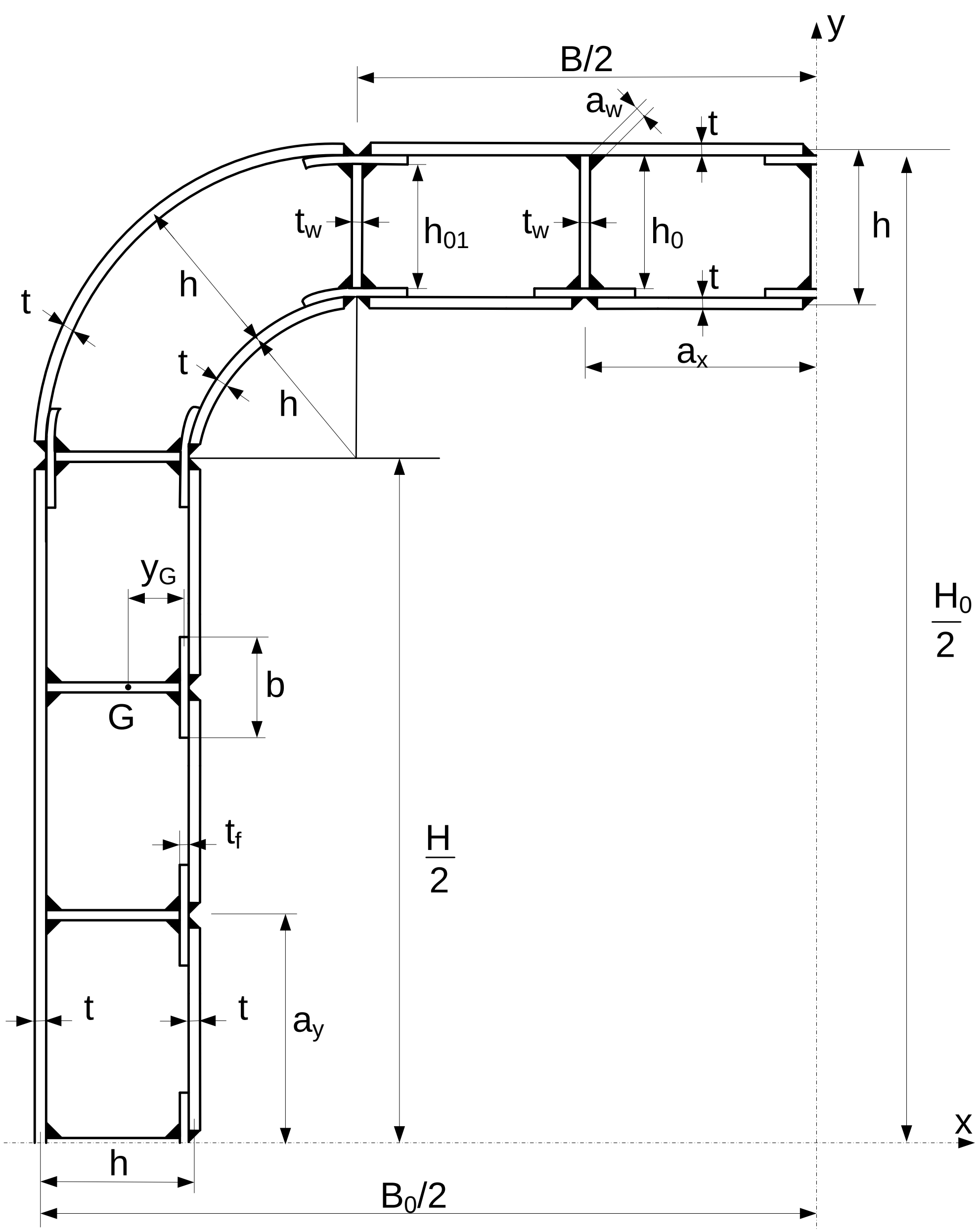

Fig.2 A quarter of the symmetric cross-section of the cellular box beam

$$B = B_0 - 4h = n_x a_x, H = H_0 - 4h = n_y a_y \tag{17}$$

Moment of inertia of a box beam flange to the *x*-axis of the whole box beam cross-section

where n_x and n_y are numbers of spacings, a_x and a_y are distances of stiffeners.

$$I_{x1} = \frac{Bt^3}{6} + Bt\left(2h + \frac{H}{2}\right)^2 + Bt\left(h + \frac{H}{2}\right)^2 + \left(n_x - 1\right)\left[I_S + A_S\left(y_G + h + \frac{H}{2}\right)^2\right] \tag{18}$$

Moment of inertia of a box beam web

$$I_{x2} = \frac{H^3 t}{6} + I_{S1} \tag{19}$$

$$I_{S1} = \left(\frac{b^3 t_f}{12} + \frac{h_0 t_w^3}{12}\right)\frac{\left(n_y^2 - 1\right)H^2}{48 n_y} \tag{20}$$

The two quarters shell parts are calculated together as a half circular hollow section. Thus, for the outer shell parts

$$I_{x3} = \frac{(2h)^3 \pi t}{2}\left(1 - \frac{8}{\pi^2}\right) + 2h\pi t\left(\frac{4h}{\pi} + \frac{H}{2}\right) \tag{21}$$

and for the inner shell parts

$$I_{x4} = \frac{h^3 \pi t}{2}\left(1 - \frac{8}{\pi^2}\right) + h\pi t\left(\frac{2h}{\pi} + \frac{H}{2}\right) \tag{22}$$

The connection of the quarter shell corner parts with the beam flanges and webs are constructed with 8 welded I-stiffeners (Fig.2). The moment of inertia of I-stiffeners connecting the beam flange (vertical corner stiffeners)

$$I_{ST1} = \frac{h_0^3 t_w}{12} + 2\left[\frac{bt_f^3}{12} + bt_f\left(\frac{h - t}{2}\right)^2\right] + A_{S1}\left(\frac{3h}{2} + \frac{H}{2}\right)^2 \tag{23}$$

and for those connecting the beam webs (horizontal corner stiffeners)

$$I_{ST2} = \frac{h_0 t_w^3}{12} + \frac{b^3 t_f}{6} + A_{S1}\left(\frac{H}{2}\right)^2 \tag{24}$$

$$A_{S1} = h_0 t_w + 2bt_f \tag{25}$$

Moment of inertia of the connecting stiffeners

$$I_{x5} = 4\left(I_{ST1} + I_{ST2}\right) \tag{26}$$

Moment of inertia of the whole beam cross-section

$$I_x = 2I_{x1} + 2I_{x2} + 2I_{x3} + 2I_{x4} + I_{x5} \tag{27}$$

Cross-section area of the whole beam

$$A = 4Bt + 2(n_x - 1)A_S + 4Ht + 2(n_y - 1)A_S + 6h\pi t + 8A_{S1} \tag{28}$$

3.3 Constraint on deflection

The deflection constraint is formulated as

$$w_{\max} = \frac{5p_{vd}L^4}{384EI_x} \le \frac{L}{1000} \tag{29}$$

In order to calculate the minimum stiffener web thickness t_w, the normal stress should be calculated

$$\sigma_d = \frac{p_{vd}L^2}{8W_{xd}}, W_{xd} = \frac{2I_x}{H_0} \tag{30}$$

This stress is small, since the constraint on deflection is active.

3.4 Constraint on minimum stiffener web thickness

$$t_{w\min} = \delta_d h_0, \delta_d = \frac{1}{42\varepsilon_d}, \varepsilon_d = \sqrt{\frac{235}{\sigma_d}} \tag{31}$$

3.5 Constraints on local buckling of plate parts between stiffeners

$$n_{x.\min} = \frac{B}{42\varepsilon_d t}, \varepsilon_d = \sqrt{\frac{235}{\sigma_d}} \tag{32}$$

$$n_{y.\min} = \frac{H}{42\varepsilon_{d1} t}, \varepsilon_{d1} = \sqrt{\frac{235}{\sigma_{d1}}}, \sigma_{d1} = \sigma_d \frac{H}{H_0} \tag{33}$$

3.6 Constraint on local buckling of an outer quarter circular shell at corners

According to DNV rules (2002) for unstiffened circular shell panels

$$\sigma_d \le \sigma_s = \frac{f_y}{\sqrt{1+\lambda_s^4}}, \lambda_s = \sqrt{\frac{f_y}{\sigma_E}}, \sigma_E = C\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{s}\right)^2, \; s = h\pi \tag{34}$$

$$C = \psi\sqrt{1+\left(\frac{\rho_0\xi}{\psi}\right)^2}, \psi = 4, \rho_0 = 0.5\left(1+\frac{R}{150t}\right)^{-0.5}, R = 2h, \quad (35)$$

$$\xi = 0.702 Z_s, Z_s = \frac{s^2\sqrt{1-\nu^2}}{Rt} \quad (36)$$

3.7 Calculation of cost

The cost parts are formulated according to the fabrication sequence. The cost calculation method is described in (Farkas and Jármai 2008).

Data: k_w = 1.0\$/min, factors for difficulty of assembly Θ = 2, Θ_1 = 3, ρ = 7.85x10^{-6} kg/mm^3

(1a) Welding of an outer plate for a flange of width B and length L_0 with SAW butt welds (number of outer plates is 2)

It is supposed that the plates are composed by plate elements of length 6000 mm and width equal or smaller than 1500 mm. Number of plate elements is q_1 = B/1500 rounded for the greater even value. The volume, weld length and number of assembled elements are as follows

$$V_{11} = BtL_0, L_{w11} = B + (q_1 - 1)L_0, \kappa_{11} = 2q_1, q_1 = 2, \quad (37)$$

while B is limited to 2000 mm to have a sufficient place for belt conveyors and service personal. The welding cost is written as

$$K_{w11} = k_w\left(\Theta\sqrt{\kappa_{11}\rho V_{11}} + 1.3C_w t^n L_{w11}\right) \quad (38)$$

For t<11 C_w = 0.1346x10^{-3} t^{2}
For t>11 C_w = 0.1033x10^{-3} $t^{1.909}$

(1b) Welding of an outer plate for a web of width H and length L_0 with SAW butt welds (number of outer plates is 2)

$$V_{12} = HtL_0, L_{w12} = H + (q_2 - 1)L_0, \kappa_{12} = 2q_2, q_2 = H/1500 \text{ rounded up} \quad (39)$$

$$K_{w12} = k_w\left(\Theta\sqrt{\kappa_{12}\rho V_{12}} + 1.3C_w t^n L_{w12}\right) \quad (40)$$

(2a) Forming of circular shells for four outer corners of radius R =2h and length of 3000. Number of shells is 4.

$$K_{F1} = k_w\Theta e^{\mu_{11}}, \mu_1 = 6.8582513 - 4.527217t^{-0.5} + 0.009541996(4h)^{0.5} \quad (41)$$

(2b) Welding together the 4 circular shell elements with SAW butt welds

$$V_2 = 4h\pi L_0 t, L_{w2} = 3x4h\pi, \kappa_2 = 4 \quad (42)$$

$$K_{w2} = k_w \left(\Theta\sqrt{\kappa_2 \rho V_2} + 1.3C_w t^n L_{w2} \right) \quad (43)$$

These circular shells are cut to form four quarter corners, but the cost of cutting is neglected.

(3) Welding of plate strips for inner plate parts and stiffener webs and flanges with SAW butt welds

(3a) Strips for beam flanges (number of strips is 2n_x

$$V_{31} = a_x t L_0, L_{w31} = a_x, \kappa_{31} = 2, a_x = B / n_x \quad (44)$$

$$K_{w31} = k_w \left(\Theta\sqrt{\kappa_{31} \rho V_{31}} + 1.3C_w t^n L_{w31} \right) \quad (45)$$

(3b) Strips for beam webs (number of strips is 2n_y)

$$V_{32} = a_y t L_0, L_{w32} = a_y, \kappa_{32} = 2, a_y = H / n_y \quad (46)$$

$$K_{w32} = k_w \left(\Theta\sqrt{\kappa_{32} \rho V_{32}} + 1.3C_w t^n L_{w32} \right) \quad (47)$$

(3c) Forming of circular shells for four outer corners of radius $R = h$ and length of 3000. Number of shells is 4.

$$K_{F2} = k_w \Theta e^{\mu_{12}}, \mu_2 = 6.8582513 - 4.527217 t^{-0.5} + 0.009541996(2h)^{0.5} \quad (48)$$

Welding together the 4 circular shell elements with SAW butt welds

$$V_3 = 2h\pi L_0 t, L_{w33} = 3x2h\pi, \kappa_3 = 4 \quad (49)$$

$$K_{w33} = k_w \left(\Theta\sqrt{\kappa_3 \rho V_3} + 1.3C_w t^n L_{w33} \right) \quad (50)$$

(3d) Strips for stiffener webs [number of strips is (2n_x-2+2n_y-2)]

$$V_{34} = h_0 t_w L_0, L_{w34} = h_0, \kappa_{34} = 2 \quad (51)$$

$$K_{w34} = k_w \left(\Theta\sqrt{\kappa_{34} \rho V_{34}} + 1.3C_w t^n L_{w34} \right) \quad (52)$$

(3e) Strips for stiffener flanges [number of strips is (2n_x-2+2n_y-2+16)](16 is number of flanges for transient stiffeners)

$$V_{35} = b t_f L_0, L_{w35} = b, \kappa_{35} = 2 \quad (53)$$

$$K_{w35} = k_w \left(\Theta\sqrt{\kappa_{35} \rho V_{35}} + 1.3C_w t^n L_{w35} \right) \quad (54)$$

(3f) Strips for webs of transient stiffeners [number of strips is 8)

$$V_{36} = (h_0 - t_f)t_w L_0, L_{w36} = h_0 - t_f, \kappa_{36} = 2 \quad (55)$$

$$K_{w36} = k_w\left(\Theta\sqrt{\kappa_{36}\rho V_{36}} + 1.3C_w t^n L_{w36}\right) \quad (56)$$

(4) Welding of stiffened beam flanges (outer plates with stiffeners) (number of flanges is 2) with SAW fillet welds

$$V_4 = BtL_0 + (n_x - 1)A_S L_0, L_{w4} = 4(n_x - 1)L_0, \kappa_4 = 1 + 2(n_x - 1) \quad (57)$$

$$K_{w4} = k_w\left(\Theta_1\sqrt{\kappa_4\rho V_4} + 1.3x0.2349x10^{-3} a_w^2 L_{w4}\right) \quad (58)$$

a_w = 3 mm

(5) Welding of stiffened beam webs (outer plates with stiffeners) (number of webs is 2) with SAW fillet welds

$$V_4 = HtL_0 + (n_y - 1)A_S L_0, L_{w5} = 4(n_y - 1)L_0, \kappa_5 = 1 + 2(n_y - 1) \quad (59)$$

$$K_{w5} = k_w\left(\Theta_1\sqrt{\kappa_5\rho V_5} + 1.3x0.2349x10^{-3} a_w^2 L_{w5}\right) \quad (60)$$

(6) Welding of a stiffener to a quarter outer circular shell. In our case it is superfluous, since the quarter outer circular shell does not need stiffening against buckling, K_{w6} = 0

(7) Welding of transient welded I stiffeners (number = 8) with SAW fillet welds

$$V_7 = A_{S1}L_0, L_{w7} = 4L_0, \kappa_7 = 3 \quad (61)$$

$$K_{w7} = k_w\left(\Theta\sqrt{\kappa_7\rho V_7} + 1.3x0.2349x10^{-3} a_w^2 L_{w7}\right) \quad (62)$$

(8) Assembly and welding of the whole outer beam part with SAW fillet welds

$$V_8 = 2V_4 + 2V_5 + 4V_6 + 8V_7, V_6 = h\pi t L_0, L_{w8} = 16L_0, \kappa_8 = 16 \quad (63)$$

$$K_{w8} = k_w\left(\Theta_1\sqrt{\kappa_8\rho V_8} + 1.3x0.2349x10^{-3} a_w^2 L_{w8}\right) \quad (64)$$

(9) Welding of the outer beam parts to the connecting plates with GMAW fillet welds

$$V_9 = V_8, L_{w9} = 2B + 2H + 4h\pi + 4(h_0 + b)(n_x + n_y - 2) + 8(h_{01} + 3b), \kappa_9 = 3 \quad (65)$$

$$K_{w9} = k_w\left(\Theta_1\sqrt{\kappa_9\rho V_9} + 1.3x0.3394x10^{-3} a_w^2 L_{w9}\right) \quad (66)$$

(10) Welding of the inner plate strips to the stiffeners' flange with SAW fillet welds

$$V_{10} = V_9 + 2t(B+H)L_0 + 2h\pi t L_0, L_{w10} = 4(n_x + n_y + 2)L_0, \kappa_{10} = 1 + 4 + 2(n_x + n_y) \quad (67)$$

$$K_{w10} = k_w\left(\Theta_1\sqrt{\kappa_{10}\rho V_{10}} + 1.3x0.2349x10^{-3}a_w^2 L_{w10}\right) \quad (68)$$

(11) Welding of the inner beam periphery to the connecting plates

$$V_{11} = V_{10}, L_{w11} = 2(2B + 2H + 2h\pi), \kappa_{11} = 1 \quad (69)$$

$$K_{w11} = k_w\left(\Theta\sqrt{\kappa_{11}\rho V_{11}} + 1.3x0.2349x10^{-3}a_w^2 L_{w11}\right) \quad (70)$$

The material cost

$$K_M = k_M \rho V_{10}, \; k_M = 1.0 \text{ \$/kg} \quad (71)$$

The painting cost

$$K_P = k_P S, S = L_0(4B + 4H + 4h\pi + 2h\pi), \; k_P = 28.8\text{x}10^{-6} \text{ \$/mm}^2 \quad (72)$$

The shell forming cost

$$K_F = K_{F1} + K_{F2} \quad (73)$$

The welding cost

$$K_w = 2(K_{w11} + K_{w12}) + K_{w2} + K_{w3} + 2(K_{w4} + K_{w5}) + 8K_{w7} + K_{w8} + 2K_{w9} + K_{w10} + K_{w11} \quad (74)$$

where

$$K_{w3} = 2n_x K_{w31} + 2n_y K_{w32} + K_{w33} + (2n_x + 2n_y - 4)K_{w34} + K_{w56} \quad (75)$$

and

$$K_{w56} = (2n_x + 2n_y + 12)K_{w35} + 8K_{w36}, \; k_w = 1.0 \text{ \$/min} \quad (76)$$

The total cost

$$K = K_M + K_F + K_w + K_P \quad (77)$$

3.8 Optimization and results

Unknown dimensions to be optimized are as follows: B_0, H_0, t, t_w, t_f, h, b, n_x, n_y.

The following fabrication constraints should be fulfilled:

(1) $B = B_0 - 4h = 2000$ mm to guarantee the inner place for belt-conveyors and service persons,
(2) minimum plate thicknesses: $t = t_w = t_f = 4$ mm,
(3) for fabrication reasons dimensions of *h* and *b* are minimized to *h* = 150, *b* = 200 mm,
(4) the buckling constraint for t_{wmin} is passive, since, in the case of active very strict deflection constraint the stress σ_d is small,
(5) numbers of spacings n_x and n_y are determined by two active constraints as follows: (a) constraints on local buckling of outer plate parts between stiffeners limit a_{xmax} and a_{ymax}, (b) the minimum distance of stiffeners' flanges a_x - *b* and a_y – *b* is limited to 300 mm.

The remained unknown H_0 is determined using the deflection constraint. In order to show that these dimensions give also the minimum cost, the cost is calculated also for *t* = 5 mm and for B_0 = 2800 (*B* = 2200 mm). The results are given in Table 2.

Table 2. Calculation results. Optimum is marked by bold letters. Dimensions in mm, stress in MPa, cost in $, volume in 10^{-9} mm^3, w_{allow} = 132 mm

B	B_0	H_0	t	n_x	n_y	σ_d	w	V_{10}	K_w	K_P	K
2200	2800	3190	4	4	5	24	131.8	1.492	13940	7598	33740
2000	**2600**	**3165**	**4**	**4**	**5**	**24**	**131.6**	**1.449**	**13830**	**7288**	**32980**
2000	2600	3240	5	4	5	25	131.7	1.719	14510	7391	36010

4 CONCLUSIONS

It is shown that the box beam with cellular plated walls is a realistic structural version, when a strict deflection constraint should be fulfilled and the unstiffened box section cannot be welded for very thick plates.

The minimum cost design of the box beam with cellular plated walls can easily be performed, since a lot of buckling and fabrication constraints should be fulfilled and the remained beam height can be determined from deflection constraint.

The comparison of Tables 1 and 2 shows that the cellular box beam can be constructed with a much less high than the unstiffened one. The comparison of the cross-section areas shows that for a 12 m long element of the unstiffened beam *A* = 2.26x10^6 mm^2 and for the cellular beam *A* = 1.449x10^9/(12x10^3) = 0.1207x10^6 mm^2, which is much less than that for unstiffened beam.

The cost function is formulated according to the fabrication sequence, which has 10 steps (From 11 steps one is superfluous.). The cost is calculated for a beam element of length 12 m and the whole beam of length 132 m is constructed from these beam elements by bolted connecting plates.

The detailed analysis of the wind load shows that, in the investigated case it can be neglected

In the case of a very strict deflection constraint the normal stress due to bending is small, thus, the minimum plate thickness of 4 mm can be used for all the structural parts. For this plate thickness the minimum fillet weld size of 3 mm is used.

It can be seen from Table 2 that the welding cost is relatively high, since the cellular plates need a lot of longitudinal welds.

The stiffened box beam is treated as one having double symmetric cross section, i.e. it is neglected that the cross-sectional parts not stressed for compression need less stiffeners.

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