8th World Congress on Structural and Multidisciplinary Optimization
June 1 - 5, 2009, Lisbon, Portugal

Cost minimization of a circular floor constructed from welded stiffened steel sectorial plates

Károly Jármai, József Farkas

University of Miskolc, Miskolc, Hungary, altjar@uni-miskolc.hu

1. Abstract
The problem of finding the most economic (minimum cost) structural version of a large diameter circular floor constructed from stiffened sectorial plate elements supported by radial beams is a threefold optimization problem:

(a) Determination of the most economic stiffening of a sectorial plate. The cost of tangential non-equidistant stiffening for a fixed number of sectors (ω=12) is calculated for different base plate thicknesses (t = 4, 6, 8). The distances of the tangential non-equidistant stiffening are determined by using an iterative algorithm. The cost of combined (tangential and radial) stiffening for different thicknesses (t = 4,8) is calculated. The costs of various stiffenings show that the non-equidistant tangential stiffening and the base plate thickness of 4 mm give the minimum cost solution.

(b) Determination of the optimum dimensions of radial welded box beams for a circular floor supported at the centre.

(c) The total costs of the floor structure calculated for different numbers of sectorial plates (8, 12 and 16) show that the number of 12 gives the minimum total cost.

2. Keywords: welded structures, stiffened sectorial plates, cost calculation, optimum design, circular floor

3. Introduction
Circular plates can be applied in many steel structures such as floors, roofs, stair landings, etc. They can be constructed from sectorial plates supported by radial beams.

The sectorial (trapezoidal) plate elements can be stiffened by tangential or radial stiffeners or these stiffeners can be combined. The stiffeners can be flat, halved rolled I-section (T-shape), trapezoidal or other shapes. They are welded to the base plate by double fillet welds.

We use in the present study halved rolled I-section stiffeners and we investigate both tangential and combined stiffening.

4. Problem formulation
The problem of finding the most economic (minimum cost) structural version of a large diameter circular floor constructed from stiffened sectorial plate elements supported by radial beams (Figure 1) is *a threefold optimization task:* (1) determination of the most economic stiffening of a sectorial plate, (2) determination of the optimum dimensions of radial beams, (3) determination of the optimum number of sectors. Cost comparisons help to find the most economic structural version. For this purpose we have developed a cost calculation method applicable mainly for welded structures [1,2,3].

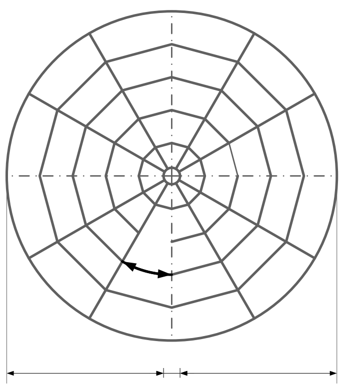

Figure 1: A large diameter circular floor with 12 tangentially stiffened sectorial plates

5. Solution strategy for the three optimization phases

(a) Calculate the cost of tangential non-equidistant stiffening for a fixed number of sectors (ω=12) for different base plate thicknesses (*t* = 4, 6, 8), then calculate the cost for combined (tangential and radial) stiffening for different thicknesses (*t* = 4,8);
(b) calculate the optimum dimensions of radial welded box beams for a circular floor supported at the centre, using stress and deflection constraints;
(c) cost calculation of the whole floor structure for different numbers of sectors (ω = 8,12,16).

6. Minimum cost design of a sectorial plate

6.1. Non-equidistant tangential stiffening

6.1.1. Calculation of stiffener distances (x_{0i})

These distances are determined using the condition that the maximum normal stress due to bending in each plate element between stiffeners should not be larger than the yield stress. The maximum bending moment in a deck plate element is calculated approximately for a simply supported rectangular plate according to Timoshenko and Woinowsky-Krieger [4]

$$M_{i\max} = \beta_i p_M a_i^2 \tag{1}$$

where a_i is the smaller side length and β_i is given in function of $b_i / a_i \geq 1$ in Table 1.

Table 1: Bending moment factors

b/a	1	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	3.0	4.0	5.0	>5
$10^4\beta$	479	554	627	694	755	812	862	908	948	985	1017	1189	1235	1246	1250

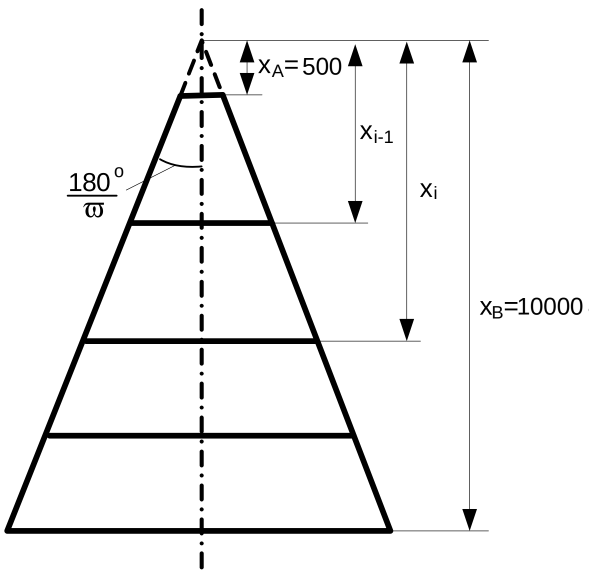

Figure 2: Non-equidistant tangential stiffening

For the calculation of stiffener lengths we introduce the factor of f_ω = 2tgα.
The values of Table 1 are approximated by the following expressions (Figure 2)

$$\beta_i = \beta_{\xi i} \quad \text{if} \quad x_i - x_{i-1} \leq x_i f_\omega \quad \text{i.e.} \quad x_i \leq \frac{x_{i-1}}{1 - f_\omega} \tag{2}$$

$$\beta_i = \beta_{\eta i} \quad \text{if} \quad x_i - x_{i-1} > x_i f_\omega \tag{3}$$

$$\beta_{\xi i} = a_0 + b\xi_i + c\xi_i^2 + d\xi_i^3 + e\xi_i^4 \qquad \xi_i = \frac{x_i f_\omega}{x_i - x_{i-1}} \tag{4}$$

$$\beta_{\eta i} = a_0 + b\eta_i + c\eta_i^2 + d\eta_i^3 + e\eta_i^4 \qquad \eta_i = \frac{x_i - x_{i-1}}{x_i f_\omega} \tag{5}$$

a_0 = -0.08022658, b = 0.180443, c = -0.061636, d = 0.009575, e = -0.00056537
t is the deck plate thickness, f_y = 235 MPa is the yield stress, $f_{y1} = f_y/1.1$ The factored intensity of the uniformly distributed normal load is p_M = 1.5x500 kg/m^2 = 7.5x10^{-3} N/mm^2.
Using Eq(1) from equation

$$M_{i\max} = f_{y1} t^2 / 6 \tag{6}$$

one obtains

$$r_i = \sqrt{\frac{t^2 f_{y1}}{6\beta_i p_M}} \tag{7}$$

and the sought stiffener distance is

$$x_{0i} = r_i + x_{i-1} \quad \text{if} \quad x_i \le \frac{x_{i-1}}{1-f_\omega} \tag{8}$$

$$x_{0i} = \frac{r_i}{f_\omega} \qquad \text{if} \quad x_i > \frac{x_{i-1}}{1-f_\omega} \tag{9}$$

The value of x_{0i} can be obtained by iteration with a MathCAD program.
It should be noted that in this calculation the transverse bending moments are neglected but the plate elements are calculated as simply supported and it is also neglected that their edges are partially clamped.

6.1.2. Design of stiffeners
A stiffener is subject to a bending moment

$$M_{si\max} = p_M s_i x_i^2 f_\omega^2 / 8 \tag{10}$$

where $s_i = \dfrac{x_{i+1} - x_{i-1}}{2}$

and the effective plate width according to design rules of Det Norske Veritas [5]

$$s_{ei} = \left(\frac{1.8}{\beta_{0i}} - \frac{0.8}{\beta_{0i}^2} \right) s_i \tag{11}$$

where

$$\beta_{0i} = \frac{s_i}{t} \sqrt{\frac{f_y}{E}}\,, \quad \text{but} \quad \beta_{0i} \ge 1 \tag{12}$$

E = 2.1x10^5 MPa is the elastic modulus.
The required section modulus is given by

$$W_{0i} = \frac{M_{si\max}}{f_{y1}} \tag{13}$$

The cross-sectional area of a stiffener of halved rolled I-section and the effective plate part (Figure 4)

$$A_{ei} = \frac{h_{1i} t_{wi}}{2} + b_i t_{fi} + s_{ei} t\,, \quad h_{1i} = h_i - 2t_{fi} \tag{14}$$

The distances of the gravity centres G_i

$$z_{Gi} = \frac{1}{A_{ei}} \left[\frac{h_{1i} t_{wi}}{2} \left(\frac{h_{1i}}{4} + \frac{t}{2} \right) + b_i t_{fi} \left(\frac{h_i + t - t_{fi}}{2} \right) \right] \tag{15}$$

and $$z_{G1i} = \frac{h_i + t - t_{fi}}{2} - z_{Gi} \tag{16}$$

the moments of inertia

$$I_{yi} = s_{ei} t z_{Gi}^2 + \frac{h_{1i}^3 t_{wi}}{96} + \frac{h_{1i} t_{wi}}{2} \left(\frac{h_{1i}}{4} + \frac{t}{2} - z_{Gi} \right)^2 + b_i t_{fi} \left(\frac{h_i + t - t_{fi}}{2} - z_{Gi} \right)^2 . \tag{17}$$

The section moduli are defined as

$$W_{yi} = I_{yi}/z_{0i} \tag{18}$$

where z_{0i} is the greater of z_{Gi} and z_{G1i}.
The required stiffener profile is selected from Table 2 to fulfil the stress constraint

$$W_{yi} \ge W_{0i} \tag{19}$$

Table 2: UB-profiles used for halved rolled I-section stiffeners

UB profile	*h*	*B*	t_w	t_f
152x89x16	152.4	88.7	4.5	7.7
168x102x19	177.8	101.2	4.8	7.9
203x133x26	203.2	133.2	5.7	7.8
254x102x25	257.2	101.9	6.0	8.4
305x102x28	308.7	101.8	6.0	8.8
305x102x33	312.7	102.4	6.6	10.8
406x178x60	406.4	177.9	7.9	12.8

6.1.3. Cost calculation for a sectorial stiffened plate element

The fabrication sequence has two parts:

(a) Welding of the base plate from 6 elements using SAW (Submerged Arc Welding) butt welding. The length of the plate (9500 mm) is divided into 6 parts welded together with 5 butt welds using SAW technology. The total length of welds is

$$L_{w1} = 27500 f_{\omega} \tag{20}$$

The general formula for the welding cost is as follows [1, 2, 3]:

$$K_w = k_w \left(C_1 \Theta \sqrt{\kappa \rho V} + 1.3 \sum_i C_{wi} a_{wi}^n C_p L_{wi} \right) \tag{21}$$

where k_w [\$/min] is the welding cost factor, C_1 is the factor for the assembly usually taken as C_1 = 1 min/kg$^{0.5}$, Θ is the factor expressing the complexity of assembly, the first member calculates the time of the assembly, κ is the number of structural parts to be assembled, ρV is the mass of the assembled structure, the second member estimates the time of welding,

C_w and n are the constants given for the specified welding technology and weld type, C_{pi} is the factor for the welding position (download 1, vertical 2, overhead 3), L_w is the weld length, the multiplier 1.3 takes into account the additional welding times (deslagging, chipping, changing the electrode).

The cost in the fabrication phase (a) is calculated as

$$K_{w1} = k_w \left(\Theta_1 \sqrt{6 \rho V_1} + 1.3 C_{w1} t^2 L_{w1} \right) \tag{22}$$

where

$k_w = 1.0\$/\mathrm{min}, \Theta_1 = 2, \rho = 7.85x10^{-6}$ kg/mm^3, C_{w1} = 0.1559x10^{-3},

$$V_1 = \frac{10000 + 500}{2} 9500 f_{\omega} t = 49.875 x 10^6 f_{\omega} t \tag{23}$$

(b) Welding of stiffeners to the base plate and to two edge radial plates to complete a sectorial plate element using fillet welds:

$$K_{w2} = k_w \left(\Theta_2 \sqrt{(n_{st} + 3) \rho V_2} + \sum_i T_i + T_s \right) \tag{24}$$

where n_{st} is the number of stiffeners, $\Theta_2 = 3$,

$$V_2 = V_1 + V_s + \sum_i V_{sti} \tag{25}$$

the volume of the edge radial plates is

$$V_s = 2x9500 h_s t_s \tag{26}$$

h_s and t_s are the dimensions of the half radial beam web of a box section estimated preliminary and obtained by iteration taking into account the self mass of the stiffened sector.

Volume of a stiffener is

$$V_{sti} = A_{sti} x_i f_{\omega}, A_{sti} = \frac{h_{1i} t_{wi}}{2} + b_i t_{fi} \tag{27}$$

welding time for a stiffener is

$$T_i = 1.3 C_{w2} a_w^2 2 x_i f_{\omega} + 1.3 C_{w3} a_w^2 2 (2 h_{1i} + 4 b_i) \tag{28}$$

where $C_{w2} = 0.2349x10^{-3}, C_{w3} = 0.7889x10^{-3}$

constants for SAW and SMAW (Shielded Metal Arc Welding) fillet welds, respectively,

a_w = 3 mm, the second part is multiplied by 2, since the welding position is mainly vertical.

The time of welding of the two edges radial plates to the base deck plate is

$$T_s = 1.3C_{w3}a_w^2 L_s, L_s = 2x9500 \qquad (29)$$

Material cost of a complete sectorial element is

$$K_{m1} = k_m \rho V_2, k_m = 1.0 \text{ \$/kg.} \qquad (30)$$

The painting cost of a complete sectorial element is

$$K_{P1} = k_P S, k_P = 28.8x10^{-6} \text{ \$/mm}^2, \qquad (31)$$

$$S = S_s + \sum_i S_{sti} + 2x49.875x10^6 f_\omega \qquad (32)$$

$$S_s = 2x9500h_s \qquad (33)$$

$$S_{sti} = (h_{1i} + 2b_i)x_i f_\omega \qquad (34)$$

The total cost of a sectorial element is

$$K_s = K_{m1} + K_{w1} + K_{w2} + K_{P1} \qquad (35)$$

Results of cost calculation for a sectorial element of ω = 12 show that the minimum cost corresponds to the thickness of t = 4 mm. (See Table 4.). Therefore the further calculations are performed for this thickness only. Table 3 shows the calculated stiffener distances and sizes for ω = 12 and t = 4 mm.

Table 3: Stiffener distances and sizes in mm for ω = 12 and t = 4 mm

x_i	500	1890	2824	3652	4451	5239	6023	6804	7585	8373	9177	1000
h	-	152.4	152.4	152.4	152.5	152.4	152.4	152.4	177.8	177.8	203.2	-

6.2. Combined tangential and radial stiffening

The equidistant tangential stiffening and a constant base plate thickness is not an economic solution, since in this case the outermost plate part is governing for the bending stress and the other parts cannot be stressed for the allowable stress. In this case it is better to use additional radial stiffeners as well. In the case of combined stiffening the most economic solution is to design near square plate parts, since a plate of square symmetry needs the minimal thickness to be stressed by bending to allowable stress.

The maximum side dimension of a square isotropic plate with all edges built-in can be calculated using the bending moment factor of β=0.0513 [4, p.197] similar that in Eq. (7)

$$a_{max} = t\sqrt{\frac{f_{y1}}{6\beta p}} \qquad (36)$$

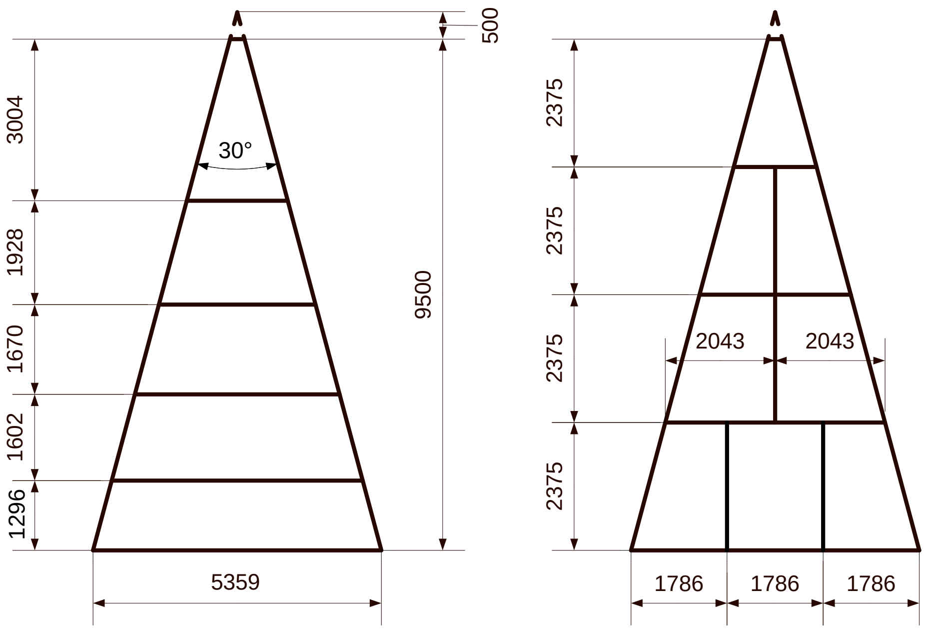

Figure 3. Non-equidistant tangential and combined stiffening in the case of ω = 12 and t = 8 mm

For f_y = 235 MPa, factored uniformly distributed normal load of intensity p= 1.5x500 = 750 kg/m^2 = 7.5x10^{-3} N/mm^2 and thickness t = 4 mm one obtains a_{max} = 1217 mm. Figure 3. shows the combined stiffening with near square distances for t = 8 mm.
The sectorial plate with combined stiffening could be designed as a grillage system, but this calculation would be very complicated. Therefore we neglect the grillage effect and design the stiffeners as simply supported beams. The tangential stiffeners can be designed according to the method shown in section 4.1.2 and the radial stiffeners as simply supported beams of span length 1187 mm.
The safety against local buckling of the base plate parts is considered by using the method of effective plate width. For the effective width there are different formulae proposed by Eurocode 3 [6] or DNV rules [5]. We use here the formulae of DNV rules [5].
It should be mentioned that the effect of normal load on the local plate buckling can be neglected, since – according to Paik and Thayamballi [7] the normal load increases the buckling strength.
The corresponding dimension of the radial stiffeners is h = 152.4 mm. It should be mentioned that the required thickness of the outermost plate part without radial stiffeners would be t = 6.1 mm, thus radial stiffeners should be used.
For thickness t = 8 mm the maximum side length is a_{max} = 2570 mm. The corresponding combined stiffening is shown is Figure 3.

6.2. Cost comparison of the sectors with different stiffenings
The costs calculated for non-equidistant tangential and for the equidistant combined stiffening are summarized in Table 4.

Table 4: Costs in $ for a sectorial stiffened plate in the case of ω = 12. K_{w3} is the welding cost of the radial stiffeners. The minimum cost is marked by bolt letters

Stiffening	t	K_m	K_{w1}	K_{w2}	K_{w3}	K_p	K_s
non-equidistant	4	1594	190	558	-	2098	**4439**
Tangential	6	1940	281	482	-	2007	4710
	8	2318	392	422	-	1958	5089
Combined	4	1524	190	511	722	2183	5130
	8	2277	392	341	402	1989	5402

It can be seen that the minimum cost solution is the sector with non-equidistant stiffening of thickness t = 4 mm. Therefore this type of sectorial plate will be applied for other values of ω.

7. Optimum design of radial beams
In order to facilitate the assembly the sectorial plates have only side plates. These plates form the webs of the radial beams of welded box section. When all sectors are assembled, these side plates are connected with upper and lower flange plates (Figure 4).
The radial beams are designed for bending considering stress and deflection constraints. The formulation of the optimum design of a radial box beam is as follows: find the optimum
values of the dimensions h_s, t_s and the cross-section area of a flange $A_f = b_s t_f$ to minimize the whole cross-section area

$$A = 2h_s t_s + 2A_f \tag{37}$$

and fulfil the following constraints:
(a) stress constraint

$$\sigma_{\max} = \frac{M}{W} \le f_{y1} \quad \text{or} \quad W \ge \frac{M}{f_{y1}} = W_0 \tag{38}$$

$$I = \frac{h_s^3 t_s}{6} + 2A_f \left(\frac{h_s}{2}\right)^2 ; W = \frac{I}{h_s/2} = \frac{h_s^2 t_s}{3} + A_f h_s \tag{39}$$

The bending moment is expressed as

$$M = p_s L^2 / 8 , \tag{40}$$

the average width of the sectorial plate is 5000f_ω, thus the intensity of the uniformly distributed normal load is 7.5x5000 f_ω = 37.5 f_ω. Furthermore the self mass of the sectorial plate is also taken into account, so

$$p_s = 37.5 f_\omega + 1.1\left(\frac{\rho_1 V_2}{L} + 2\rho_1 A_f\right), \ \rho_1 = 7.85\text{x}10^{-5}\ \text{N/mm}^3. \tag{41}$$

Since A_f is not known an iteration is needed.
The floor is supported at the centre by a column, thus the span length of a radial beam is L=9500 mm.
(b) deflection constraint

$$w_{\max} = \frac{C_w}{I} \leq w_{adm} = \frac{L}{\phi}; \quad C_w = \frac{5 p_d L^4}{384E}; \quad \phi = 300 \tag{42}$$

or

$$I \geq I_0 = \frac{5 p_d L^4}{384 E w_{adm}} \tag{43}$$

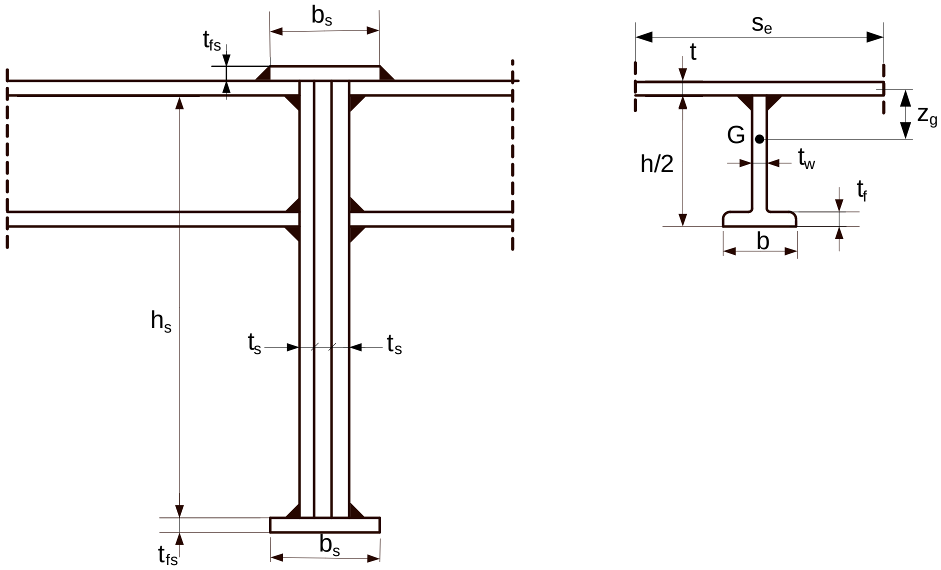

Figure 4: Radial beam of box section and the connected stiffened sectorial plates

For the deflection constraint the load intensity is calculated without safety factors, thus

$$p_d = 2.5 f_\omega + \frac{\rho_1 V_2}{L} + 2\rho_1 A_f \tag{44}$$

(c) constraint on local buckling of webs

$$\frac{h_s}{t_s} \leq \frac{1}{\beta}; \text{ or } \quad t_s \geq \beta h_s \tag{45}$$

where $\quad 1/\beta = 69\varepsilon; \varepsilon = \sqrt{\frac{235}{f_y}}$ (46)

Considering the local buckling constraint as active the stress constraint can be written as

$$W = \frac{\beta h_s^3}{3} + A_f h_s \geq W_0 \tag{47}$$

substituting A_f from Eq(37) into one obtains

$$A = \frac{2W_0}{h_s} + \frac{4\beta h_s^2}{3} \tag{48}$$

From the condition

$$\frac{dA}{dh_s} = 0 \tag{49}$$

one obtains the optimum value of h_s from the stress constraint

$$h_{s\sigma} = \sqrt[3]{\frac{3W_0}{4\beta}} \tag{50}$$

Similarly from deflection constraint

$$h_{sw} = \sqrt[4]{\frac{3I_0}{\beta}}; I_0 = \frac{\phi C_w}{L} \quad (51)$$

and $2A_{fw} = \sqrt{\frac{4\beta I_0}{3}}$ (52)

The optimum dimensions of the radial beams are summarized in Table 5. (Figure 3).

Table 5: Optimum dimensions of the radial beams

ω	webs h_sxt_s	flanges
8	530x8	170x8
12	475x7	160x7
16	450x7	160x6

8. Optimum number of sectorial plates

Calculation of the total cost of the floor structure.

The additional cost of the welding of radial beam flanges with double fillet welds of size a_{wf}=5 mm

$$K_{add} = k_w \left(\Theta_2 \sqrt{2\omega\rho V_{add}} + 1.3x0.3394x10^{-3} a_{wf}^2 L_{wf} \right) \quad (53)$$

where

$$V_{add} = 2A_f \omega L \quad (54)$$

and the weld length is

$$L_{wf} = 4\omega L \quad (55)$$

The additional painting cost is

$$K_{padd} = k_p 4b_s \omega L \quad (56)$$

The total cost is

$$K_{total} = \omega K + K_{add} + K_{padd} \quad (57)$$

The costs for different numbers of sectorial plates are summarized is Table 6.

Table 6: Costs in $ of different numbers of sectorial plates

ω	K	ωK	K_{add}	K_{padd}	K_{total}
8	7141	57128	3837	1488	62450
12	4567	54804	3877	2181	60790
16	3636	58176	7519	2802	66770

The corresponding masses are summarized in Table 7.

Table 7: Masses in kg for different numbers of sectorial plates

ω	G	ωG	G_{add}	G_{total}
8	2684	21472	1623	23095
12	1604	19248	2005	21253
16	1281	20496	2290	22786

It can be seen that the optimum number of sectorial plates is 12, which gives the minimum total cost and minimum total mass of the floor.

9. Cost comparison with an unstiffened thick-base-plate version

In order to show the cost difference between stiffened thin plate and unstiffened thick plate version we calculate the total cost for a structural version in which the sectorial plates are unstiffened and the radial beams are of rolled Í-section.

Timoshenko and Woinowsky-Krieger [4] have given formulae for bending of sectorial plates. We use the bending moment for angle of $\pi/4$, which corresponds to ω = 8. The maximum bending moment is

$$M = 0.0183 p_M R^2 \quad (58)$$

From the stress constraint

$$\sigma_{max} = \frac{6M}{t^2} \le f_{y1} \quad (59)$$

the required thickness of an unstiffened sectorial plate is

$$t \ge \sqrt{\frac{6M}{f_{y1}}} \quad (60)$$

For our case t = 19.3 rounded 20 mm.
The required moment of inertia of a radial beam from the deflection constraint (Eq 43)
I_o = 43.385x10^7 mm^4.
In Eq (43) the value of p_o should be calculated taking into account the self mass of the sectorial plate

$$V_p = \frac{R^2 \pi t}{8} = 7.854x10^8 \text{ mm}^3,$$

$$p_0 = 25 f_\omega + \frac{\rho_1 V_p}{L} = 27.2 \text{ N/mm.}$$

We select for radial beams a rolled I-profile of UB457x191x98 with I = 45.73x10^7 mm^4.
Specific self mass of a radial beam is G = 98.3 kg/m.
Material cost of the whole base plate

$$K_{Mplate} = k_M \rho R^2 \pi t = 49323 \text{ \$.}$$

Material cost of the radial beams is

$$K_{rad} = k_M 8GL = 7471 \text{ \$.}$$

Cost of welding of a sectorial plate, using SAW butt welds to connect 7 plate strips, the calculated weld length is 23 m

$$K_{wplate} = k_w \left(2\sqrt{7x49323} + 1.3x0.1033x20^{1.9}x23\right) = 2090 \text{ \$.}$$

Cost of welding of the sectorial plates to the radial beams with SAW double fillet welds of size 10 mm

$$K_{w1} = k_w \left(2\sqrt{8x49323} + 1.3x0.2349x10^2 2x8x9.5\right) = 5898 \text{ \$.}$$

Cost of painting

$$K_{p1} = k_p \left(2R^2\pi + 8(2x428 + 4x192.8)\right) = 21657 \text{ \$.}$$

Total cost is

$$K_{unstiff} = K_{Mplate} + K_{rad} + 8K_{wplate} + K_{w1} + K_{p1} = 101069 \text{ \$.}$$

The cost difference between the unstiffened and stiffened circular floor structure is
(101069-62210)/101069x100=38%, thus it can be concluded that the stiffening is very cost effective.

10. Conclusions

A large-diameter circular floor structure is optimized. This welded steel structure consists of sectorial stiffened plates and radial beams. The floor is supported by a circumferential beam or wall and a column at the centre.
The optimization procedure is a threefold process as follows: (a) optimum stiffening is sought for a trapezoid-like sectorial plate, (b) optimum dimensions of welded box radial beams are determined, (c) optimum number of sectorial plates is determined.
The optima are determined by cost comparisons. A cost calculation method is developed and applied. The cost function consists of costs of material, assembly, welding and painting.
The sectorial plates can be stiffened by non-equidistant tangential stiffeners or by a combination of equidistant tangential and radial stiffeners. The distances of non-equidistant tangential stiffeners are calculated using an algorithm, which considers the condition that all the base plate parts should be fully stressed from bending moments.
The costs of various stiffenings show that the non-equidistant tangential stiffening and the base plate thickness of 4 mm give the minimum cost solution.
The optimization of the radial beams is performed by the minimization of the cross-section area of the welded box profile with the design constraints of stress, deflection and local web buckling.
The total costs of the floor structure calculated for different numbers of sectorial plates show that the number of 12 gives the minimum total cost.
The stiffened structure is 38% cheaper than the unstiffened one, since the base plate thickness is 4 mm instead of 20 mm.

11. Acknowledgements

The authors gratefully acknowledge the support of the Hungarian Scientific Research Fund under the OTKA 75678 project number.

12. References

[1] J. Farkas. and K. Jármai, *Analysis and optimum design of metal structures*, Rotterdam: Balkema, 1997.
[2] J. Farkas. and K. Jármai, *Economic design of metal structures*, Rotterdam: Millpress, 2003.
[3] J. Farkas. and K. Jármai, *Design and optimization of metal structures*, Chichester: Horwood Publishing Ltd., 2008.
[4] S. Timoshenko. and S. Woinowsky-Krieger,. *Theory of plates and shells*, New York-Toronto-London: McGraw Hill, 1959.
[5] Det Norske Veritas (DNV) *Buckling strength analysis.* Classification Notes No.30.1. Høvik, Norway, 1995. DNV *Buckling strength of plates structures.* Recommended practice DNV-RP.C201. Høvik, Norway, 2002.
[6] Eurocode 3. *Design of steel structures.* Part 1-1. General structural rules. 2002.
[7] J.K. Paik and A.K. Thayamballi, *Ultimate limit state design of steel-plates structures.* Chichester, UK. John Wiley and Sons Ltd. 2003.