International Institute of Welding
Hungarian & German Delegations

Institut International de la Soudure
Délégations Hongroise & Allemande

Optimum design of welded cellular plates for ship deck panels

by

Jármai,K.*, Farkas,J.*, Petershagen,H.**

* Prof., University of Miskolc, Hungary

**Prof., Technical University Hamburg-Harburg,
(formerly Institute of Naval Architecture) Germany

IIW-Doc. XV-987-98, XV-F-67-98

51th IIW Annual Assembly
Hamburg, Germany

1998

Abstract: The investigated cellular plates consist of two face sheets and some longitudinal ribs of square hollow section (SHS) welded between them using arc-spot welding technology. The cellular deck panels are subject to axial compression and a transverse load causing bending. In the optimization procedure the dimensions and number of longitudinal SHS ribs as well as the thickness of face sheets are sought which minimize the cost function and fulfil the design constraints. The width and length of the three-span panel are known. The cost function contains the material and fabrication cost. The design constraints relate to the stress due to compression and bending and to the eigenfrequency of the structure. A computer program of Rosenbrock's Hillclimb mathematical programming method is used for optimization.

Keywords: structural optimization, minimum cost design, fabrication cost, arc-spot welding, cellular plates, structural stability, eigenfrequency of beams, welded structures

1. Introduction

Cellular plates consist of two face sheets and a grid of ribs welded between them. The main advantage of such plate structure is that the cells have a large torsional stiffness, which allows designers to construct plates of small height. The disadvantage of cellular plates lies in fabrication difficulty, since, when the height is smaller than 800-1000 mm, it is impossible to weld the ribs to the face sheets from inside.

Some applications of cellular plates are as follows: double bottoms of ships, rudders of ships, floating roofs of cylindrical storage tanks, box gates for dry docks, wings of aircraft structures, bridge decks, floating bridges, offshore platforms, elements of machine tool structures (press tables, mounting desks, base plates), mining shields, floors of buildings, lightweight roofs, etc.

Design problems of cellular plates have been treated in a number of articles. A brief literature survey and the minimum cost design procedure can be found in the book of Farkas and Jármai (1997). From theoretical and experimental studies we mention here the works of Williams (1969), Farkas (1976), Pettersen (1979), Farkas (1982), Evans and Shanmugam (1984), Shanmugam and Balendra (1986), Farkas (1992), Farkas and Jármai (1994).

Regarding the fabrication of cellular plates there are more possibilities to join the ribs to the face sheets. The simplest but not the cheapest solution is to use face plate elements and weld them to ribs from outside by fillet welds. Special welds such as arc-spot welds, slot or plug welds as well as electron-beam or laser welds can be used without cutting larger face sheet parts. A combination of fillet and arc-spot welds is shown in Fig.1.

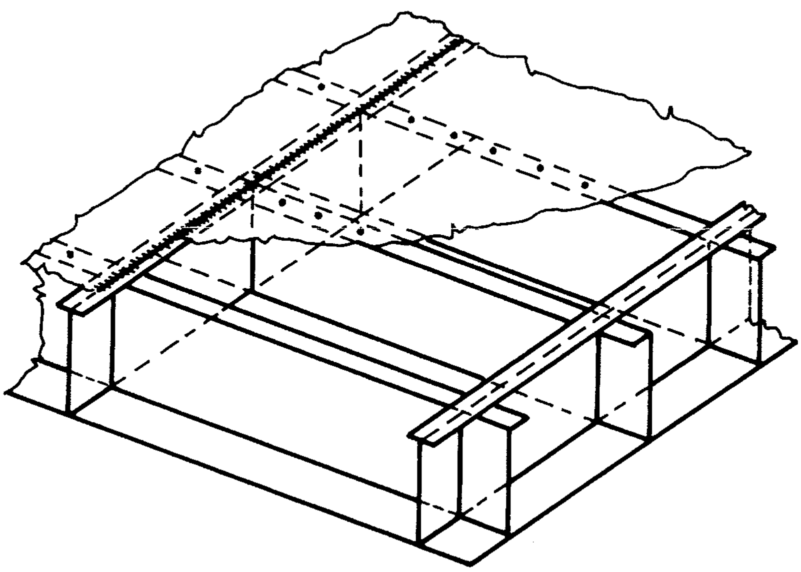

Fig.1. Cellular plate with a combination of fillet and arc-spot welds

It should be mentioned that, in the case of closed cellular structures, some problems can arise as follows: to avoid the overload caused by an occasional change of air pressure, some holes can be used, but when using holes, the wet air can cause internal corrosion. Holes are dangerous, since water can flow in and freeze.

Suruga and Maeda (1976) have proposed a special cellular plate construction for bridge decks (Fig.2), but this solution is too expensive.

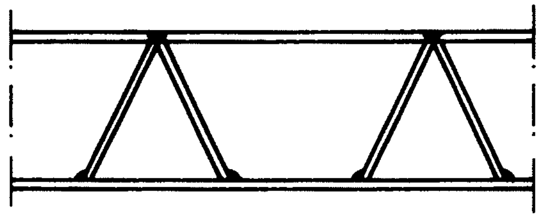

Fig.2. A special cellular plate with longitudinal stiffeners proposed by Suruga and Maeda (1976)

An interesting application of cellular plates for ship deck panels has been proposed by the third author. The main specialties of this application are as follows: 1) only longitudinal ribs of square hollow section (SHS) are used joined to the face sheets by arc-spot welding, thus, in the cost function the fabrication cost of arc-spot welds should be included; 2) to avoid the vibration resonance, the first eigenfrequency of the plate should be larger than a prescribed value.

The aim of present study is to work out a minimum cost design of such cellular plates considering, besides of stress constraint, the eigenfrequency constraint as well, and the fabrication cost of arc-spot welds.

2. The cost function

The cross-section of the deck panel is shown in Fig.3. The cellular plate consists of two face sheets of thickness t_f and longitudinal SHS ribs of number n with dimensions of b and t.

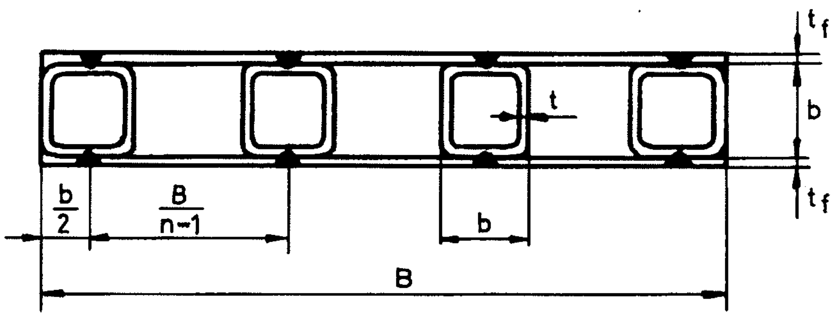

Fig.3. Cross-section of the ship deck panel investigated in present study

In the longitudinal direction the plate ends are clamped and the panel is supported in two places, thus, it can be calculated as a three-span beam (Fig.4) loaded axially with a compression stress $\sigma = N / A_{eff}$, A_{eff} being the effective cross-section for compression (Fig.6), and transversely by a uniformly distributed normal load of a factored intensity p.

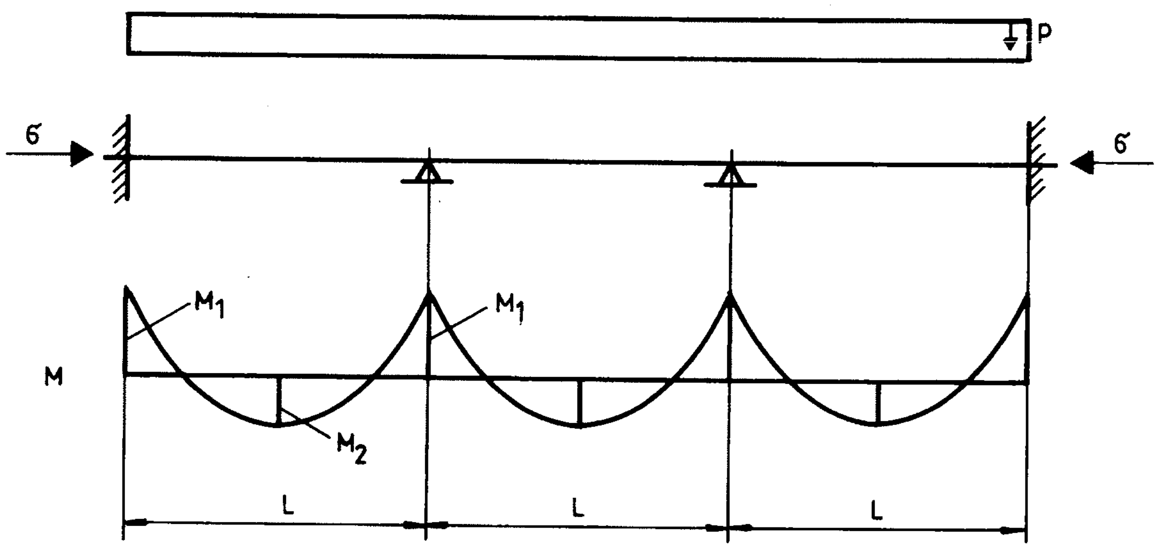

Fig.4. Bending moment diagram of the ship deck panel

The cost of a welded structure consists of material and fabrication cost

$$K = K_m + K_f = k_m \rho V + k_f \sum_i T_i \qquad (1)$$

where k_m and k_f are the material and fabrication cost factors, ρ is the material density, V is the volume of the structure, T_i are the times of fabrication phases. It is advantageous to write (1) in the form of

$$\frac{K}{k_m} = \rho V + \frac{k_f}{k_m} \sum_i T_i \qquad (2)$$

where k_f/k_m = 0 - 2 kg/min, since k_m = 0.5 - 1.0 \$/kg and k_f = 0 - 60 \$/h = 0 - 1 \$/min, with a change of this ratio the results may be applied internationally. The volume of the structure is

$$V = 3L(nA_{SHS} + 2Bt_f) \qquad (3)$$

The cross-sectional area of a SHS is, considering the corner roundings according to a formula given by DASt (1986), approximately

$$A_{SHS} = 0.99 * 4(b-t)t\left(1 - 0.43\frac{t}{b-3t}\right) \qquad (4)$$

The fabrication times are as follows. The time of preparation, assembly and tacking can be expressed as

$$T_1 = C_1 \Theta_d (\kappa \rho V)^{0.5} \tag{5}$$

where C_1 = 1.0 min/kg$^{0.5}$, $\Theta_d = 2-4$ is the difficulty factor expressing the effect of type of structure (planar or spatial), κ is the number of assembled structural elements, in our case it is $\kappa = n+2$.

The time of arc-spot welding is given by

$$T_2 = n_S T_S \tag{6}$$

where n_S is the number of spots, T_S is the time of welding of one spot weld and of the electrode transfer to the next spot.

The additional time for deslagging, chipping and changing the electrode can be calculated as

$$T_3 = 0.3\ T_2 \tag{7}$$

Since data for T_S cannot be found in literature, we take T_S = 0.3 min noting that it depends on the welding equipment and the degree of automation.

The number of spots can be calculated by means of the spot pitch a. The required minimum spot pitch can be determined considering a spot weld as a pin (Blodgett 1978, Füchsel et al.. 1990) .

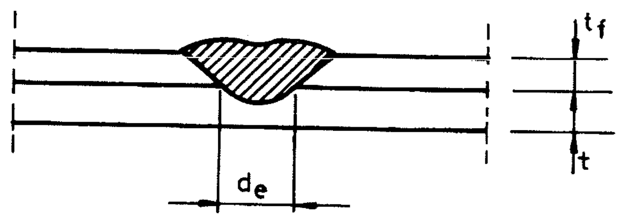

Fig.5. Effective diameter of an arc-spot weld

Limiting forces for a pin, according to Eurocode 3 (EC3) (1992) are as follows:

for bearing $$F_b = 1.5 t_f d_e f_y / \gamma_{Mp} \tag{8}$$

with $d_e = 2t_f$ (Fig.5) and $\gamma_{Mp} = 1.25$ $\quad F_b = 2.4 t_f^2 f_y$;

for shear $$F_Q = 0.6t \frac{\pi d_e^2}{4} f_u / \gamma_{Mp} = 1.508 t_f^2 t f_u \tag{9}$$

For steel Fe 360 the ultimate strength is f_u = 360 and the yield stress f_y = 235 MPa, for steel Fe 510 is f_u = 510 and f_y = 355 MPa.

A spot weld is loaded by a force F_W from the shear acting in a bent beam

$$F_W = \frac{Q S_\xi}{I_\xi} a \tag{10}$$

where Q is the shear force, S_ξ and I_ξ are the static moment and moment of inertia of an effective cross-section as shown in Fig. 7 and given in (34a,b), respectively, a is the spot pitch.

From the condition $F_W \le F_{b,Q}$ one obtains the required maximum spot pitch

$$a_{\max} = \frac{F_{b,Q} I_\xi}{Q S_\xi} \qquad \text{but} \quad a_{max} \le 50\, t_f \tag{11}$$

The number of spots in (6) can be expressed as

$$n_S = 6nL/a \tag{12}$$

3. The design constraints

3.1 *Constraint on eigenfrequency*

A serviceability constraint can be defined expressing that the first eigenfrequency of a simply supported bent beam of span length L should be larger than a prescribed value

$$f_1(Hz) = \frac{\pi}{2L^2}\left(\frac{10^3 EI_x}{m}\right)^{1/2} \ge f_0 \tag{13}$$

where E is the modulus of elasticity, the moment of inertia of the whole cross-section is

$$I_x = nI_{SHS} + Bt_f(b+t_f)^2/2 \tag{14}$$

According to DASt (1986) the moment of inertia of a SHS is approximately

$$I_{SHS} = \frac{2}{3}(b-t)^3 t\left(1 - 0.86\frac{t}{b-3t}\right) \tag{15}$$

In the mass m an additive mass m_{add} should be considered, thus

$$m = \rho\left(nA_{SHS} + 2Bt_f\right) + m_{add} \tag{16}$$

It should be mentioned that f_1 is in reality larger than it is calculated with formula (13) because the beam is clamped and not simply supported, but this approximation can be used since this constraint is not active.

3.2 *Constraint on stability due to compression and bending*

According to EC3, the stress constraint should be defined for a section of class 4 as follows:

$$\frac{N}{\chi A_{eff} f_{y1}} + \frac{k_x \psi M_1}{W_\xi f_{y1}} \le 1 \qquad f_{y1} = f_y/\gamma_{M1};\ \gamma_{M1} = 1.1 \tag{17}$$

where χ is the overall buckling factor

$$\chi = \frac{1}{\phi + \left(\phi^2 - \bar{\lambda}^2\right)^{1/2}} \tag{18}$$

$$\phi = 0.5\left[1 + 0.34\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right] \tag{19}$$

$$\bar{\lambda} = \frac{KL}{\lambda_1 r} \beta_A^{1/2} \tag{20}$$

for a beam with clamped ends $K = 0.5$,

$$\lambda_1 = \pi (E / f_y)^{1/2} \beta_A^{1/2} ; \qquad r = (I_{eff} / A_{eff})^{1/2} \tag{21}$$

$$\beta_A = \frac{A_{eff}}{nA_{SHS} + 2Bt_f} \tag{22}$$

To obtain the effective cross-section, the effective width of face sheets should be calculated according to EC3

$$b_e = \rho_p \frac{B}{n-1} \tag{23}$$

$$\bar{\lambda}_p = \frac{B / [(n-1) t_f]}{28.4 \varepsilon k_\sigma^{1/2}} , \qquad \varepsilon = (235 / f_y)^{1/2} \tag{24}$$

with $k_\sigma = 4$ $\qquad \bar{\lambda}_p = \dfrac{B}{56.8\varepsilon(n-1)t_f}$ (25)

when $\bar{\lambda}_p \le 0.673$ $\qquad \rho_p = 1$ (26a)

when $\bar{\lambda}_p \ge 0.673$ $\qquad \rho_p = \dfrac{1}{\bar{\lambda}_p} - \dfrac{0.22}{\bar{\lambda}_p^2}$ (26b)

Considering the effective cross-section shown in Fig.6 we get

$$A_{eff} = nA_{SHS} + 2B_e t_f , \qquad B_e = b + (n-1) b_e \tag{27}$$

and

$$I_{eff} = nI_{SHS} + B_e t_f (b + t_f)^2 / 2 \tag{28}$$

According to the moment diagram shown in Fig.2

$$M_1 = BpL^2 / 12 \tag{29}$$

this bending moment should by multiplied by a dynamic factor ψ .

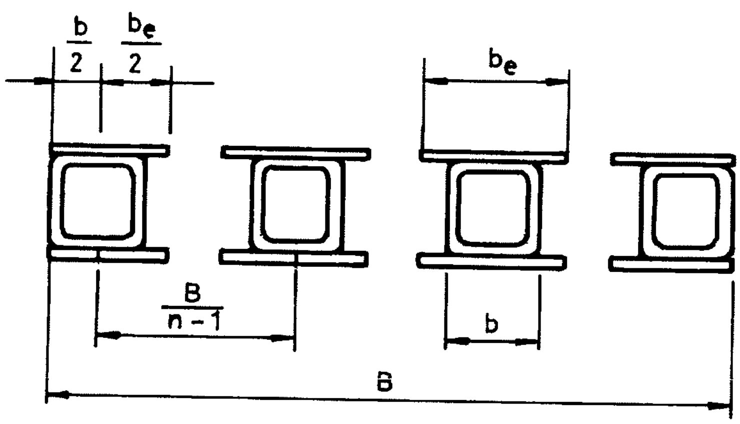

Fig.6. Effective cross-section for compression

$$k_x = 1 - \frac{\mu_x N}{\chi\left(nA_{SHS} + 2Bt_f\right)f_y} \qquad \text{but} \quad k_x \le 1.5 \tag{30}$$

$$\mu_x = \overline{\lambda}\left(2\beta_M - 4\right) \qquad \text{but} \quad \mu_x \le 0.9 \tag{31}$$

For our case it is $\beta_M = 1.3$ and $\mu_x = -1.4\overline{\lambda}$, thus

$$k_x = 1 + \frac{1.4\overline{\lambda}\beta_A N}{\chi f_y A_{eff}} \tag{32}$$

For bending another asymmetric effective cross-section should be taken into account as shown in Fig.7. The distance of gravity centre G is

$$\eta_G = \frac{nA_{SHS}\left(b + t_f\right)/2 + B_e t_f\left(b + t_f\right)}{nA_{SHS} + B - 2nt_f + B_e t_f} \tag{33}$$

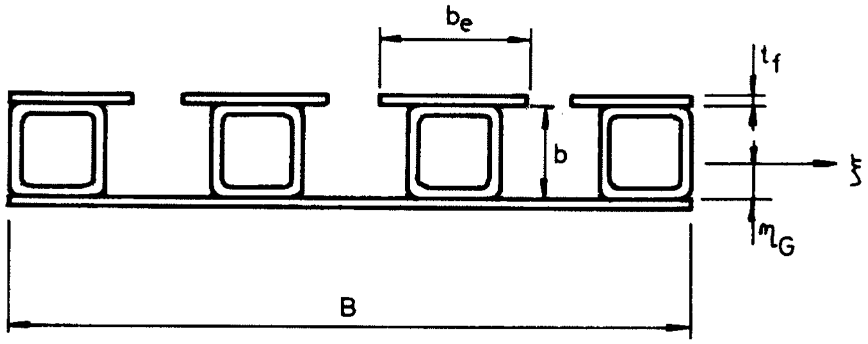

Fig.7. Effective cross-section for bending

The deduction of holes caused by arc-spot welds is considered for face sheet subject to tension by taking B - $2nt_f$ instead of B.

The moment of inertia is given by

$$I_\xi = nI_{SHS} + nA_{SHS}\left(\frac{b + t_f}{2} - \eta_G\right)^2 + (B - 2nt_f)t_f\eta_G + B_e t_f\left(b + t_f - \eta_G\right)^2 \tag{34a}$$

the static moment for the calculation of (10) and (11) is

$$S_\xi = b_e\left(b + t_f - \eta_G\right) \tag{34b}$$

and the section modulus is

$$W_\xi = \frac{I_\xi}{b + t_f - \eta_G} \tag{35}$$

3.3 *Stress constraint for the upper face sheet*

The upper face sheet is subject to bending and compression in longitudinal direction as well as to bending in transverse direction. The stress due to longitudinal compression and bending is

$$\sigma_L = \frac{N}{A_{eff}} + \frac{\psi M_1}{W_\xi} \quad (36)$$

and the stress due to transverse bending, considering a plate strip with clamped edges of span length $B/(n-1) - b$, is

$$\sigma_T = \frac{\psi p}{2t_f^2}\left(\frac{B}{n-1} - b\right)^2 \quad (37)$$

The stress constraint can be expressed as

$$\left(\sigma_L^2 + \sigma_T^2 + \sigma_L \sigma_T\right)^{0.5} \le f_{y1} \quad (38)$$

4. The optimization procedure

In the minimum cost design the optimum values of b, t, t_f and n are sought, which minimize the cost function (2) and fulfil the design constraints (13), (17) and (38). In the first phase the above mentioned variables are treated as continuous ones and the optima are determined using the Rosenbrock's hillclimb mathematical programming method. In the second phase the discrete values of variables are calculated using a complementary search method. In this search the minimum values are taken as

$b_{min} = 30$, $t_{min} = 2$, $t_{fmin} = 2$ mm and $n_{min} = 4$.

The discrete values of SHS are sought according to the pre-standard prEN 10219-2 (1992).

Note that the minimum number of ribs $n_{min} = 4$ has been selected, since the transverse stiffness of the panel is in the case of $n = 3$ too small. In the case when the normal loading is not uniformly distributed in transverse direction, it would be necessary to use transverse ribs as well to avoid too large torsional deformations.

The numerical data are as follows: $f_0 = 18$ Hz, $E = 2.1*10^5$ MPa, $B = 2000$, $L = 2250$ mm, $\Theta_d = 3$, $\rho = 7850$ kg/m^3 = $7.85*10^{-6}$ kg/mm^3, $m_{add} = 2*50 = 100$ kg/m = 0.1 kg/mm, $p = 3.5$ kN/m^2 = $3.5*10^{-3}$ N/mm^2, $\psi = 1.4$, $\sigma = N / A_{eff} = 150$ MPa.

The computational results are summarized in Tables 1 and 2.

Table 1. Optimization results for $f_y = 235$ MPa: number of ribs n, optimum dimensions in mm and K/k_m-values in kg for cost in function of the ratio k_f/k_m

k_f/k_m	n	t_f	b	t	K/k_m
	4	8	120	3	1987
	5	4	120	3	1212
	6	4	30	2	916
0	7	2.5	40	2	639
	8	2	50	2	582
	9	2	50	2	602
	10	2	50	2	621
	4	8	120	3	2367
	5	4	120	3	1620
	6	4	30	2	1331
1	7	2.5	40	2	1161
	8	2	50	2	1232
	9	2.5	40	2	1306
	4	8	120	3	2747
	5	4	120	3	2027
	6	4	30	2	1745
2	7	2.5	40	2	1683
	8	2.5	40	2	1813
	9	2.5	40	2	1943

5. Conclusions

The optimum number of ribs is larger for minimum weight design ($k_f/k_m = 0$) i.e. $n = 8$ for $f_y = 235$ and $n = 6$ for $f_y = 355$ MPa, than for minimum cost design ($k_f/k_m = 1$ or 2) i.e. $n = 7$ for $f_y = 235$ and $n = 6$ or 5 for $f_y = 355$ MPa.

The optimum number of ribs depends on f_y.

Cost savings of 14-18% can be achieved using steel of yield stress 355 instead of 235 MPa.

The cost difference between the best and worst solution for $f_y = 235$ MPa and $k_f/k_m = 2$ is 100(2747-1683)/1683 = 63%, which emphasizes the importance of structural optimization.

Calculations show that the stability and stress constraints are in most cases active and the eigenfrequency constraint is passive.

Table 2. Optimization results for $f_y = 355$ MPa: number of ribs n, optimum dimensions in mm, and K/k_m-values in kg for cost in function of the ratio k_f/k_m

k_f/k_m	n	t_f	b	t	K/k_m
	4	5	30	2	1105
	5	2.5	50	2	629
0	6	2	40	2	517
	7	2	40	2	533
	8	2	40	2	548
	4	5	30	2	1434
1	5	2.5	40	2	1014
	6	2	40	2	1026
	4	5	50	2	1803
2	5	2.5	40	2	1420
	6	2	40	2	1535
	7	2.5	40	2	1683

References

Blodgett,O.W. 1978. Report on proposed standards for sheet steel structural welding. Weld.J. Vol.57, April, pp.15-24.

DASt (Deutscher Ausschuss für Stahlbau) 1986. Richtlinie 016. Bemessung und konstruktive Gestaltung von Tragwerken aus dünnwandigen kaltgeformten Bauteilen. Köln.

Eurocode 3: Design of steel structures. Part 1.1 General rules and rules for buildings. European Committee for Standardization, Brussels, 1992.

Evans,H.R. and Shanmugam,N.E. 1984. Simplified analysis for cellular structures. J. Struct. Eng. ASCE 110, pp.531-543.

Farkas,J. 1976. Structural synthesis of welded cell-type plates. Acta Techn. Hung. 83, pp.117-131.

Farkas,J. 1982. Minimum cost design of welded square cellular plates. Publ. of Techn. Univ. for Heavy Ind., Miskolc, Ser. C. Machinery, Vol.37. Fasc. 1. pp.111-130.

Farkas,J. 1992. Cost comparisons of plates stiffened on one side and cellular plates. Welding in the World 30, No.5-6. pp.132-137.

Farkas,J. and Jármai,K. 1994. Minimum cost design of laterally loaded welded rectangular cellular plates. Structural Optimization 8, No.4.pp.262-267.

Farkas,J. and Jármai,K. 1997. Analysis and optimum design of metal structures. Rotterdam, Brookfield, Balkema.

Füchsel,S., Möbius,W. and Steinert,G. 1990. Empfehlung zur Berechnung von MAG-Punktschweissverbindungen. ZIS-Report, Halle, 1, pp.31-36.

Pettersen,E. 1979. Analysis and design of cellular structures. University of Trondheim, Norwegian Institute of Technology.

prEN 10219-2: 1992. Cold formed structural hollow sections of non-alloy and fine grain structural steels. Part 2. Tolerances, dimensions and sectional properties. European Committee for Standardization, Brussels. German version DIN EN 10219 Teil 2. Entwurf. 1993.

Shanmugam,N.E. and Balendra,T. 1986. Free vibration of thin-walled multi-cell structures. Thin-Walled Struct. 4, 467-483.

Suruga,T. and Maeda,Y. 1976. Selection of hollow steel plate deck... 10th IABSE Congress, Tokyo, Final Report, pp.19-22.

Williams,D.G. 1969. Analysis of double plated grillage under in-plane and normal loading. Ph.D. thesis, University of London, Imperial College.

Acknowledgements

This work has been supported by the Hungarian Fund for Scientific Research grants OTKA 19003 and 22846.