eurosteel
coimbra 2002

MINIMUM COST DESIGN OF LONGITUDINALLY STIFFENED WELDED STEEL PLATES LOADED BY ECCENTRIC COMPRESSION

Jármai, K.[1], Farkas,J.[2] ,Simoes,L.C.[3] and Virág,.Z.[4]

ABSTRACT

The global buckling strength of welded stiffened plates is calculated according to Mikami and Niwa. This method takes into account the effect of residual welding stresses and deformations. Mikami and Niwa have proposed a decreasing factor, which considers the eccentricity of uniaxial compressive load. This eccentricity means the distance between the middle plane of the base plate to the centroid of a cross section, which includes the stiffener and the effective base plate part. In the optimization procedure the cost function is minimized taking into account the design constraints. These constraints are as follows: global buckling of the stiffened plate, local buckling of the base plate, local buckling of plate parts of stiffeners, limitation of the distortion caused by the shrinkage of welds. Trapezoidal stiffeners are investigated. The optimum dimensions and number of stiffeners are determined by entropy-based unconstrained optimization and by Rosenbrock`s hillclimb method.

Key Words: welded structures, structural optimization, stiffened plates, plate buckling

1. INTRODUCTION

Welded stiffened plates are widely used in various load-carrying structures, e.g. ships, bridges, bunkers, tank roofs, offshore structures, vehicles, etc. They are subject to various loadings, e.g. compression, bending, shear or combined load. The shape of plates can be square, rectangular, circular, trapezoidal, etc. They can be stiffened in one or two directions with stiffeners of flat, L, trapezoidal or other shape.

From these structural versions we select here rectangular plates loaded by eccentric compression and stiffened by trapezoidal ribs in the direction of the compressive load.

[1] Professor, Dr., University of Miskolc, Hungary
[2] Professor em. , Dr., University of Miskolc, Hungary
[3] Professor, Dr., University of Coimbra, Portugal
[4] PhD student, University of Miskolc, Hungary

It should be mentioned that we have worked out minimum cost design procedure of square and rectangular orthogonally stiffened and cellular plates loaded in bending [1], uniaxially compressed rectangular plates with flat and L-stiffeners [2], welded bridge decks with open- and closed-section stiffeners [3,4].

It is well known that the instability phenomena are significantly affected by initial imperfections and residual welding stresses. For instance, it has been shown that a compression strut designed using the classical Euler method can be 30% unsafe [1]. Thus, these effects should be considered in all stability calculations.

In [2] we have used the design rules of API [5]. Mikami and Niwa [6,7] have recently developed a calculation method for orthogonally stiffened uniaxially compressed rectangular plates taking into account the initial imperfections and residual welding stresses. Their formulae are based on experimental results. In their study a simple formula is given for eccentric compression, when the uniaxial compressive force acts in the plan of the base plate.

The aim of the present study is to apply the Mikami-Niwa method for the optimum design of plates loaded by eccentric compression (Fig.1). In the minimum cost design the characteristics of the optimal structural version are sought which minimize the cost function and fulfil the design constraints. In recent years we have developed a cost function containing the material and fabrication costs [1,8] and we have included in the design constraints also the quality requirement, which prescribes the allowable deformation caused by residual welding distortions [9,10].

These two important aspects in the design of welded structures are included in the present study as well. First the general formulae for the cost function and design constraints are treated, then the special calculation of trapezoidal stiffeners is described.

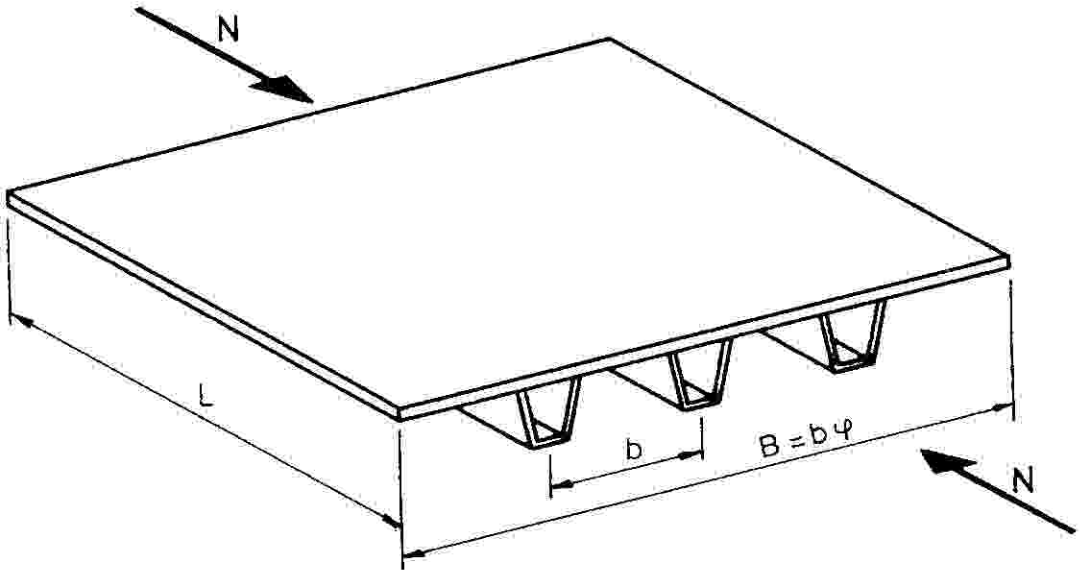

Fig.1. Longitudinally stiffened plate loaded by eccentric compression

2.COST FUNCTION

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

$$K = K_m + K_f = k_m \rho V + k_f \sum T_i \tag{1}$$

or in another form

$$\frac{K}{k_m} = \rho V + \frac{k_f}{k_m}(T_1 + T_2 + T_3) \tag{2}$$

where ρ is the material density, V is the volume of the structure, K_m and K_f as well as k_m and k_f are the material and fabrication costs as well as cost factors, respectively, T_i are the fabrication times as follows:

time for preparation, tacking and assembly

$$T_1 = \Theta_d \sqrt{\kappa \rho V} \tag{3}$$

where Θ_d is a difficulty factor expressing the complexity of the welded structure, κ is the number of structural parts to be assembled;

T_2 is time of welding, and T_3 is time of additional works such as changing of electrode, deslagging and chipping. $T_3 \approx 0.3T_2$, thus,

$$T_2 + T_3 = 1.3\sum C_{2i} a_{wi}^n L_{wi} \tag{4}$$

where L_{wi} is the length of welds, the values of $C_{2i} a_{wi}^n$ can be obtained from formulae or diagrams constructed using the COSTCOMP software [11,12], a_w is the weld dimension.

3.DESIGN CONSTRAINTS

3.1 Global buckling of the stiffened plate

According to Mikami and Niwa the effect of initial imperfections and residual welding stresses is considered by defining buckling curves for a reduced slenderness

$$\lambda = \left(f_y / \sigma_{cr}\right)^{1/2} \tag{5}$$

where σ_{cr} is the classical critical buckling stress, which does not contain the above mentioned effects, f_y is the yield stress.

The classical critical buckling stress for a uniaxially compressed longitudinally stiffened plate (Fig.1) is

$$\sigma_{cr} = \frac{\pi^2 D}{hB^2}\left(\frac{1+\gamma_S}{\alpha_R^2} + 2 + \alpha_R^2\right) \quad \text{for} \quad \alpha_R = L/B < \alpha_{R0} = \left(1+\gamma_S\right)^{1/4} \tag{6}$$

$$\sigma_{cr} = \frac{2\pi^2 D}{hB^2}\left[1 + \left(1+\gamma_S\right)^{1/2}\right] \quad \text{for} \quad \alpha_R \ge \alpha_{R0} \tag{7}$$

where, with $\nu = 0.3$

$$D = \frac{Et_F^3}{12\left(1-\nu^2\right)} = \frac{Et_F^3}{10.92} \tag{8}$$

$$h = t_F + \frac{A_S}{b}\ ; \qquad b = \frac{B}{\varphi} \tag{9}$$

A_S is the cross-sectional area of a stiffener, $\varphi - 1$ is the number of stiffeners,

$$\gamma_S = \frac{EI_S}{bD} \tag{10}$$

I_S is the moment of inertia of a stiffener about the ξ axis (Fig.3).

Knowing the reduced slenderness (Eq.5) the actual global buckling stress can be calculated as follows:

$$\sigma_U / f_y = 1 \quad \text{for} \quad \lambda \le 0.3 \tag{11a}$$

$$\sigma_U / f_y = 1 - 0.63(\lambda - 0.3) \quad \text{for} \quad 0.3 \le \lambda \le 1 \tag{11b}$$

$$\sigma_U / f_y = 1/\left(0.8+\lambda^2\right) \qquad \text{for} \qquad \lambda > 1 \tag{11c}$$

This buckling curve is shown in Fig.2. It can be seen that the used buckling curve contains the effect of initial imperfections $(a_0 \neq 0)$ and residual welding stresses $(\sigma_R \neq 0)$, therefore it gives much lower values that the classical critical buckling curve, which neglects these effects.
The global buckling constraint is defined by

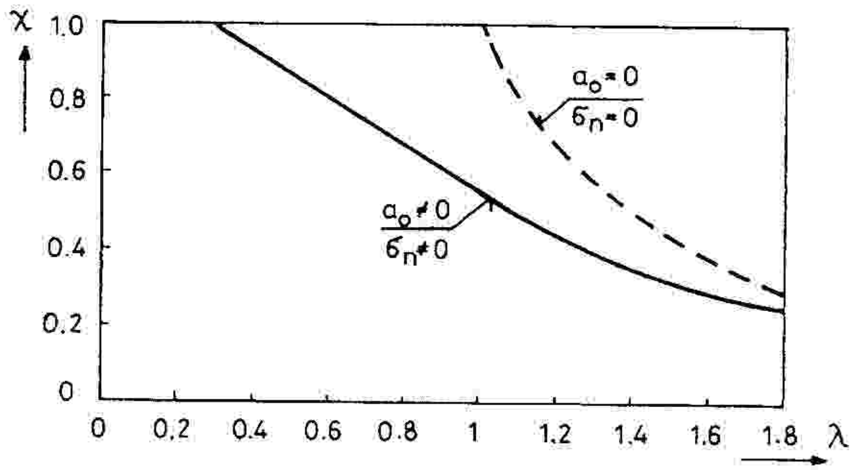

Fig.2. Global buckling curve considering the effect of initial imperfections $(a_0 \neq 0)$ and residual welding stresses $(\sigma_R \neq 0)$

$$\frac{N}{A} \leq \psi\sigma_U^* = \psi\sigma_U \frac{\rho_P + \delta_S}{1+\delta_S} \tag{12}$$

where

$$A = Bt_F + (\varphi - 1)A_S \tag{13}$$

and $$\delta_S = \frac{A_S}{bt_F} \tag{14}$$

ρ_P can be determined considering the single panel buckling of the base plate parts between the stiffeners. The factor $(\rho_P + \delta_S)/(1+\delta_S)$ expresses the effect of the effective width of the base plate parts.

The factor expressing the effect of load eccentricity is given by

$$\psi = \frac{1}{1+0.2 y_G c / r^2}, \qquad c = y_G + t_F/2 \tag{15}$$

where y_G is the distance from the centroid to the middle plane of the base plate, c is the distance between the centroid and the plate surface opposite to the longitudinal stiffener, r is the radius of gyration. A cross-section is considered, which includes the part of the base plate and one stiffener (Fig.3).

3.2 Single panel buckling

This constraint eliminates the local buckling of the base plate parts between the stiffeners. From the classical buckling formula for a simply supported uniformly compressed in one direction

$$\sigma_{crP} = \frac{4\pi^2 E}{10.92}\left(\frac{t_F}{b_1}\right)^2 \tag{16}$$

the reduced slenderness is

$$\lambda_P = \left(\frac{4\pi^2 E}{10.92 f_y}\right)^{1/2} \frac{b_1}{t_F} = \frac{b_1 / t_F}{56.8\varepsilon}; \qquad \varepsilon = \left(\frac{235}{f_y}\right)^{1/2} \tag{17}$$

where $b_1 = b - 300$ if $b - 300 \ge a_3$

$b_1 = a_3 = 300$ if $b - 300 \le a_3$,

and the actual local buckling stress considering the initial imperfections and residual welding stresses is

$$\sigma_{UP} / f_y = 1 \quad \text{for} \quad \lambda_P \le 0.526 \tag{18a}$$

$$\frac{\sigma_{UP}}{f_y} = \left(\frac{0.526}{\lambda_P}\right)^{0.7} \quad \text{for} \quad \lambda_P \ge 0.526 \tag{18b}$$

Then the factor ρ_P is as follows:

$$\rho_P = 1 \quad \text{if} \quad \sigma_{UP} > \sigma_U \tag{19a}$$

$$\rho_P = \sigma_{UP} / f_y \quad \text{if} \quad \sigma_{UP} \le \sigma_U \tag{19b}$$

3.3 Distortion constraint

In order to assure the quality of this type of welded structures large deflections due to weld shrinkage should be avoided. It has been shown that the curvature of a beam-like structure due to shrinkage of longitudinal welds can be calculated by relatively simple formulae [9]. The allowable residual deformations f_0 are prescribed by design rules. For compression struts Eurocode 3 (EC3) [13] prescribes $f_0 = L/1000$, thus the distortion constraint is defined as

$$f_{max} = CL^2 / 8 \le f_0 = L / 1000 \tag{20}$$

where the curvature is for steels

$$C = 0.844 x 10^{-3} Q_T y_T / I_x \tag{21}$$

Q_T is the heat input, y_T is the weld eccentricity

$$y_T = y_G - t_F / 2 \tag{22}$$

I_x is the moment of inertia of the cross-section containing a stiffener and the base plate strip of width b.

For GMAW-C (Gas Metal Arc Welding with CO_2) welded fillet welds

$$C_2 a_w^n = 0.3394 x 10^{-3} a_w^2 \qquad (L \text{ in mm}) \tag{23}$$

$a_W = 0.5 t_1$, but $a_{Wmin} = 4$ mm.

3.4 Data for trapezoidal stiffeners (Fig.3)

$$A_S = (a_1 + 2a_2)t_S; \qquad I_S = a_1 h_S^3 t_S + \frac{2}{3} a_2^3 t_S \sin^2\alpha \tag{24}$$

According to [14] $a_1 = 90$, $a_3 = 300$ mm, thus

$$h_S = \left(a_2^2 - 105^2\right)^{1/2}; \qquad \sin^2\alpha = 1 - \left(\frac{105}{a_2}\right)^2 \tag{25}$$

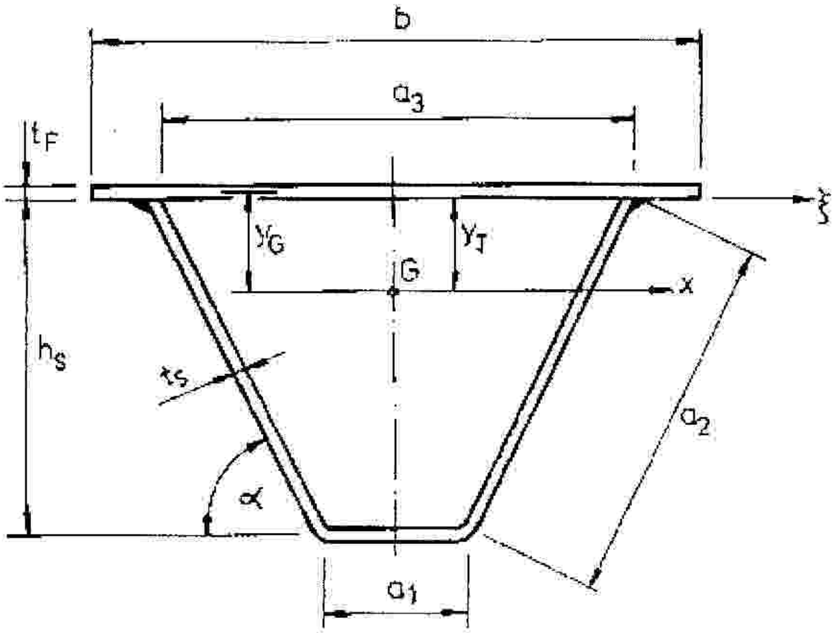

Fig.3. Dimensions of a trapezoidal stiffener

$$y_G = \frac{a_1 t_S (h_S + t_F/2) + 2a_2 t_S (h_S + t_F)/2}{bt_F + A_S} \tag{26}$$

$$I_x = \frac{bt_F^3}{12} + bt_F y_G^2 + a_1 t_S \left(h_S + \frac{t_F}{2} - y_G \right)^2 + \frac{1}{6} a_2^3 t_S \sin^2\alpha + 2a_2 t_S \left(\frac{h_S + t_F}{2} - y_G \right)^2 \tag{27}$$

$a_W = 0.5t_3$, but $a_{Wmin} = 4$ mm.

Local buckling of a trapezoidal stiffener is defined as

$$a_2 / t_S \leq 38\varepsilon \tag{28}$$

This constraint is treated as active.

Furthermore, the heat input for a stiffener is

$$Q_T = 2x59.5a_W^2 \tag{29}$$

4. NUMERICAL EXAMPLE

Given data: B = 6000 mm, L = 9000 mm, $N = 1.974 x 10^7$ [N], f_y = 235 MPa, E = $2.1 x 10^5$ MPa, $G = E/2.6$, $\rho = 7.85 x 10^{-6}$ kg/mm^3. The variables are as follows: φ, t_F and t_S.

The cost function is formulated according to the fabrication sequence. First the base plate is welded from plates of dimensions 6 mx1.5 m and 3 mx1.5 m with butt welds, then the stiffeners are welded to the base plate with fillet welds. The welding time for the first phase is

$$T_{W1} = 2\sqrt{8\rho BLt_F} + 1.3C_W a_W^n L_{W1} \tag{30}$$

where $L_{W1} = 3L + B$ and, for SAW (submerged arc welding) butt welds

for $t_F = 4 - 15$ mm $\quad C_W a_W^n = 0.1346 x 10^{-3} t_F^2$

for $t_F = 15 - 40$ mm $\quad C_W a_W^n = 0.1033 x 10^{-3} t_F^{1.904}$

The welding time of the second phase is

$$T_{W2} = 3\sqrt{\varphi[\rho BLt_F + \rho(\varphi-1)LA_S]} + 1.3x0.3394x10^{-3}a_W^2 L_{W2} \quad (31)$$

where $a_W = 0.5t_S$ but $a_{Wmin} = 4$ mm, $L_{W2} = 2(\varphi-1)L$.

The material cost is

$$K_M = k_M \rho[BLt_F + (\varphi-1)LA_S] \quad (32)$$

and the total cost is calculated as

$$\frac{K}{k_M} = \rho[BLt_F + (\varphi-1)LA_S] + \frac{k_F}{k_M}(T_{W1} + T_{W2}) \quad (33)$$

In order to give internationally usable solutions, we consider for material cost factor (steel) k_M = 0.5-1.2 \$/kg, for fabrication including overheads k_F = 15-45 \$/manhour = 0.25-0.75 \$/min. Thus, the ratio of k_F/k_M may vary in the range of 0-1.5 kg/min. The value of 0 corresponds to the minimum volume design.

We have used two different methods of optimization, such as entropy-based unconstrained optimization [15], and the Rosenbrock's hillclimb method [1]. Since these methods result in continuous optima, an additional discretization is used to find rounded discrete optimum values.

In the entropy-based minimization the simultaneous minimization of the cost and constraints is sought. All these goals are cast in a normalized form. The strategy adopted was an iterative sequence of explicit approximation models, formulated by taking Taylor series approximations of all the goals truncated after the linear term.

The hillclimb algorithm is a direct search method without derivatives [1]. In this iterative procedure an improvement can be made by lining the search directions in an orthogonal system.

These two methods have been effective for the present problem.

The results are summarized in Table 1. Optimum is denoted by bold numbers.

Table 1. Characteristics of the optimized structural versions in the function of stiffener thickness, as well as the fulfilling of design constraints

Thicknesses and B/b			K/k_M (kg)		Constraints	
t_S (mm)	t_F (mm)	φ	k_F/k_M=0	k_F/k_M=1.5	Eq.(12)	Eq.(20)
4	14	10	**6937**	12189	201<205	8.1<9
5	14	8	7096	11855	196<215	7.2<9
6	13	8	7131	11634	196<206	7.7<9
7	12	8	7240	11510	192<196	7.6<9
8	11	8	7424	**11483**	188<189	7.3<9
8	12	8	7848	12162	178<195	6.6<9

5. CONCLUSIONS

It can be seen from Table 1 that, to achieve a mass or cost minimum, the eccentricity should be decreased by increasing the faceplate thickness and by decreasing the stiffener dimensions. Since the eccentricity is decreased, the welding distortion is small, this constraint is passive.

The mass minimum can be achieved in this numerical example by taking t_S =4, t_F = 14 mm and the number of stiffeners 9. The cost minimum is given by t_S = 8, t_F = 11 and number of stiffeners 7 The mass and cost difference between the best and worst structural version indicated in Table 1 is 100(7848-6937)/6937 =13% and 100(12189-11483/11483 = 6%, thus the structural optimization enables to achieve significant mass and cost savings in the design stage.

ACKNOWLEDGEMENTS

The research work was supported by the Hungarian Scientific Research Foundation grants OTKA T38058, T37941 and T29326 and the Fund for the Development of Higher Education FKFP 8/2000 project. The project was also supported by the Hungarian-Portuguese Intergovernmental S&T Co-operation program. The Hungarian partner is the Ministry of Education, R&D Deputy Undersecretary of State, the Portuguese partner is the Portuguese Institute for Scientific and Technological Cooperation ICCTI and Oriente Foundation.

REFERENCES

[1] Farkas,J., Jármai,K.: Analysis and optimum design of metal structures. Balkema, Rotterdam-Brookfield, 1997.

[2] Farkas,J.,Jármai,K.: Economic design of welded steel structures. J. Constructional Steel Research 46(1998) Nos.1-3. 35-36. Full paper on CD-ROM. Paper No.142.

[3] Jármai,K., Horikawa,K., Farkas,J.: Economic design of steel bridge decks with open ribs. Transactions of JWRI (Joining and Welding Research Institute Osaka University) 26(1997)No.1 147-161.

[4] Jármai,K.,Farkas,J., Horikawa,K.: Economic design of steel bridge decks. Welding in the World 41(1998)No.1. 49-59.

[5] American Petroleum Institute API Bulletin on design of flat plate structures. Bulletin 2V. Washington, 1987.

[6] Mikami,I., Niwa,K. Ultimate compressive strength of orthogonally stiffened steel plates. J. Structural Engineering ASCE 122(1996)No.6. 674-682.

[7] Discussion of [6] by Bedair,O. and authors' closure. J.Struct.Engng 123(1997)No.8.1116-1119.

[8] Jármai,K., Farkas,J.: Cost calculation and optimization of welded steel structures. J. Construct.Steel Res. 50(1999)No.2. 115-135.

[9] Farkas,J.,Jármai,K.: Analysis of some methods for reducing beam curvatures due to weld shrinkage. Welding in the World 41(1998)No.4. 385-398.

[10] Farkas,J.,Jármai,K. Optimum design of a stiffened conical roof considering the residual welding distortions. Welding in the World 43(1999) No.5. pp.54-59.

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

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

[13] Eurocode 3. Design of steel structures. Part 1.1. General rules and rules for buildings. European Prestandard ENV 1993-1-1. CEN European Committee for Standardisation, Brussels, 1992.

[14] Stahlbau Handbuch Band 2. Köln, Stahlbau-Verlag, 1985.

[15] Simoes, L.M., Negrao, J.H.J.O. Optimization of cable-stayed bridges subjected to earthquakes with nonlinear behaviour. Eng. Opt. 31(1999), 457-478.