International Institute of Welding
Hungarian & Japanese Delegations

Institut International de la Soudure
Délégation Hongroise & Japonaise

Economic Design of Steel Bridge Decks

by

JÁRMAI, Károly *
HORIKAWA, Kohsuke **
FARKAS, József *

MISKOLCI EGYETEM 1735

* Prof., University of Miskolc, Hungary

** Prof., Joining and Welding Research Institute
Osaka University, Japan

IIW-Doc. XV-951-97, XV-F-52-97

50th IIW Annual Assembly
San Francisco, USA
1997

Abstract

In the structural synthesis the cost function is minimized considering design constraints on fatigue stress range of welded joints, local buckling, deflection and size limitations. In the cost function the material and fabrication costs are defined. The bending moments and the torsional stiffness of trapezoidal longitudinal stiffeners are calculated according to the Pelikan-Esslinger method, neglecting the flexibility of crossbeams. In fatigue constraints the published recent experimental research results are taken into account. A computer program of the Rosenbrock mathematical programming method is used to determine the following optimum dimensions: thickness of the deck plate, thicknesses and heights of stiffeners and crossbeams as well as the distance between crossbeams. The optimization procedure is illustrated by a numerical example.

Keywords

bridge decks, fatigue of welded joint, cost calculation, structural optimization, stiffened plates

Introduction

Stiffened steel decks form are important structural parts of bridges. Their welded joints can fail due to fatigue because of heavy traffic load. The fabrication cost form the main part of total cost, therefore it affects significantly the optimum dimensions of the deck structure. Although there are many size limitations, some dimensions can be optimized. These dimensions are as follows: thickness of the deck plate, height and thickness of stiffeners and crossbeams as well as the distance between crossbeams.

For the calculation of bending moments and the torsional stiffness of stiffeners the Pelikan-Esslinger method is used. The formulae are relatively complicate, thus a computer method should be applied. The Rosenbrock *Hillclimb* method was efficient for this purpose. The optimization procedure is illustrated by a numerical example. It should be noted that the optimum design of bridge decks with flat stiffeners has been treated in [1]. In the present paper the case of trapezoidal stiffeners is dealt with.

Assumptions

- The deck plate is simply supported around its periphery.
- The effect of the flexibility of crossbeams on bending moments in stiffeners is neglected.
- The main bridge girder and its connections to the deck structure are excluded from the calculations.
- The self-mass is neglected, since in the governing fatigue stress range constraint it has not to be considered.
- The spectrum factor in the calculation of fatigue stress range is taken as 1, the number of load cycles is $2*10^6$.

The live load considered for highway bridges

We select for our numerical example as live load trucks shown in Fig.1. According to DIN 1072 a truck has two wheel loads of 50 kN and two wheel loads of 30 kN. The widths of wheels are 400 and 260 mm respectively.

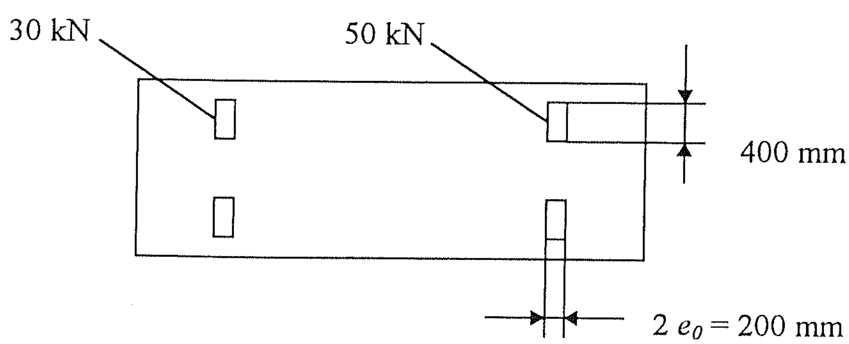

Fig.1. Data of trucks considered as live load

The bending and torsional stiffness of stiffeners

The dimensions of a stiffener section are given in Fig.2.

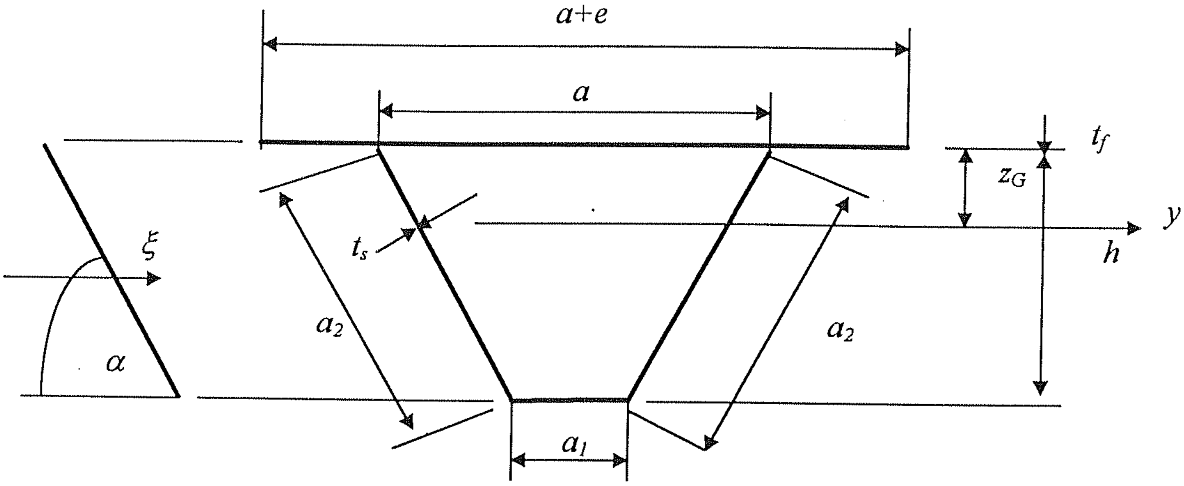

Fig.2. Dimensions of a stiffener section

The specific bending stiffness is defined by

$$B_y = \frac{I_y E}{a+e} \tag{1}$$

where the moment of inertia is given by

$$I_y = (a+e)t_f z_G^2 + a_1 t_s (h - z_G)^2 + I_\xi + 2a_2 t_s \left(\frac{h}{2} - z_G\right)^2 \tag{2}$$

with $$I_\xi = t_s \frac{a_2^3}{12} \sin^2 \alpha \tag{3}$$

and $$z_G = h t_s \frac{a_1 + a_2}{t_s(a_1 + 2a_2) + (a+e)t_f} \tag{4}$$

The specific torsional stiffness can be calculated by using the Pelikan-Esslinger formulae [2], which take into account the local deformations of trapezoidal stiffeners.

$$H = \frac{\mu GT}{2(a+e)} \tag{5}$$

where $$\frac{1}{\mu} = \frac{GT}{GT_{red}} \tag{6}$$

$$\frac{1}{\mu} = 1 + \frac{GT}{EI_0} \frac{a^3}{12(a+e)^2} \left(\frac{\pi}{t_2}\right)^2 \left[\left(\frac{e}{a}\right)^3 + \left(\frac{e-a_1}{a+a_1} + \lambda\right)^2 + \frac{\lambda^2}{\kappa_0}\left(\frac{a_1}{a}\right)^3 + \frac{24a_2}{\kappa_0 a}\left(c_1^2 + c_1 c_2 + \frac{c_2^2}{3}\right) \right] \tag{7}$$

T is the torsional stiffness for a closed thin-walled section [3,4].

$$T = \frac{4A_k^2}{\sum_i \frac{b_i}{t_i}} \tag{8}$$

$$t_2 = 0.81t \tag{9}$$

$$EI_0 = \frac{Et_f^3}{10.92} \tag{10}$$

$$A_k = h\frac{a + a_1}{2} \tag{11}$$

$$c_1 = \frac{\lambda}{2}\frac{a_1}{a} \tag{12}$$

$$c_2 = \left(\frac{\lambda}{2}\frac{a - a_1}{a}\right) - \left(\frac{a + e}{a + a_1}\frac{a_1}{2a}\right) \tag{13}$$

$$h = \sqrt{a_2^2 - \left(\frac{a - a_1}{2}\right)^2} \tag{14}$$

$$\lambda = \frac{(2a + a_1)(a + e)a_1 a_2 - \kappa_0 a^3 (e - a_1)}{(a + a_1)\left[2a_2\left(a^2 + aa_1 + a_1^2\right) + a_1^3 + \kappa_0 a^3\right]} \tag{15}$$

$$\kappa_0 = \left(\frac{t_s}{t_f}\right)^3 \tag{16}$$

The moment of inertia of crossbeams

The dimensions of a crossbeam section are shown in Fig.3.

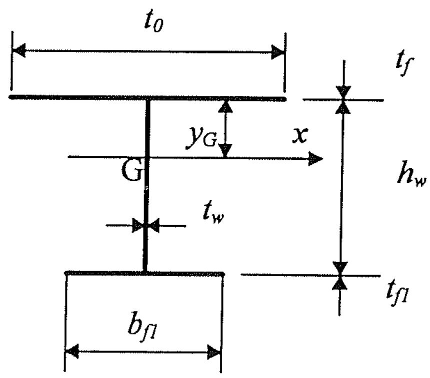

Fig.3. Dimensions of a crossbeam section

$$I_x = t_0 t_f y_G^2 + \frac{h_w^3 t_w}{12} + h_w t_w \left(\frac{h_w}{2} - y_G\right)^2 + b_{f1} t_{f1} (h_w - y_G)^2 \tag{17}$$

$$y_G = \frac{b_{f1} t_{f1} h_w + h_w t_w \dfrac{h_w}{2}}{t_0 t_f + b_{f1} t_{f1} + h_w t_w} \tag{18}$$

$t_0 = 0.917t$ approximately according to a Pelikan-Esslinger diagram.

Note that the torsional stiffness of crossbeams can be neglected.

The bending moment of stiffeners at midspan

The homogeneous differential equation of an orthotropic plate, neglecting the bending stiffness in transverse direction, can be written as

$$B_y \frac{\partial^4 w}{\partial y^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} = 0 \tag{19}$$

the solution of which is

$$w = \left[C_1 \sinh(\alpha y) + C_2 \cosh(\alpha y) + C_3 \alpha y + C_4\right] \sin\frac{n\pi x}{b} \tag{20}$$

Considering the boundary conditions, the bending moment at midspan due to a concentrated force Q is given by

$$M_{00} = \psi Q t \left\{ \frac{1}{2\alpha^2 t e_0} \left[1 - \frac{\cosh\left[\alpha\left(\frac{t}{2} - e_0\right)\right]}{\cosh\left(\alpha \frac{t}{2}\right)} \right] + \frac{M_{0t}}{t} \left(1 - \frac{\sinh(\alpha e_0)}{\alpha e_0} \frac{1}{\cosh\left(\alpha \frac{t}{2}\right)} \right) \right\} \tag{21}$$

where $Q = 50$ kN/400 mm is the specific force (wheel load reduced to the plane of a stiffener). M_{00} is the specific bending moment acting in the plane of a stiffener. The bending moment acting on a stiffener is

$$M = M_{00}(a + e) \tag{22}$$

$$M_{0t} = \frac{\kappa_m}{a_t(1 - \kappa_m)} \frac{1}{2\cosh\left(\alpha \frac{t}{2}\right)} t \tag{23}$$

$$\kappa_m = -c + \sqrt{c^2 - 1} \tag{24}$$

$$a_t = -a_{11} B_y \alpha^2 t \tag{25}$$

$$a_{11} = -\frac{1}{B_y \alpha^2 t} \frac{\sinh(\alpha t) - \alpha t}{\sinh(\alpha t)} \tag{26}$$

$$a_{22} = -\frac{2}{B_y \alpha^2 t} \frac{\alpha t \cosh(\alpha t) - \sinh(\alpha t)}{\sinh(\alpha t)} \tag{27}$$

$$\alpha = \frac{n\pi}{b} \sqrt{\frac{2H}{B_y}} \tag{28}$$

The dynamic factor can be calculated according to DIN 1072 [5] as

$$\psi = 1.4 - 0.008 \frac{t}{1000} - 0.1 h_a \tag{29}$$

h_a is the thickness of asphalt.

The bending moment of a crossbeam at midspan

The reactive force from the wheel loads, according to Fig.4, is

$$P = 50\,kN + \frac{2*30(t-3000)}{t} \qquad \text{when} \quad t > 3000 \text{ mm}$$

and

$$P = 50\,kN \qquad \text{when} \quad t < 3000 \text{ mm}$$

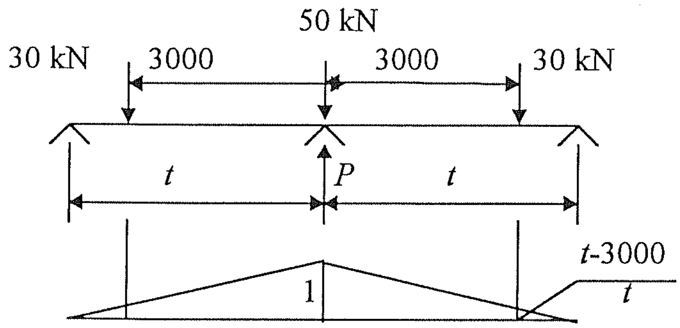

Fig.4. Reactive force P acting on a crossbeam from wheel loads

In our numerical example we treat a bridge deck of width b = 12 m, thus we calculate with 8P (Fig.5).

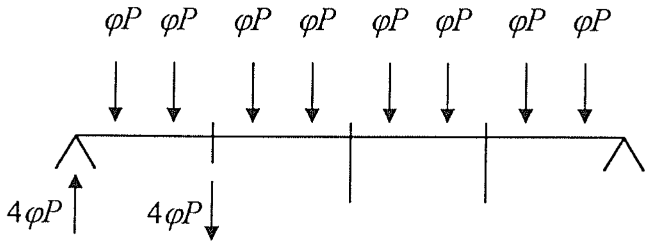

Fig.5. Forces acting on a crossbeam

$$M_{\max} = Pb \tag{30}$$

Fatigue constraint for stiffeners

For bridge decks the fatigue constraint of welded joints is governing. There are some published recent experimental research results in addition to the design categories defined by Eurocode 3 (EC3) [6,7,8]. Although the trapezoidal stiffeners can be prefabricated in workshop and need not splices welded in site, we consider a site splice with butt weld, no splice plate, with backing strip, full penetration, root gap >4 mm, for which the recommended design category according to [8] is 80 MPa. Since this important joint is difficult to access for a test, we consider a safety factor according to EC3 of $\gamma_{Mf} = 1.35$. Spectrum factor can be taken as 1, the number of load cycles is $2*10^6$.

The fatigue constraint is formulated as

$$\frac{M_{00}(a+e)}{I_y}(h - z_G) \le \frac{\Delta\sigma}{\gamma_{Mf}} = \frac{80}{1.35} \tag{31}$$

Fatigue constraint for crossbeams

$$\sigma_{\max} = \psi \frac{M_{\max}}{I_x}(h_w - y_G) \leq \frac{\Delta\sigma_1}{\gamma_{Mf1}} \tag{32}$$

The fatigue stress range is 125 MPa, according to EC3, for automatically welded continuous longitudinal fillet welds carried out from both sides with no stop/start positions. The safety factor can be taken as 1.15 for important accessible joints.

Deflection constraints

Deflection should be calculated without dynamic factor.

Stiffeners:

$$w_{\max} = \frac{Pt^3}{48EI_y} \leq w^* \tag{33}$$

According to Eurocode 3 Part 2 $w^* = 5$ mm.

Crossbeams:

$$\frac{5p_{red}b^4}{384EI_x} \leq w^* \tag{34}$$

where $$p_{red} = \frac{8P}{\mathrm{b}} \tag{35}$$

Local buckling constraint for stiffeners

According to Eurocode 3 Part 2

$$a_2 \leq 38\varepsilon t_s \tag{36}$$

Shear buckling constraint for crossbeam web

$$\frac{F_A}{h_w t_w} \leq \frac{\tau_{ba}}{\gamma_{M1}} \tag{37}$$

where the partial safety factor for shear is

$$\gamma_{M1} = 1.1 \tag{38}$$

$$\lambda_w = \frac{\frac{h_w}{t_w}}{37.4\varepsilon\sqrt{\kappa_\tau}} \tag{39}$$

if $\lambda_w \leq 0.8$ then $\tau_{ba} = \frac{f_y}{\sqrt{3}}$

if $0.8 \le \lambda_w \le 1.2$ then $\tau_{ba} = \frac{f_y}{\sqrt{3}}\left[1 - 0.625(\lambda_w - 0.8)\right]$

if $\lambda_w \ge 1.2$ then $\tau_{ba} = \frac{0.9 f_y}{\lambda_w \sqrt{3}}$ (40)

Frequency constraints

The stiffeners' first eigenfrequency is limited for according to Eurocode 3. Part 2. [7] in 2 Hz. The calculation of eigenfrequency is as follows

$$f_{1s} = \frac{\pi}{2t^2}\sqrt{\frac{EI_y}{m_s}} \tag{41}$$

The cross section area of the stiffener and the deck plate is

$$A_s = a_1 t_s + 2a_2 t_s + (a+e)t_f \tag{42}$$

The mass of this part is

$$m_s = A_s * 7.85 * 10^{-6}(a+e) \tag{43}$$

The cross beam's first eigenfrequency is also limited in 2 Hz. The calculation of eigenfrequency is as follows

$$f_{1c} = \frac{\pi}{2b^2}\sqrt{\frac{EI_x}{m_c}} \tag{44}$$

The cross section area of the cross beam and the deck plate is

$$A_c = t_0 t_f + b_{f1} t_{f1} + h_w t_w \tag{45}$$

The mass of this part is

$$m_c = A_c * 7.85 * 10^{-6}(a+e) \tag{46}$$

Size limitations

According to Eurocode 3 Part 2

$$e \le 25\, t_f \tag{47}$$

$t_s \ge 6$ mm

$t_f \ge 12$ mm

Formulation of a cost function according to the fabrication steps

The cost of a structure is the sum of the material and fabrication costs. The fabrication cost elements are the welding-, cutting-, preparation-, assembly-, tacking-, painting costs etc. It is very difficult to obtain such cost factors, which are valid all over the world, because there are great differences between the cost factors at the highly developed and developing countries. If we choose the time, as the basic data of a fabrication element we can handle this problem. The fabrication time depends on the technological level of the country and the manufacturer, but it is much closer to the real process to calculate with. After computing the necessary time for a fabrication work element one can multiply by a specific cost factor, which can represent the development level differences. Although the whole production cost depends on many parameters and it is very difficult to express their effect mathematically, a simplified cost function can serve as a suitable tool for comparisons useful for designers and manufacturers [9,10].

The cost function can be expressed as

$$K = K_m + K_f = k_m V + k_f \sum_i T_i \tag{48}$$

where K_m and K_f are the material and fabrication costs, respectively, k_m and k_f are the corresponding cost factors, ρ is the material density, V is the volume of the structure, T_i are the production times.

Fabrication times for welding

Table 1. Welding times T_2 (min) in function of weld size a_w (mm) for longitudinal fillet welds downhand position (see also Fig. 6.)

Welding method	a_w (mm)	$10^3 T_2 = 10^3 C_2 a_w^n$
SMAW	2 - 5	$4.0\ a_w$
	5 - 15	$0.7889\ a_w^2$
GMAW-C	2 - 5	$1.70\ a_w$
	5 - 15	$0.3394\ a_w^2$
SAW	2 -5	$1.190\ a_w$
	5 - 15	$0.2349\ a_w^2$

Eq.(48) can be written in the following form

$$\frac{K}{k_m} = \rho V + \frac{k_f}{k_m}(T_1 + T_2 + T_3) \tag{49}$$

where

$$T_1 = C_1 \Theta_d \sqrt{\kappa \rho V} \tag{50}$$

is the time for preparation, assembly and tacking, Θ_d is a difficulty factor, κ is the number of structural elements to be assembled.

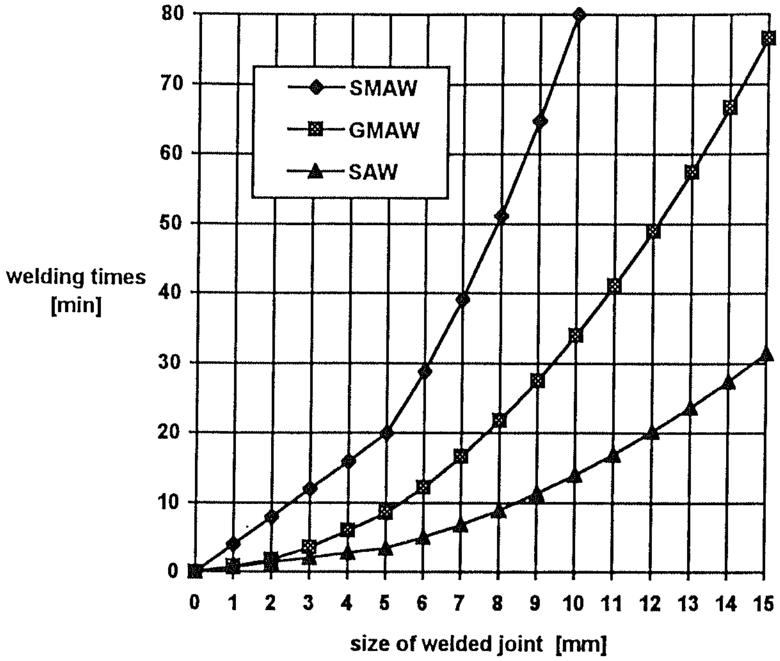

Fig. 6 Welding times for fillet welds of size a_w

Table 2. Welding times T_2 (min) in function of weld size a_w (mm) for longitudinal 1/2 V butt welds downhand position

Welding method	a_W (mm)	$10^3 T_2 = 10^3 C_2 a_w^n$
SMAW	4 - 6 6 - 15	$2.7\ a_w$ $0.45\ a_w^2$
GMAW-C	4 - 15	$0.1939\ a_w^2$
SAW	4 - 15	$0.1346\ a_w^2$

Table 3. Welding times T_2 (min) in function of weld size a_w (mm) for longitudinal K-butt welds downhand position

Welding method	a_W (mm)	$10^3 T_2 = 10^3 C_2 a_w^n$
SMAW	5 - 16	$1.4029 a_w^{1.25}$
GMAW-C	5 - 16	$0.129\ a_w^2$
SAW	5 - 16	$0.089\ a_w^2$

$$T_2 = \sum_i C_{2i} a_{wi}^n L_{wi} \qquad (51)$$

is the time of welding, a_{wi} is the weld size, L_{wi} is the weld length in mm, C_{2i} and n are constants given for different welding technologies.

$$T_3 = \sum_i C_{3i} a_{wi}^n L_{wi} \qquad (52)$$

is the time of additional fabrication actions such as changing the electrode, deslagging and chipping.
The different welding technologies are as follows: *SMAW, GMAW-C, SAW.*
Ott and Hubka [10] proposed that $C_{3i} = 0.3\ C_{2i}$, so

$$T_2 + T_3 = 1.3\sum_i C_{2i} a_{wi}^n L_{wi} \tag{53}$$

Values of C_{2i} and n may be given according to *COSTCOMP* [11] as follows. It gives welding times and costs for different technologies [12].To compare the costs of different welding methods and to show the advantages of automation, the manual *SMAW*, semi-automatic *GMAW-C* and automatic *SAW* methods are selected for fillet welds. The analysis of *COSTCOMP* data resulted in constants given in Fig. 6 and Table 1-3 for different joint types.

One can establish other fabrication components, can calculate the fabrication time to even plates, the surface preparation time, the painting time, the cutting and edge grinding times, etc, but the main problem is how to formulate the equation concerning the time. The difficulty factor δ represents that the welding, or painting is overhead, or vertical, or horizontal and also the complexity of the structure. In our case we focused on welding costs [13,14,15,16,17,18]. The robot welding and some new technologies have different cost aspects [19,20].

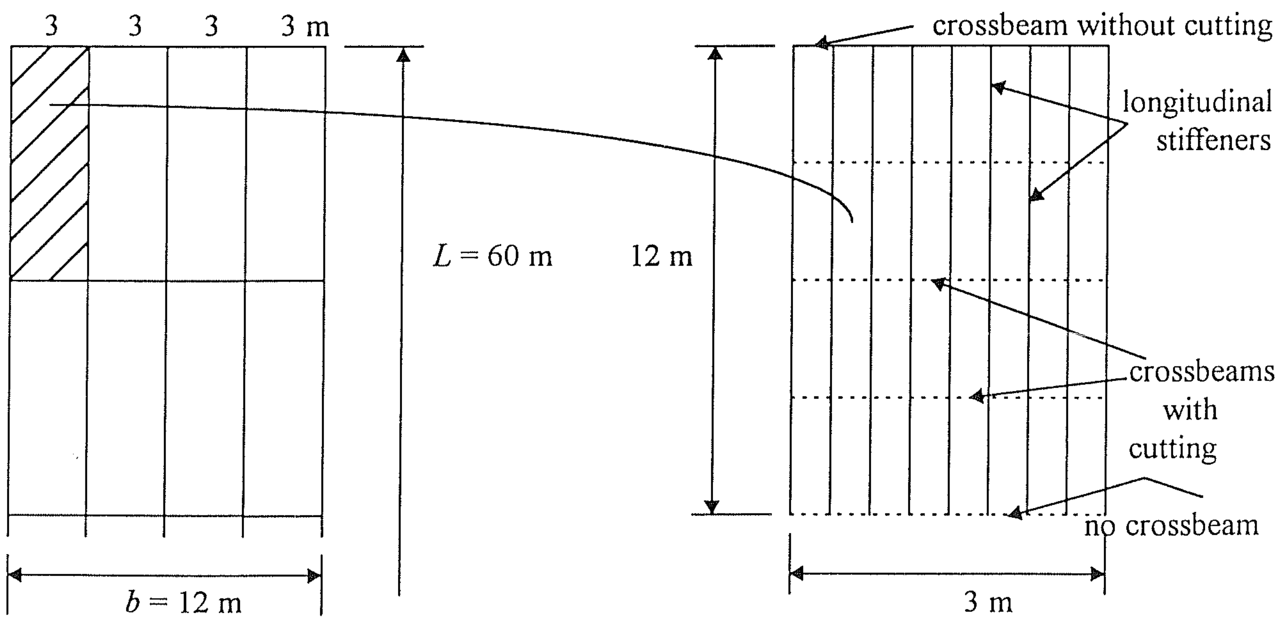

Fig.7. The main dimensions of the bridge of our numerical example

The following formulae and values are used for the fabrication steps in our numerical example.

A complete bridge deck is constructed from structural elements of transportable dimensions [21]. In our numerical example a bridge of length L = 60 m and width b = 12 m is selected, composed from elements of dimensions L_0 = 12 m and b_0 = 3 m (Fig.7).

*1) Fabrication of 12*3 m elements in workshop*

a) welding of stiffeners to deck plate with SAW fillet welds

$$C_2 = 0.2349\ a_w^2, \qquad a_w = 1.25\ t_s, \qquad L_w = 2\Phi_s \Phi t$$

b) cutting the crossbeam webs excluding the end crossbeam

$$C_2 = 1.1388 t^{0.25}, \qquad t = t_w, \qquad L_w = \Phi_s (a_1 + 2a_2)(\Phi - 1)$$

c) welding of crossbeam lower flanges to webs with SAW fillet welds

$$C_2 = 0.2349\ a_w^2, \qquad a_w = 0.5\ t_w, \qquad L_w = 2\Phi_s \Phi t$$

d) welding of crossbeam webs to deckplate and to stiffeners with GMAW fillet welds

$$C_2 = 0.3394\ a_w^2. \qquad a_w = 1.25\ t_s, \qquad L_w = (e + 2a_2)\Phi_s 2(\Phi - 0.5$$

Table 4. The place, type, cost, size and length of the welded joint for the fabrication of the *12*3 m elements in workshop*

Place of the joint	Welding technology	Cost parameter	Welded joint size	Welded joint length
stiffener-deck plate	SAW	$0.2349\ a_w^2$	t_s	$2\Phi_s \Phi t$
cutting	normal acetilene	$1.1388 t^{0.25}$	t_w	$\Phi_s (a_1 + 2a_2)(\Phi - 1)$
crossbeam lower flange- web	SAW	$0.2349\ a_w^2$	$0.5\ t_w$	$2\Phi_s \Phi t$
crossbeam web-deck plate	GMAW	$0.3394\ a_w^2$	$1.25\ t_w$	$(e + 2a_2)\Phi_s 2(\Phi - 0.5)$

The volume of an element in which the number of stiffeners is $\Phi_s = \dfrac{b_0}{a + e}$ and the number of crossbeams is $\Phi = \dfrac{L_0}{t}$

$$V_1 = \Phi_s (a_1 + 2a_2) t_s \Phi t + b_0 \Phi\, t\, t_f + (h_w t_w + A_f)\Phi b_0 \tag{54}$$

The number of assembled elements is

$$\kappa_1 = \Phi_s + \Phi + 1 \tag{55}$$

The cost function for an element can be written as

$$\frac{K_1}{k_m} = \rho V_1 + \frac{k_f}{k_m}\left\{\Theta_d \sqrt{\kappa_1 \rho V_1} + 1.3\left[C_{2SAW}(0.5t_w)^n 2b_0 \Phi + C_{2CUT} t_w^2 \Phi_s (a_1 + 2a_2)(\Phi - 1) + \ldots\right]\right\}$$

$$\left\{\left[\ldots + C_{2SAW}(1.25t_s)^n 2\Phi_s \Phi t + C_{2GMAW}(1.25t_s)^n (e + 2a_2)\Phi_s 2(\Phi - 0.5)\right]\right\} \tag{56}$$

2) Fabrication of the whole deck structure in site

a) welding of stiffeners to end crossbeams with GMAW fillet welds

$$C_{2GMAW} = 0.3394\ a_w^2 \qquad a_w = 1.25\ t_s, \qquad L_w = \frac{b}{b_0}\frac{L}{L_0}(a_1 + 2a_2 + e)\Phi_s$$

b)welding of the longitudinal and transverse splices with SAW butt welds

$$C_{2SAW} = 0.2349\ a_w^2 \qquad a_w = t_f, \qquad L_w = \left(\frac{b}{b_0} - 1\right)b$$

c)welding of crossbeam web and lower flange splices (these splices can also be realized by using bolted connections) with GMAW butt welds

- web splices $\quad C_{2GMAW} = 0.3394\ a_w^2. \quad a_w = t_w, \quad L_w = \left(\frac{b}{b_0} - 1\right)\left(\frac{L}{L_0} + 1\right)$

- flange splices $\quad C_{2GMAW} = .0.3394\ a_w^2. \quad a_w = t_{f1}, \quad L_w = \left(\frac{b}{b_0} - 1\right)\left(\frac{L}{L_0} + 1\right)$

Table 5. The place, type, cost, size and length of the welded joint for the fabrication of the *whole deck in site*

Place of the joint	Welding technology	Welding cost parameter	Welded joint size	Welded joint length
stiffeners to end crossbeams	*GMAW*	$0.3394\ a_w^2$	$1.25\ t_s$	$\frac{b}{b_0}\frac{L}{L_0}(a_1 + 2a_2 + e)\Phi_s$
longitudinal and transverse splices	*SAW*	$0.2349\ a_w^2$	t_f	$\left(\frac{b}{b_0} - 1\right)b$
web splices	*GMAW*	$0.3394\ a_w^2.$	t_w	$\left(\frac{b}{b_0} - 1\right)\left(\frac{L}{L_0} + 1\right)$
flange splices	*GMAW*	$0.3394\ a_w^2.$	t_{f1}	$\left(\frac{b}{b_0} - 1\right)\left(\frac{L}{L_0} + 1\right)$

The volume of the whole deck structure is

$$V_2 = \kappa_2 V_1 \tag{57}$$

$$\kappa_2 = \frac{bL}{b_0 L_0} \tag{58}$$

The cost function of the whole deck structure is

$$\frac{K_2}{k_m} = \frac{k_f}{k_m}\left\{\Theta_d\sqrt{\kappa_2\rho V_2} + 1.3\left[C_{2GMAW}(1.25t_s)^n\frac{b}{b_0}\frac{L}{L_0}(a_1 + 2a_2 + e)\Phi_s + C_{2SAW}t_f^2\left[\left(\frac{b}{b_0} - 1\right)b\right] + \ldots\right]\right\}$$

$$\left\{\left[\ldots + C_{2GMAW}\left(h_w t_w^n + b_{f1} t_{f1}^n\right)\left(\frac{b}{b_0}-1\right)\left(\frac{L}{L_0}+1\right)\right]\right\} \tag{59}$$

The whole cost function to be minimized is

$$\frac{K}{k_m} = \frac{20K_1 + K_2}{k_m} \tag{60}$$

The optimization procedure for the numerical example

The cost function (Eq. 60) is to be minimized considering the following constraints: Eqs.31, 32, 33, 34, 36, 37 and 41.

Size limitations

The variables are as follows: t_s, h_w, t_w, $A_{f1} = b_{f1}.t_{f1}$, which are optimized for a series of discrete values of t to obtain t_{opt} corresponding to K_{min}.

$$12 \le t_f \le 28 \text{ mm}$$

$$300 \le h_w \le 2000 \text{ mm}$$

$$8 \le t_w \le 25 \text{ mm}$$

$$200 \le A_f \le 1600 \text{ mm}$$

$$6 \le t_s \le 20 \text{ mm}$$

Results and conclusions

The optimization is made with the following data:

steel yield stress is $f_y = 235$ MPa, the cost ratio is between $\frac{k_f}{k_m} = 0 \div 2$, the number of unknown variables is 5, they are as follows

$$x_1 = t_f,\ x_2 = h_w,\ x_3 = t_w,\ x_4 = A_f = b_{f1}\, t_{f1},\ x_5 = t_s \tag{61}$$

The number of constraints for the cross beam is 5, for the stiffener is 3 according to Eqs.(31, 32, 33, 34, 36, 37, 41, 44 and 47).

The mathematical optimization technique is the Rosenbrock's Hillclimb procedure [4].

The results for normal steel, $f_y = 235$ MPa, and cost ratio $\frac{k_f}{k_m} = 0.5$ can be seen in Table 6.

Table 6. The minimum cost of bridge deck due to different cross beam distance with normal steel and cost ratio = 0.5

t mm	Total cost \$	Material cost \$	Fabrication cost \$
1000	344199.8	261422.0	112381.0
1500	272902.0	215660.5	89674.1
2000	269494.5	217542.1	83933.3
2500	$\boxed{245737.0}$	179673.0	66064.0
3000	246077.6	177011.8	69065.8
3500	327045.2	202234.9	124810.2
4000	314094.2	199084.1	115010.0
4500	324922.8	186202.2	138720.6

The results for normal steel, $f_y = 235$ MPa, and cost ratio $\frac{k_f}{k_m} = 1.0$ can be seen in Table 7. and Fig. 8.

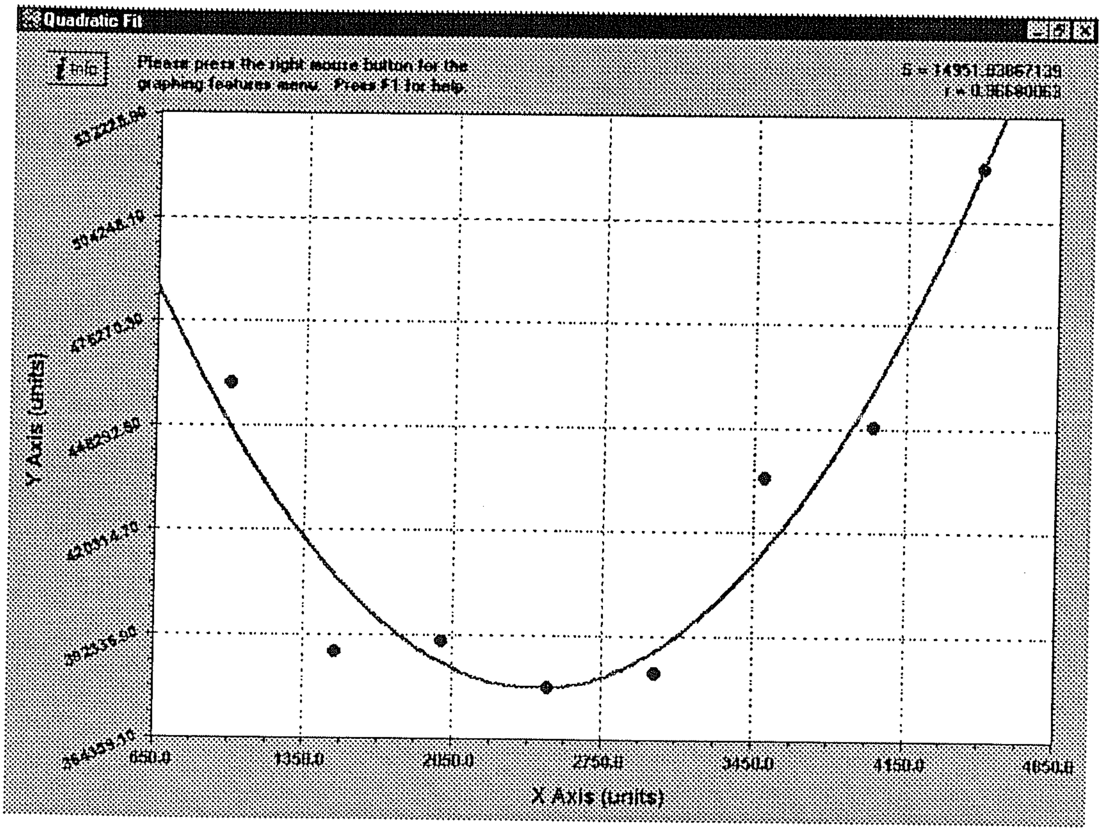

Fig. 8. The optimum bridge decks' cost in the function of the cross beam distance

The results for normal steel, $f_y = 235$ MPa, and cost ratio $\frac{k_f}{k_m} = 2.0$ can be seen in Table 8.

Table 7. The minimum cost of bridge deck due to different cross beam distance with normal steel and cost ratio = 1.0

t mm	Total cost \$	Material cost \$	Fabrication cost \$
1000	459578.3	323394.6	182805.8
1500	377686.3	285605.9	177337.8
2000	390670.2	269446.8	188773.8
2500	378348.4	228944.2	149404.2
3000	382242.1	222327.5	159884.6
3500	434997.3	217004.6	217992.7
4000	448506.2	241262.1	207244.1
4500	518236.9	239987.3	278249.5

Table 8. The minimum cost of bridge deck due to different cross beam distance with normal steel and cost ratio = 2.0

t mm	Total cost \$	Material cost \$	Fabrication cost \$
1000	686824.3	500915.1	350528.2
1500	612183.4	405159.3	355136.3
2000	582811.4	389439.0	343595.5
2500	671563.8	336396.2	398456.2
3000	666933.8	311444.8	355488.9
3500	706580.4	296284.2	410296.2
4000	758852.9	339493.8	419359.2
4500	792211.7	323837.4	468374.3

The results show, that the optimum distance between cross beams for $\frac{k_f}{k_m} = 0.5$ is about $t_{opt} = 2600$ mm, for $\frac{k_f}{k_m} = 1.0$ is $t_{opt} = 2500$ mm, for $\frac{k_f}{k_m} = 2.0$ is $t_{opt} = 1900$ mm. The higher fabrication cost results closer cross beam, which means weaker stiffeners.

The ratio between material over fabrication costs for $\frac{k_f}{k_m} = 0.5$ ratio is about *30% - 70%,* for $\frac{k_f}{k_m} = 1.0$ is *43% - 57%,* for $\frac{k_f}{k_m} = 2.0$ is *46% - 54%* respectively.

Using higher strength steel $f_y = 355$ MPa, for $\frac{k_f}{k_m} = 2.0$ the optimum cross beam distance is $t_{opt} = 2050$, instead of 1900 mm. It means, that higher strength steel increase the distance between cross beams.

We have fixed the minimum stiffener thickness in 6 mm. If we decrease this limit to 4 mm, the distance of cross beams became smaller due to the smaller stiffener costs.

The developed computer program runs on Pentium PC, under MS Fortran Power Station Developing System. The user interface is made by MS Visual Basic. One runtime is a few minutes. The calculation contains the discretization after finding the continuous values. This is very important for the fabrication.

The computer calculation shows, that this program is useful for the predesign of bridge decks, taking into account fatigue, stability, deflection and frequency constraints.

Acknowledgements

The first author would like to thank the Japan Society of Promotion of Science (JSPS) for the possibility of being a visiting professor at Osaka University in 1996/97, for 6 months and also Mitsubishi Heavy Industry Ltd. in Kobe and Sumitomo Heavy Industry Ltd. Toyo Works, Imazaike for their help. Thanks for the calculation work of Ms.Klara Gyulai undergraduate student. This work received support from the Hungarian Fund for Scientific Research Grants OTKA T- 22846 and T- 19003.

References

1.Jármai,K.,Horikawa,K.,Farkas,J.: Economic design of steel bridge decks, Transactions of JWRI , Vol. 26. No. 1. 1997. 16 p. (under publication)

2 W. Pelikan, M. Esslinger: Die Stahlfahrbahn-Berechnung und Konstruktion, M.A.N. Forschungsheft, No.7, 1957.

3. J. Farkas: Optimum Design of Metal Structures. Ellis Horwood, Chichester, 1984.

4. J. Farkas, K. Jármai: Analysis and Optimum Design of Metal Structures. Balkema Publishers, Rotterdam, Brookfield, 1997. (under publication)

5. DIN 1072, Strassen- und Wegbrücken, Deutsche Norm, 1985. Berlin

6. Eurocode 3. Design of Steel Structures. Part 1.1 General rules and rules for buildings. CEN - European Committee for Standardization. Febr. 1992.

7. Eurocode 3. Steel Bridges and Plated Structures. Part 2. 2nd draft, Brussels, CEN - European Committee for Standardization. Jun. 1995.

8. M.H.Kolstein, J.R. Cuninghame, A.Bruls: Fatigue classification of welded details in orthotropic steel bridge decks, Proceedings International Conference on Fatigue of Welded Components and Structures, Paris, pp.15-23. 1996. Les Editions Physique.

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

10. H. H. Ott, V. Hubka: 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.

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

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

13. J. Farkas: Minimum cost design of welded structures. In Engineering Design in Welded Constructions. Proceedings of the Int. Conference IIW, Madrid, Spain. Pergamon Press, Oxford, etc. pp.135-142. 1992.

14. J. Farkas, K. Jármai: Minimum cost design of laterally loaded welded rectangular cellular plates. In Structural Optimization '93 World Congress, Rio de Janeiro. Proc. Vol. 1. pp. 205-212. 1993.

15. J. Farkas, K. Jármai: Fabrication cost calculations and minimum cost design of welded structural parts. Welding in the World, Pergamon Press, Vol. 35. 1995. No. 6, pp. 400-406.

16. K. Jármai, J. Farkas: Optimum design of welded steel structures. Tools and Methods for Concurrent Engineering Symposium, TMCE'96, Technical University of Budapest, May 29-31. 1996. Budapest, Proceedings pp. 449-468.

17. J. Farkas, K. Jármai: Optimum design of welded bridges. 49th Annual Assembly of International Institute of Welding, International Conference on Welded Structures in Particular Welded Bridges, September 2-3, 1996. Budapest, GTE, Scientific Society of Mechanical Engineering, Proceedings pp. 299-310.

18. J. Farkas, K. Jármai: Fabrication cost calculation and optimum design of welded steel silos. Welding in the World, Pergamon Press, Vol. 37. 1996. No. 5, pp. 225-232.

19. K. Horikawa, T. Nakagomi et al.: The present position in the practical application of arc welding robots and proposals on joint design for robot welding of steel structures. Welding in the World Vol. 30. No. 9 - 10. pp. 256-274. 1992.

20. K. Horikawa, N. Watanabe: Computer-aided fabrication of welded structures. 49th Annual Assembly of International Institute of Welding, International Conference on Welded Structures in Particular Welded Bridges, September 2-3, 1996. Budapest, GTE, Scientific Society of Mechanical Engineering, Proceedings pp. 77-91.

21. F.R. Weitz: Neuzeitliche Gesichtspunkte im schweissenden Brückenbau, Der Stahlbau, Vol. 43. No.3, 1974, pp.73-81.