Economic Design of Steel Bridge Decks with Open Ribs

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

Abstract

In order to minimize the cost of welded orthotropic deck bridge structures, optimization studies should be performed, which need mathematical formulation of the cost function. The previously used Pahl-Beelich method is modified by using the COSTCOMP program to have cost functions for various welding methods. Illustrative numerical examples of different welded orthotropic deck bridge structures have been worked out to show the influence of fabrication cost on the optimal sizes of the structure. The analysis of the bridge deck is made by the Pelikan-Esslinger method. Static stress, fatigue, overall and local buckling constraints have been taken into account. It is shown that the optimal sizes depend on welding method, so, to achieve economic structures, the designer should consider also the fabrication aspects. The optimization is made by the Hillclimb method of Rosenbrock in Fortran language on PC.

KEY WORDS: (Fabrication Cost) (Minimum Cost Design) (Orthotropic Bridge Deck) (Welding Method) (Stiffened Plates) (Structural Optimization)

1. Introduction

The economy of welded orthotropic deck bridge structures plays an important role in the research and production, therefore it is included in the work of *IIW Commission XV*. It needs a co-operation of designers and manufacturers, so it is a main task for the Subcommission XV-F "Interaction of design and fabrication".

The decrease of costs may be achieved by various ways. One efficient way is to use the mathematical optimization methods. In structural optimization the version is sought which minimizes the objective function and fulfils the design constraints [1,2]. As objective function the mass (weight) is often defined, but the minimum weight design does not give the optimal version for minimum cost. Therefore a more complex cost function should be defined including not only the material but also the fabrication costs.

In the industry it is common to use the cost/tonne concept [3], but it is not suitable for optimization. If we use a cost/tonne cost factor for fabrication cost, then the material and fabrication costs will give similar, non-conflicting functions, which do not lead to an optimum. To find an optimum we need conflicting functions, thus, we should use a more suitable fabrication cost calculation method based on a more detailed cost analysis. Many ways of cost calculation have been proposed [4-13].

In recent publications [18-23] the authors have used a relative simple cost function proposed by *Pahl and Beelich* [14]. These authors have given the production times only for *SMAW* (shielded metal arc welding) and *GMAW-C* (gas metal arc welding with CO_2). To apply the cost calculations for other welding technologies, mainly for *SAW* (submerged arc welding), the *COSTCOMP* [16] software has been used [17]. The values of COSTCOMP enable us to define cost functions for different welding technologies. The second author dealt with the computer aided fabrication and robotization of welded structure production [24,25].

The aim of the present study is to apply the minimum cost design procedure for welded structures to show the effect of fabrication cost on the optimal sizes of a bridge deck structure by cost comparisons.

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2. Pelikan-Esslinger method

The approach presented by *Pelikan & Esslinger* [26] is a practical and relatively simple procedure for designing orthotropic steel bridges. The main simplifying assumption is that the orthotropic plate is continuous and supported rigidly by the main girders and elastically by the floor beams. The bending moments in a steel plate treated as an orthotropic plate depend on (*i*) the loading, (*ii*) the floor beam spacing, (*iii*) the main girder spacing and the magnitude and ratio of the three characteristic rigidities. The *i, ii, iii* are the main elements of orthotropic plate representing the actual system.

The bridge deck is divided into three distinct systems, whose individual actions are added to obtain the final bridge response.

System I. The local behaviour of the deck plate spanning the distance between supporting ribs,

System II. The response of the deck plate, ribs and transverse floor beams forming the bridge deck,

System III. The action of the main longitudinal girder, interacting with the deck plate and longitudinal ribs, assumed to be large beam spanning between supports.

The major contribution of Pelikan & Esslinger is in the prediction of the behaviour of the System II, the continuous orthotropic plate on flexible supports.

For the plate deck with open or closed ribs, two different computational steps are usually necessary. In the first step the maximum values of the bending moments in the longitudinal ribs and in the floor beams are computed assuming that the floor beams are infinitely rigid. In the second step the effect of the elastic flexibility of the floor beams are determined and the bending moments obtained in the first step are adjusted.

Total bending moments are found by superimposing the influence of dead and live load assuming rigid supports and live loads assuming elastic floor beams (**Fig. 1.**).

2.1. Deck plates with open ribs

The complete solution of the orthotropic plate was first presented by Huber. The differential equation giving the relationship between deflection and the loading of the orthotropic plate, often referred as Huber's equation, is

$$D_x \frac{\partial^4 w}{\partial x^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_y \frac{\partial^4 w}{\partial y^4} = p(x,y) \quad (1)$$

where w is the deflection of the middle surface of the plate at the point (x,y). The parameters D_x, D_y and H are the flexural and torsional rigidity coefficients respectively and $p(x,y)$ is the loading intensity at any point expressed as a function of coordinates x and y.

Deck plates with open ribs have negligible stiffnesses D_x and H as compared to the primary longitudinal D_y (**Fig. 2.**). Therefore the general plate equation becomes

$$D_y \frac{\partial^4 w}{\partial y^4} = p(x,y) \quad (2)$$

This equation represents the load-deflection condition response of the beam of the stiffness D_y, which is continuous over the transverse floor beam.

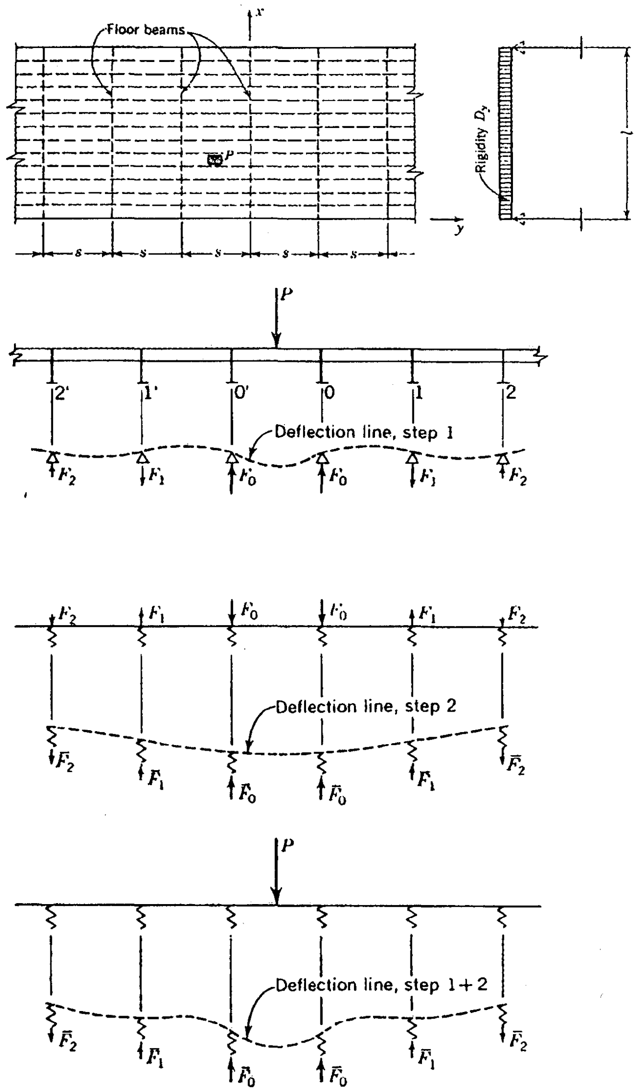

Fig. 1. Pelikan & Esslinger method, stages I and II

2.2. Deck plates with closed ribs

Deck plates stiffened by closed ribs have negligible transverse rigidity D_x as compared with D_y and H (**Fig. 3.**). Therefore the plate equation reduces to

$$2H \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_y \frac{\partial^4 w}{\partial y^4} = p(x,y) \quad (3)$$

In the analysis of either open or closed stiffened-rib deck plates the following design information is required:

- *effective width of stiffened deck plate,*
- *effective wheel-load distribution on the deck plate,*
- *properties of stiffened deck plate,*
- *influence lines of deck plate,*
- *effects of flexible floor beams on deck plate.*

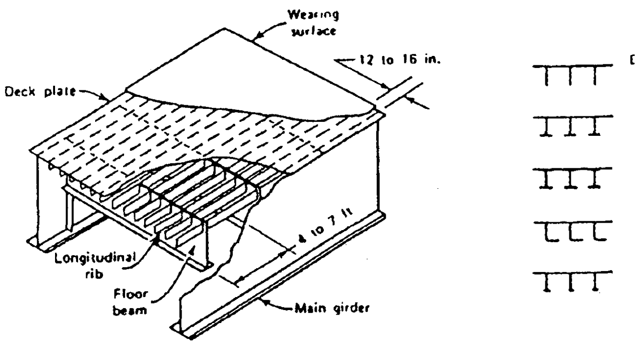

Fig. 2. Deck plates with open ribs[27)]

The effective width of the deck plate acting with one rib may be smaller, equal to, or larger than the rib spacing. It is defined as a_0. The effective span of the rib is defined as the average length of the positive moment distribution of the rib.

$$\beta = \frac{\pi a^*}{L_1} \qquad (4)$$

where $L_1 = 0.7L$, L spacing of floor beams,

a^* is the effective rib spacing which accounts for unequal load distribution.

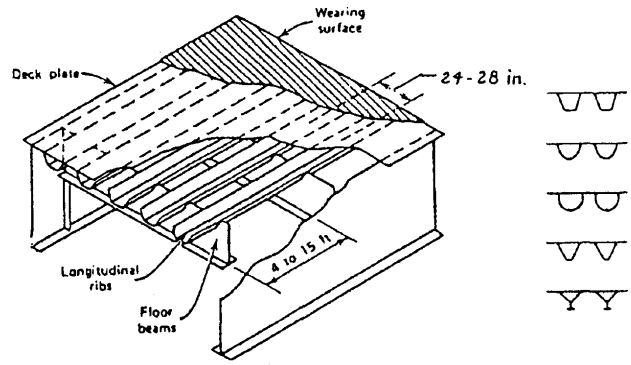

Fig. 3. Deck plates with closed ribs[27)]

2.3. Effective width of stiffened deck plate

Calculation of these dimensions can be obtained by using **Fig. 4.**

The calculation of a^* depends on the loading cases: 1st wheel load over one open stiffener, 2nd wheel load over several stiffeners. These two cases relate the load distribution the effective rib spacing (a^*) and the actual spacing (a) as a function of B/a. B = 2 g is the tire width. The calculation of the equivalent loads to each rib can also be obtained by using **Fig. 5a and 5b.**

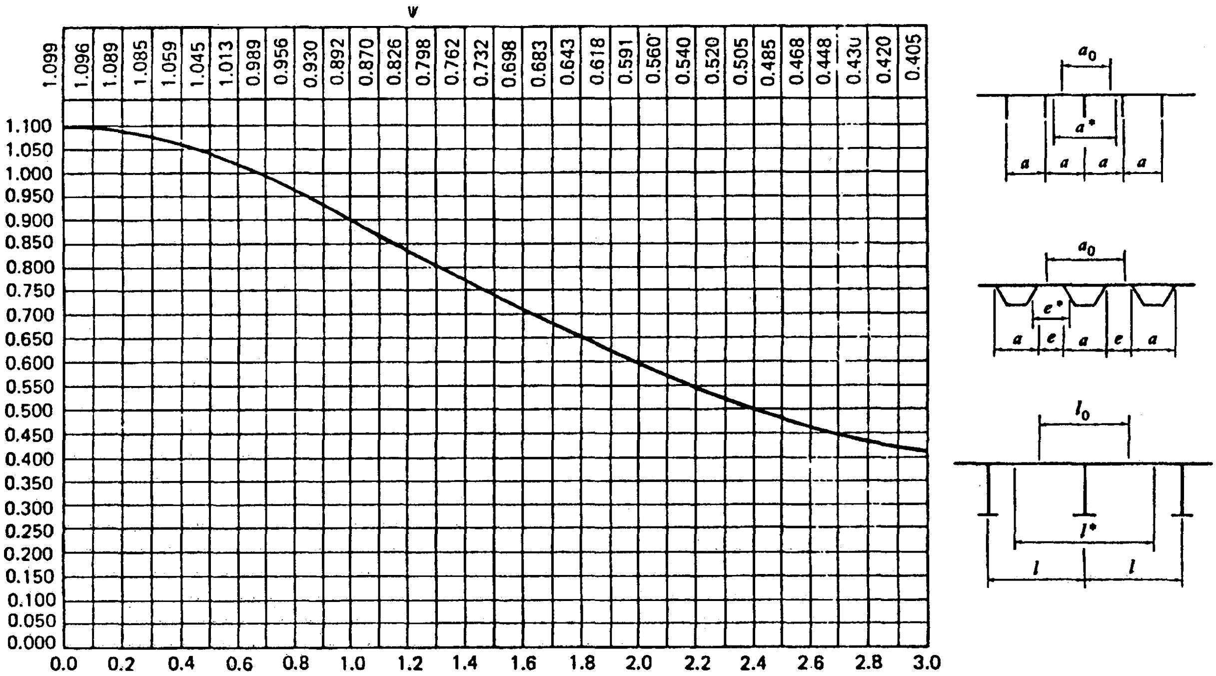

Fig.4. Calculation of the effective widths of deck plate a^*

For the floor beams

$$\beta = \frac{\pi L^*}{b} \qquad (5)$$

where b is the spacing between main girders,

L^* is the effective floor beam spacing which accounts for unequal load distribution.

In general L^* is equal to L, the actual floor-beam spacing.

In evaluating the effective width of closed ribs, we can assume the following

$$a_0 = \lambda_1 a + \lambda_2 e \qquad (6)$$

where λ_1 is found from $\beta_1 = \dfrac{\pi a}{L_1}$,

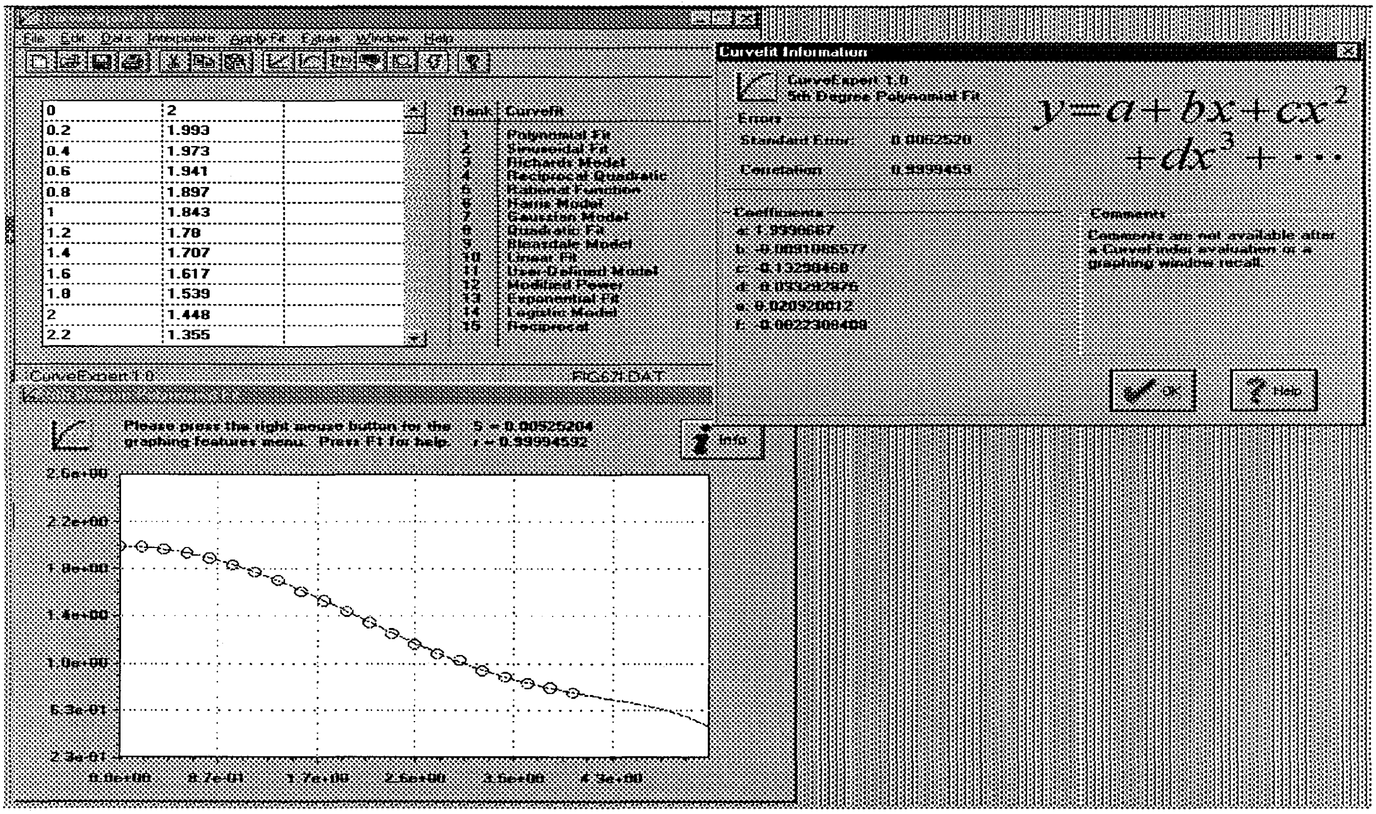

Fig. 5a. Ideal spacing of flexible ribs in System I, $y=\frac{a^*}{a}$, $x=\frac{B}{a}$ and the approximation by a 5[th] order polynom

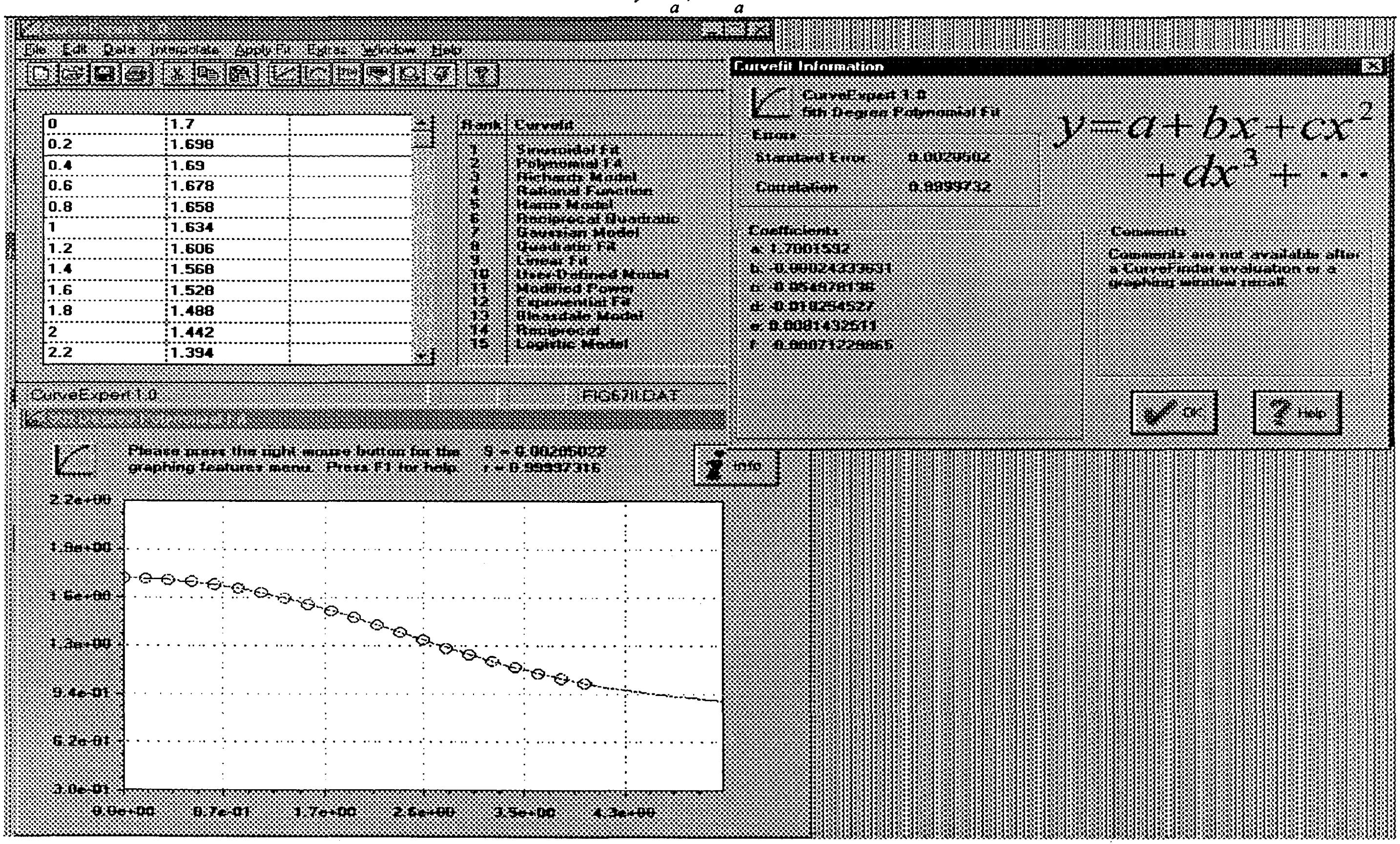

Fig. 5b. Ideal spacing of flexible ribs in System II, $y=\frac{a^*}{a}$, $x=\frac{B}{a}$ and the approximation by a 5[th] order polynom

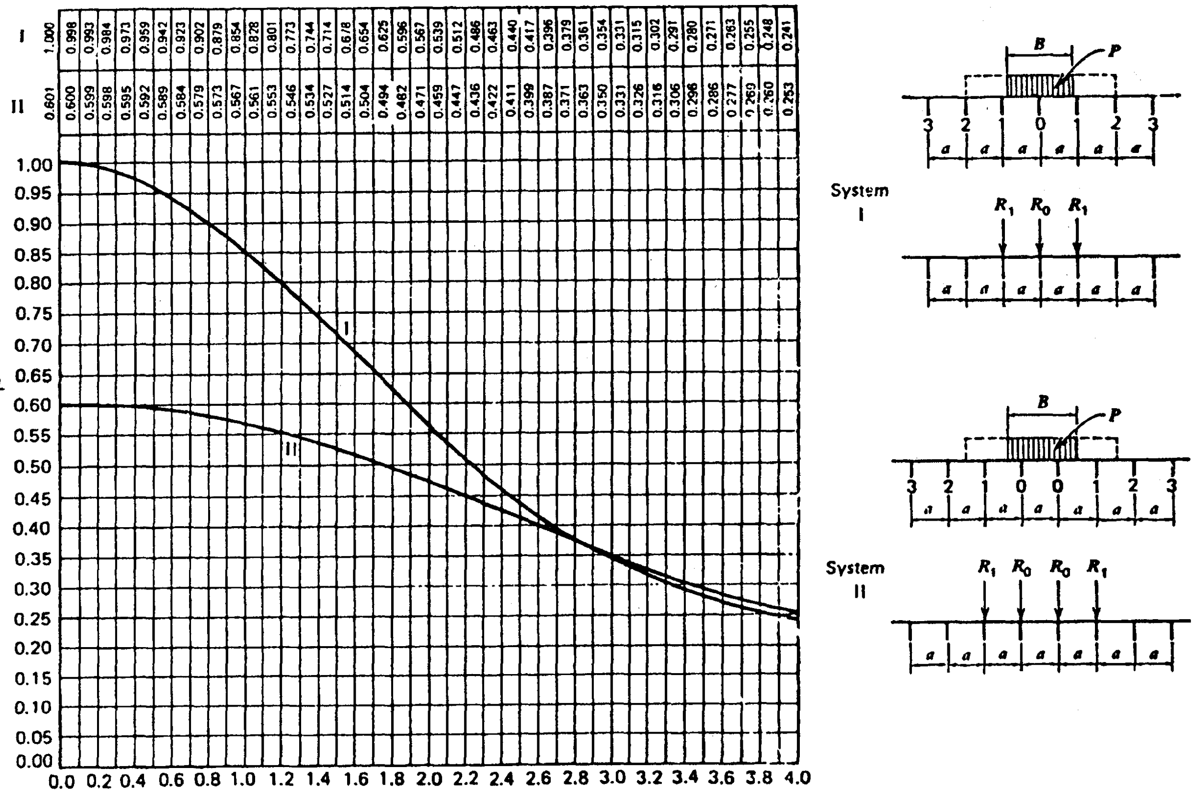

Fig.6. Calculation of R_o and R_1

2.4. Solution for open-rib deck plate, rigid floor beams

Bending moments and reactions are computed from influence lines of a continuous beam. The influence line for the bending moment at any point of a continuous beam is defined as the deflection line of the beam due to a unit rotation at any point. Similarly the influence line for the reaction at the support is the deflection line due to a unit deflection at the support where the reaction is sought. The moment at midspan is given by two expressions, Eq. (7) considers a distributed wheel load (2c) at $y=L/2$

$$\left[\frac{M_c}{PL}\right]_{00'} = \left[0.1708 - 0.250\left(\frac{c}{L}\right) + 0.1057\left(\frac{c}{L}\right)^2\right] \qquad (7)$$

where $2c$ = contact wheel length.

If the loads are applied in any other span, the effects of the wheel load dimensional width $2c$ is neglected and a point load is assumed, thus the moment is given by

$$\left[\frac{M_c}{PL}\right]_m = \left[-0.183\frac{y}{L} + 0.317\left(\frac{y}{L}\right)^2 - 0.134\left(\frac{y}{L}\right)^3\right](-0.268)^m \qquad (8)$$

where P is any concentrated wheel load,

L is the spacing between floor beams,

m is the smaller of the two support numbers enclosing the span under consideration,

y is the location of the wheel with respect to left support of loaded span.

The influence lines for the midspan moment (M_c) of the deck are shown in **Fig. 7.**

The moment over the support is given by the following equation:

$$\left[\frac{M_s}{PL}\right]_m = \left[-0.5\frac{y}{L} + 0.866\left(\frac{y}{L}\right)^2 - 0.366\left(\frac{y}{L}\right)^3\right](-0.268)^m \qquad (9)$$

The influence lines for the support moment (M_s) of the deck are shown in **Fig. 8.**

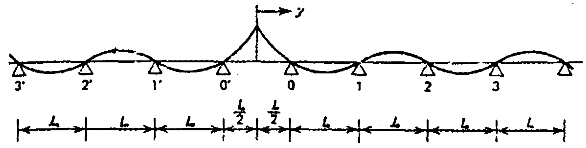

Fig. 7. The influence lines for the midspan moment (M_c) of the deck

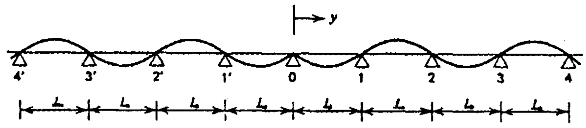

Fig. 8. The influence lines for the support moment (M_s) of the deck

When the load is between 0-1' or 0-1, the term $(-0.268)^m$ is equal to 1.0 and the remaining of equation

(Eq. 9.) is applied. The forces on the floor beams due to the action of the load through the deck are also important for the bridge system design. The reaction at support 0 below due to a load applied in span 0-1 or in any other span can be determined by the following formulae:

$$\left[\frac{F_0}{P}\right]_{0-1} = \left[1.-2.196\left(\frac{y}{L}\right)^2 + 1.196\left(\frac{y}{L}\right)^3\right] \tag{10a}$$

For any other span:

$$\left[\frac{F_0}{P}\right]_m = \left[-0.804\left(\frac{y}{L}\right) + 1.392\left(\frac{y}{L}\right)^2 - 0.5885\left(\frac{y}{L}\right)^3\right](-0.268)^{m-1} \tag{10b}$$

where F_0 is the support reaction.

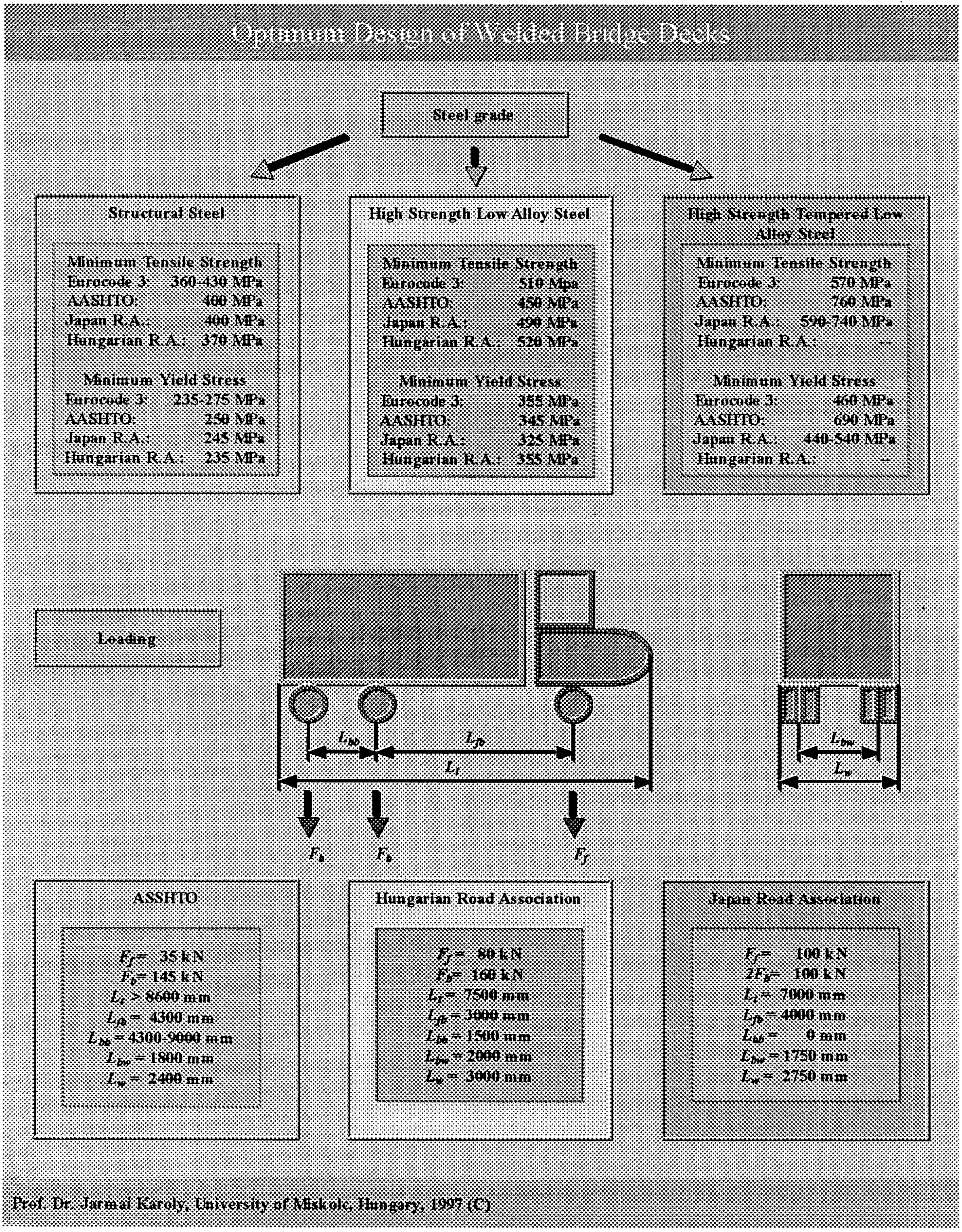

Fig. 9. Truck loading and steel grades according to different standards

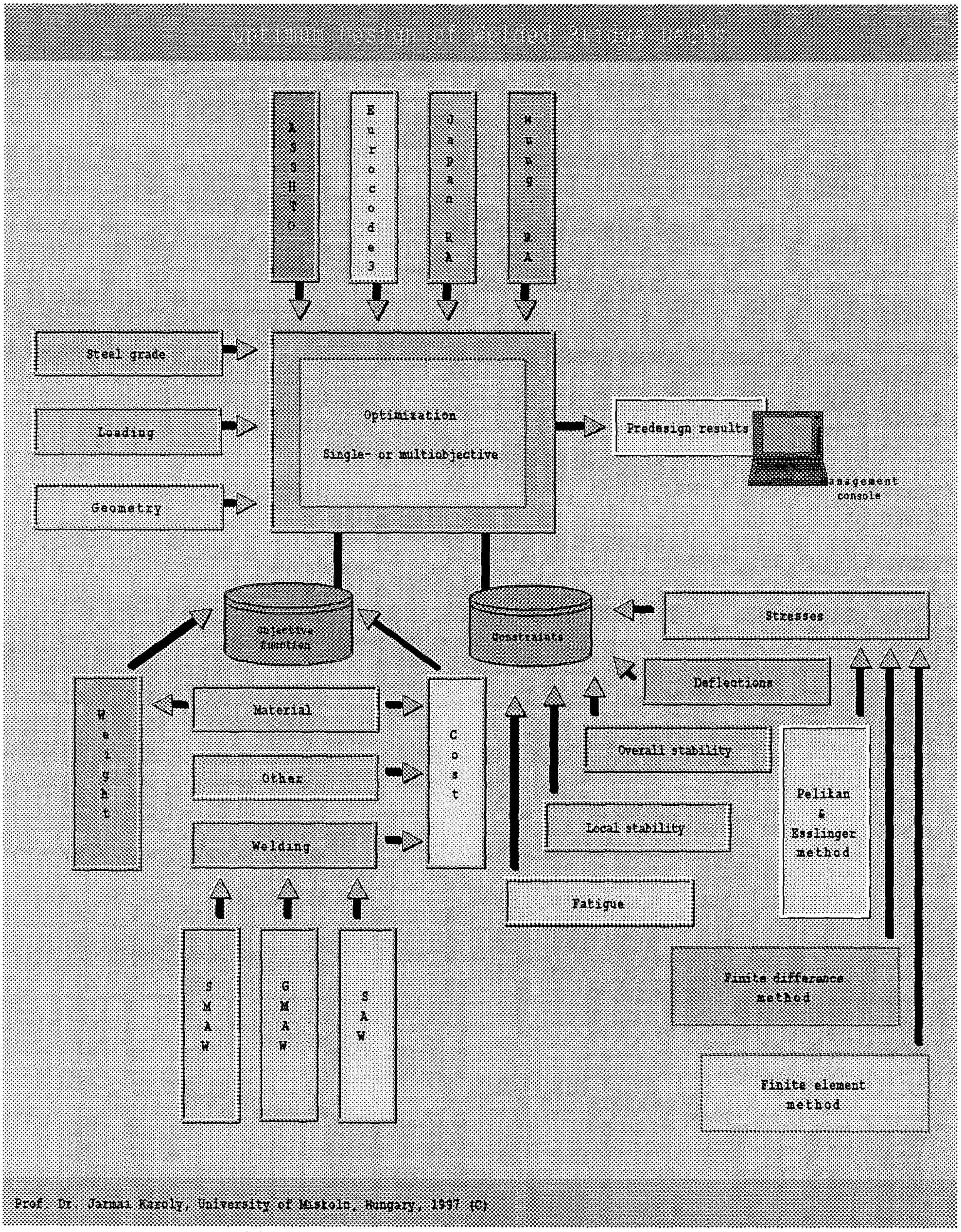

Fig. 10. The system of optimum design at bridge decks

2.5. Flexible floor beams

The bending moments and reactions resulting from the above calculations assume a rigid support. The interaction of the deck and floor-beam is necessary to consider floor-beam flexibility. Such kind of interaction is related to the stiffness first

$$\gamma = \frac{I_r b^4}{I_f L^3 a \pi^4} \tag{11}$$

where I_r is the inertia of open rib,

I_f is the inertia of floor beam including effective plate width,

b is the spacing of floor beams,

a is the spacing of ribs.

Assuming this relative stiffness, influence line tables have been developed for M, M_c and F_0. Using a curvefit program, like that above (**Fig. 7, 8.**) the modified moments in the deck and floor-beams can be determined. The influence line ordinates (η) are as follows for different span between ribs

$$\frac{\eta}{L_0} = \frac{0.020255206*6.9837984+1.4199006*\gamma^{0.47456776})}{6.9837984+\gamma^{0.47456776}} \quad (12)$$

$$\text{If } \gamma \le 0.4 \text{ then} \quad \frac{\eta}{L_1} = \frac{-0.0019931042-0.81820442*\gamma}{1.+12.251167*\gamma+5.0379075*\gamma^2} \quad (13)$$

$$\text{Else } \frac{\eta}{L_1} = -0.048571785-0.013317976*\gamma+0.034803565*\gamma^2-0.01725081*\gamma^3+0.0047435409*\gamma^4-$$

$$-0.0.00077578116\ *\gamma^5+0.000074801612*\gamma^6-0.0000039256189*\gamma^7+6.478443*10^{-9}*\gamma^8 \quad (14)$$

$$\text{If } \gamma \le 0.1 \text{ then} \quad \frac{\eta}{L_2} = -0.006205574+0.008684298*\cos(18.80646*\gamma+0.16075814) \quad (15)$$

$$\text{If } 0.1<\gamma \le 1.0 \text{ then} \quad \frac{\eta}{L_2} = \frac{0.0035636233*0.37888409-0.096464639*\gamma^{1.2340666}}{0.37888409+\gamma^{1.2340666}} \quad (16)$$

$$\text{If } \gamma > 1.0 \text{ then} \quad \frac{\eta}{L_2} = -0.019776194-0.08363159*\gamma+0.047276019*\gamma^2-0.016260564*\gamma^3+ \quad (17)$$

$$+0.0036367224*\gamma^4-0.00052154753*\gamma^5+4.60719*10^{-5}*\gamma^6-2.2764088*10^{-6}*\gamma^7+4.8048292*10^{-8}*\gamma^8$$

$$\text{If } \gamma \le 0.08 \text{ then} \quad \frac{\eta}{L_3} = -0.0009825+0.070525*\gamma-0.49375*\gamma^2 \quad (18)$$

$$\text{Else} \quad \frac{\eta}{L_3} = \frac{-0.14077367*0.47485795+0.0052633893*\gamma^{-1.2175133}}{0.47485795+\gamma^{-1.2175133}} \quad (19)$$

2.6. Bending moment modification at midspan of rib

Because of the flexibility of the floor beam, the moment M_c (Eq. 7.) for the rigid-support is modified by

$$\Delta M_c = Q_0 La \frac{Q_{ix}}{Q_0} \sum \frac{F_m}{F} \frac{\eta_m}{L} \quad (20)$$

$$\text{where} \quad Q_0 = \frac{P}{2g} \quad (21)$$

P is the wheel load intensity,

$2g$ is contact width of tires,

F_m are the reaction forces due to load P at support m of the continuous beam on rigid supports,

η_m are the influence line ordinates at flexible support m for the bending moment at midspan,

a is the width of rib.

$$\frac{Q_{ix}}{Q_0} = \frac{8}{\pi}\sin\frac{\pi g}{b}\cos\frac{\pi e}{b}\sin\frac{\pi d_2}{b}\sin\frac{\pi x}{b} \quad (22)$$

where e is the spacing between wheels,

d_2 is the distance from support to center of gravity of wheel set,

b is the spacing of the main floor beam.

The moment ΔM_c increases the previously computed moment.

The bending moment modification at support of rib due to flexibility of the floor beams generally reduces these moments, therefore is generally neglected.

2.7. Bending moment modification at floor-beam

The wheel loads applied to the continuous rib on rigid supports induce a reaction F_0 as given in Eq. 10. If the floor beams or supports are flexible those reactions are modified, induce a modified floor-beam moment as given in [14].

$$\Delta M_f = Q_0\left(\frac{L}{\pi}\right)^2 \frac{Q_{ix}}{Q_0}\left[\frac{F_0}{P} - \sum \frac{F_0}{P}\gamma_m\right] \quad (23)$$

where $\frac{F_0}{P}$ are the induced reactions, when floor beams are rigid, as given by Eq. 10,

γ_m are the influence lines for the reaction of the flexible floor beam.

3. Loading

The orthotropic bridge should be designed for those loads that are generally considered for most highway bridges:

- *dead load,*
- *live load,*

- *impact load.*

The dead load consists of the dead weight of the bridge system including the wearing surface. The live load is in accordance with the various standards, Eurocode 3, Part 3, AASHTO, Japan Road Association, Hungarian Road Association, etc. (**Fig. 9**). A 40 tons truck loading is taken into account.

The impact factor can be as follows:

$$\mu = 1.05 + \frac{5}{L+5} \leq 1.4 \tag{24}$$

where L is the span length of the bridge in m.

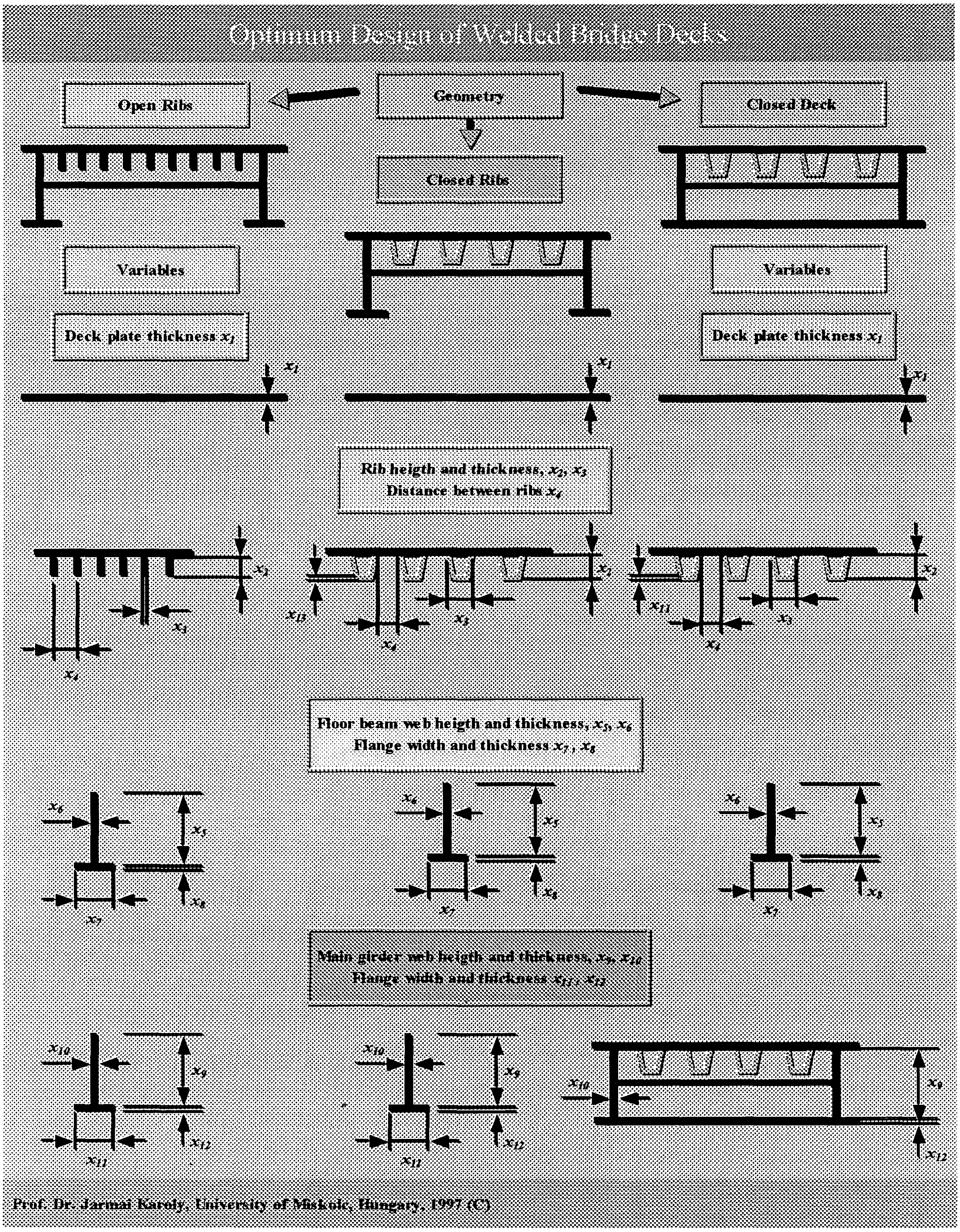

Fig. 11. Different geometry of bridge decks

Table 1. The calculated stresses in the ribs, floor-beams and main girder

	Rib	x direction			Main girder	Floor-	beam	y direction	
	at	midspan	at	support	x direction	at	midspan	at	bottom
	at top	at bottom	at top	at bottom	lower flange	at top	at bottom	at top	at bottom
System Rigid	Y	Y	Y	Y		Y	Y	Y	Y
II Elastic	Y	Y	-	-		Y	Y	-	-
System III	Y	Y	Y	Y	Y	-	-	-	-
Linear summary	Y	Y	Y	Y	Y	Y	Y	Y	Y

4. Constraints

The constraints are needed for the optimization. These constraints concern most of the important mechanical parameters and sizes of the structure, like stress, local and overall stability, fatigue, deflection. The optimization system is visible in **Fig. 10.** The different geometry of the deck plate can be seen in **Fig. 11.**

Stress constraints

From the bending moments we can calculate the stresses in the ribs at midspan and at the support, both in rib top and rib bottom. The same for the floor-beams, where we can calculate the stresses at midspan and at the support, both in floor-beam top and floor-beam bottom. Also in the main girder in the lower flange. **Table 1.** shows the stresses, which were calculated.

It is important to calculate the reduced stresses, because they represent the yielding condition in the plate. At midspan of rib in bridge axis (midspan of floor-beam) and at midspan of rib, at support of floor-beam (over main girder) the reduced stress is as follows:

$$\sigma_r^{(1)} = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x \sigma_y + 3\tau_{xy}^2} \qquad (25)$$

The shear stress is neglected, $\tau \approx 0$ and σ_x, σ_y are stresses in the floor beam upper flange.

Shear stress of the floor beam, because the floor beam web height is reduced by the rib height due to cut

$$\tau = \frac{F}{(h_f - h_r)t_f} \le \tau_{adm} \qquad (26)$$

In our optimization we take into account the stresses in rib-midspan-top, in rib-support-bottom, main girder-lower-flange, in floor-beam-midspan-bottom, in floor-beam-support-bottom and the two reduced stresses. So the number of stress constraints is seven.

Stability constraints

Overall stability of the orthotropic deck plate

When the deck plate is connected to the upper flange of the main girder, the calculation of the overall stability of the orthotropic deck plate requires the calculation of the average compression force in the main girder and comparison with the ultimate compression force according to the standards.

From main girder action the average compression force from the main girder action (system III)

$$\frac{N_r}{A_r} \le \varphi_k \sigma_{all} \qquad (27)$$

where

$$N_r = \bar{\sigma} A_r \qquad (28)$$

$\bar{\sigma}$ is the average stress in the rib,

A_r is the area of the rib

The slendernesses are

$$\lambda_1 = L \sqrt{\frac{A_r}{I_{rx} + \frac{a_0 t^3}{11}\left(2\frac{L^2}{4B^2} + \frac{L^4}{16B^4}\right)}} \qquad (29)$$

$$\lambda_2 = L \sqrt{\frac{I_r + I_z}{0.04 L^2 I_r + h^2 I_z}} \qquad (30)$$

From the slendernesses the j_k coefficients can be calculated according to standards.

According to the Hungarian Standard, the calculation of φ_k, using the curvefit program is the same as above.

From main girder action plus direct load on rib

The increment of excentricity

$$\frac{1}{\varphi_k'} \frac{N_r}{A_r} + \psi \sigma_t \le \sigma_{all} \qquad (31)$$

$$\frac{N_r}{A_r} + \psi \sigma_b \le \varphi_k' \sigma_{all} \qquad (32)$$

where $$\psi = \frac{1}{1 - 1.15\frac{N_r}{A_r \sigma_{all}}\left(\frac{\lambda_1}{100}\right)^2} \qquad (33)$$

σ_t and σ_b are the stresses at the top and bottom of rib.

Local stability, local buckling

The calculation of the local stability of the orthotropic bridge deck plate elements require calculation of the slenderness of the different parts, like rib (height/thickness), deck plate between ribs, floor-beam web and flange, main girder web and flange and comparison with the ultimate limit slenderness of the plate according to the steel grade, loading and standards[32,33,34].

Stability control according to BS 5400

Stability control for simple plate ribs

$$\frac{h_r}{t_t}\sqrt{\frac{f_y}{355}} \le 10 \qquad (34)$$

where h_r and t_r the height and thickness of the rib.

Stability control for floorbeam web

$$\frac{h_f}{t_f}\sqrt{\frac{f_y}{355}} \le 31 \qquad (35)$$

and

$$\frac{h_f t_f}{t_r^2}\sqrt{\frac{f_y}{355}} \le 10 \qquad (36)$$

where h_f and t_f the height and thickness of the floor beam.

Stability control for main girder web

If there are two longitudinal stiffeners on the main girder web the limit slenderness is as follows

$$\frac{h_m}{t_m} \le 310\sqrt{\frac{f_y}{355}} \qquad (37)$$

where h_m and t_m the height and thickness of the main girder.

Fatigue constraints

The fatigue of the welded joint is very important part of constraints. We had to take into account the most dangerous welded joint fatigue according to the steel grade, loading and standards [32,33,35]. The S-N curve due to [35] is visible on **Fig. 12.** for normal stresses. The Classes of the different welded joint can be seen on **Fig. 13.**

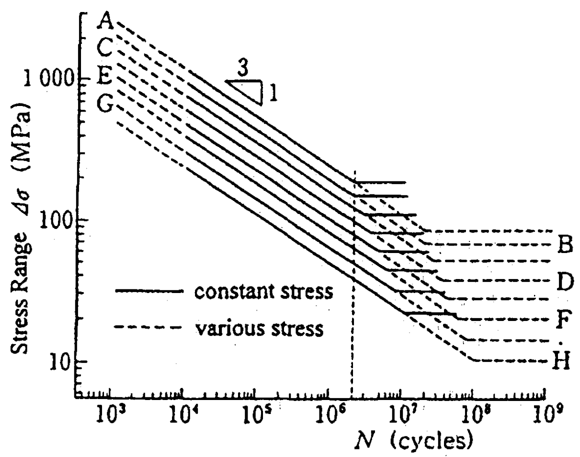

Fig. 12. S-N curves for normal stresses

② the stress in the rib at support, bottom
Class D $\Delta\sigma = 100$ MPa

③ the stress in the rib at midspan, top
Class E $\Delta\sigma = 80$ MPa

④ the stress in the floor beam at support, bottom
Class E $\Delta\sigma = 80$ MPa

⑤ the stress in the floor beam at midspan, bottom
Class D $\Delta\sigma = 100$ MPa

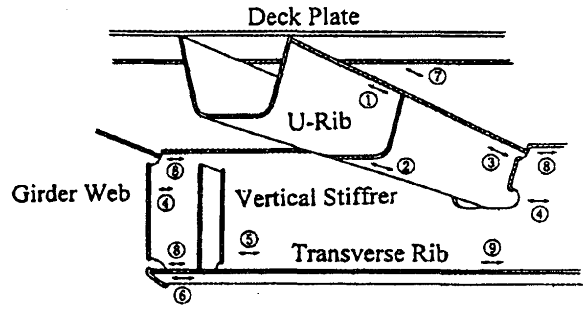

Fig. 13. Classes of deck plate welded joint

Deflection constraint

The deflection is composed of two parts, a global deflection of the bridge, which limit is usually given by standards and the local deflection between ribs, where there are only some suggestions. The local deflection of the plate between the ribs can be calculated according to Klöppel [31]

$$w_m = \frac{5}{6}\frac{1}{384}\frac{p a_r^4}{E I_d} \qquad (38)$$

where p is the wheel load unit pressure,

a_r is the rib spacing,

I_d is the moment of inertia of the deck plate,

$$I_d = \frac{t_d^3}{12} \qquad (39)$$

t_d is the deck plate thickness.

This formula is useful to calculate the minimum thickness of deck plate. If the deflection is limited in 1/300 of the span length (a), in this case for open rib the thickness is as follows

$$t_d \ge a\sqrt[3]{\frac{180}{23}\left(\frac{p}{E}\right)} \qquad (40)$$

5. Fabrication cost calculations

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, that are valid all over the world, because there are great differences between the cost factors in 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.

The cost function can be expressed as

$$K = K_m + K_f = k_m V + k_f \sum_i T_i \quad (41)$$

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.

5.1. Fabrication times for welding

Eq.(36) can be written in the following form

$$\frac{K}{k_m} = \rho V + \frac{k_f}{k_m}(T_1 + T_2 + T_3) \quad (42)$$

where

$$T_1 = C_1 \delta_d \sqrt{\kappa \rho V} \quad (43)$$

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

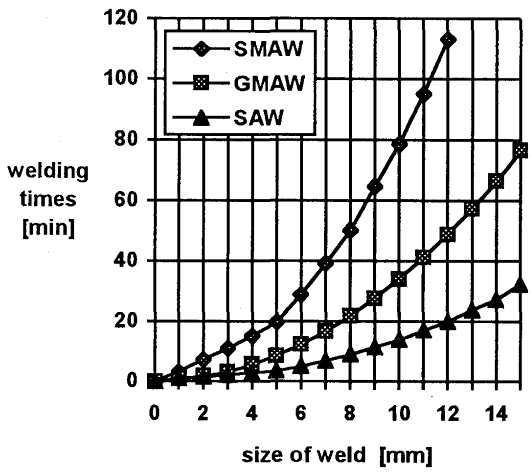

Fig. 14 Welding times for fillet welds of size a_w

$$T_2 = \sum_i C_{2i} a_{wi}^n L_{wi} \quad (44)$$

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.

$$T_3 = \sum_i C_{3i} a_{wi}^n L_{wi} \quad (45)$$

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*.

Table 2. Welding times T_2 (min) in function of weld size a_w (mm) for longitudinal fillet welds downhand position (see also Fig. 14.)

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	0.786 a_w^2
GMAW-C	2 - 5	1.70 a_w
	5 - 15	0.339 a_w^2
SAW	2 -5	1.190 a_w
	5 - 15	0.236 a_w^2

Table 3. 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	2 - 5	3.86 a_w
	5 - 15	1.139 $a_w^{1.758}$
GMAW-C	2 - 5	1.26 a_w
	5 - 15	0.144 $a_w^{2.348}$
SAW	2 -5	0.52 a_w
	5 - 15	0.178 a_w^2

Table 4. 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

Ott and Hubka [15] proposed that $C_{3i} = 0.3\, C_{2i}$, so

$$T_2 + T_3 = 1.3 \sum_i C_{2i} a_{wi}^n L_{wi} \quad (46)$$

Values of C_{2i} and n may be given according to *COSTCOMP* [16] as follows. It gives welding times and costs for different technologies [17]. 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. 14** and **Table 2-4** 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.

6. Numerical example

In order to show the effect of various welding methods on the optimal sizes and cost of welded structures, illustrative numerical examples are worked out and the structural versions optimized for various welding methods are compared to each other. We made the optimum design of a simply supported bridge deck with the following data:
span L = 50 meters, mild steel, yield stress f_y = 240 MPa, 400 kN truck loading, the admissible normal stress R_{adm} = 160 MPa, and admissible shear stress $\tau_{adm} = \frac{R_{adm}}{\sqrt{3}}$

Uniformly distributed loading p_{dist} = 4 kN/m^2, Tire width $2g$ = 600. mm, contact tire length $2c$ = 200. mm density of steel is ρ_{Steel} = 7.85*10^{-6} kg/mm^3, density of the asphalt is $\rho_{asphalt}$ = 2.4*10^{-6} kg/mm^3, thickness of asphalt is 60 mm.

Welding parameters: C_1 = 1, δ= 2, downhand welding, $C_3 = 0.3C_2$, the cost ratio is $\frac{k_f}{k_m}$ = 0, 0.5, 1, 1.5, 2 respectively, C_2 = 0.786 for *SMAW*, 0.339 for *GMAW-C*, 0.236 for *SAW*, for fillet welds, working thickness is a_w = 0.5t for fillet welds. If the two plate thicknesses, which has been welded are t_1 and t_2 and $t_1 < t_2$, then

$$t_1 \le a_w \le \sqrt{2t_2} \qquad (47)$$

The specific material cost is k_m = 0.5 - 1.2 \$/kg, 57 - 137 Yen/kg respectively, the specific labour cost is k_f = 15 - 60 \$/manhour, 1700 - 7000 Yen/manhour respectively.

The unknowns variable values are as follows:

$x(1)$ is the deckplate thickness,
$x(2)$ is the rib height,
$x(3)$ is the rib thickness,
$x(4)$ is the distance between ribs,
$x(5)$ is the floor-beam web height,
$x(6)$ is the floor-beam web thickness,
$x(7)$ is the floor-beam flange width,
$x(8)$ is the floor-beam flange thickness,
$x(9)$ is the main girder web height,
$x(10)$ is the main girder web thickness,
$x(11)$ is the main girder flange width,
$x(12)$ is the main girder flange thickness.

The objective function was the mass and the cost of the structure, there were seven static stress constraints at rib, floor-beam, main girder, two overall and six local buckling constraints and four fatigue constraints. Mild steel has been used, with fillet weld and *SMAW* technique.

At the original conventional-calculation of the bridge deck the floor-beams were well balanced, because there was not too great difference in moments at midspan and support. **Table 5.** shows the results of the conventional calculations and the optimization. The computer program is written on PC in Fortran Power Station. A screen of the computer code is visible on **Fig. 15.** The runtime is about a minute with a Petium 166 MHz processor.

Table 5. The results of the conventional calculations and the optimization of welded bridge deck with open ribs

	$x(1)$ mm	$x(2)$	$x(3)$	$x(4)$	$x(5)$	$x(6)$	$x(7)$	$x(8)$	$x(9)$	$x(10)$	$x(11)$	$x(12)$	Weight kg/m
Conventional calculation	12	240	20	300	800	12	200	25	4000	16	800	50	2179
Optimum design													
k_f/k_m= 0	12	250	25	392.3	470	9	580	11	4160	14	530	22	1858
k_f/k_m= 0.5	13	260	25	463.63	470	9	590	11	4180	14	510	22	2658
k_f/k_m= 1	14	250	28	510	480	9	610	10	4150	14	540	22	3923
k_f/k_m= 1.5	16	250	28	566.7	490	9	550	11	4180	14	550	20	4166
k_f/k_m= 2	17	240	32	637.5	470	8	610	11	4180	14	520	22	6131

If we consider the weight of the original design to be 100 %, then the optimum design for weight is 85 % (**Table 6.**). If we increase the cost ratio k_f/k_m the cost savings are different, between 15 and 31 %, because of the discretization only limited values are available. The average cost reduction is approximately 20 %. The greatest effect is on the rib distance. It is increasing with the k_f/k_m ratio from 392.3 mm up to 637.5 mm. It means, that higher fabrication cost decreases the number of ribs, lower fabrication cost increases it as we have described in[20]. If we increase the steel grade, we can reduce some more weight but later on the weight is also increasing with the strength. The minimum is about at 500-550 MPa tensile stress steel.

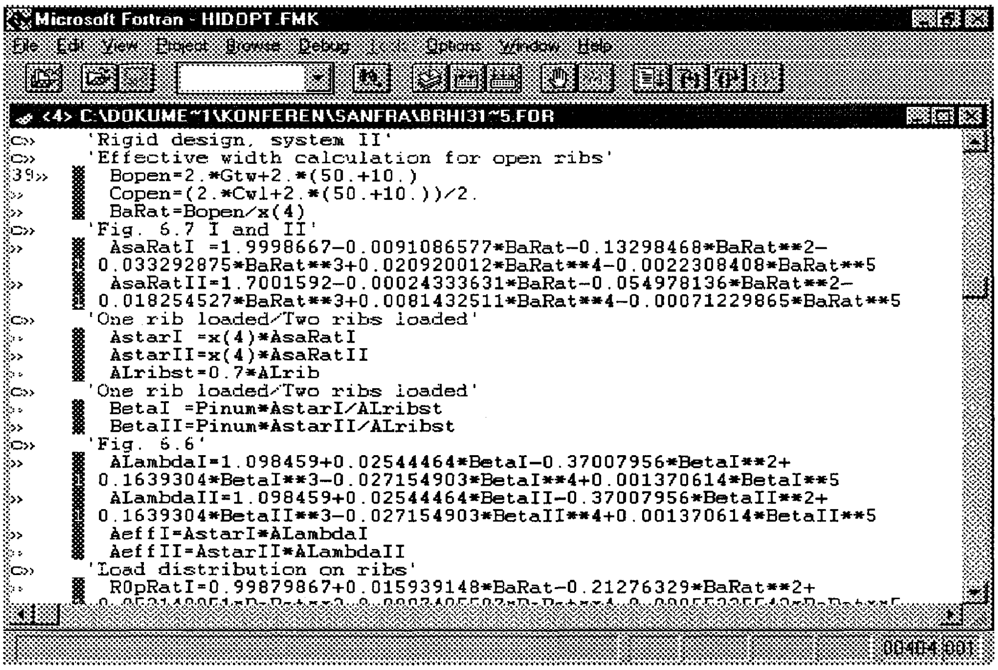

Fig. 15. Computer program in Fortran Power Station for the optimum design

Table 6. Cost savings due to different cost parameters

	Conventional calculation	Optimum design	
	Weight in kg/m	Weight in kg/m	Savings in cost %
k_f/k_m= 0	2179	1858	14.74
k_f/k_m= 0.5	3474	2658	23.48
k_f/k_m= 1	4769	3923	17.7
k_f/k_m= 1.5	6065	4166	31.3
k_f/k_m= 2	7361	6131	16.7

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