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OPTIMUM DESIGN OF WELDED BRIDGES

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Abstract
The three main phases of the optimum design of structures are treated. A survey of selected literature shows examples of the optimization of welded plate girders, trusses, cable-stayed bridges, Vierendeel beams and belt-conveyor bridges. The minimum cross-sectional area design of welded unstiffened I-beams subject to bending and shear is worked out according to Eurocode 3.

1 INTRODUCTION

Optimum design is a structural synthesis which collects all important engineering aspects to develop up-to-date structural versions not only safe but also economic. The economy is achieved by minimizing a cost function and the safety is guaranteed by fulfilling the design constraints. Thus, the problem can be defined mathematically as a constrained function minimization task which may be solved by mathematical programming methods.

The optimum design procedure can be divided into 3 main phases as follows:

1) preparation: defining structural variants by selection of materials, profiles, types of structure, fabrication and erection methods, formulation of design constraints and the cost function;
2) constrained function minimization by computerized mathematical methods;
3) engineering evaluation of computed results, comparison of optimal versions, working out design rules and expert systems.

In the case of welded bridges the following details can be mentioned.
Materials: steels including high-strength steels, aluminium alloys, fibre-reinforced plastics, reinforced concrete as the material for deck plates;
Profiles: rolled, welded, thin-walled open and hollow (tubular) sections, extruded aluminium sections;
Types of structure: statically determinate and indeterminate continuous girders, triangulated and Vierendeel trusses, frames, arches, shells, prestressed structures, suspension and cable-stayed bridges;

Applications: highway and railway bridges, footbridges, military bridges, bridges for belt-conveyors and pipelines;
Fabrication technology, joints: welding, bolting, riveting, gluing;
Design constraints: limitation of stresses and deflections as well as vibrations, constraints on overall and local buckling, fatigue, requirements of fabrication, transport, erection, inspection, maintenance and aesthetics. In most bridges moving loads should be considered.

Objective (merit) functions: the weight or cost can be defined. The difficulty is that the cost of fabrication, erection, maintenance is in many cases not available or it is difficult to define it mathematically. Therefore predominantly the weight is minimized. It should be mentioned that, in many cases, a cost function is defined by multiplying the total structural weight with a cost factor. In this case the minimal cost solution coincides with that of minimal weight. In cases when the material and fabrication costs have the opposite tendency, i.e. are conflicting functions, they should be separated to achieve a realistic optimum. This is the case of a stiffened plate with welded longitudinal ribs: the optimal number of ribs is larger for minimal weight and smaller when the fabrication (welding) cost increases (see e.g. in [1] where a relatively simple fabrication cost calculation method is treated).
In the case of a *multiobjective optimization* problem more objective functions are defined (see e.g. [2]).
The mathematical optimization methods are described e.g. in [3, 4].

2 SURVEY OF SELECTED LITERATURE

A detailed survey of structural optimization studies published after 1960 is worked out by *Cohn and Dinovitzer* [5]. A catalogue is given which shows the main characteristics of examples which can be found in 44 publications, mainly books. The model beams, frames, trusses and plates treated in published works may be applied to bridges. The characteristics of cited examples are as follows: sketch of the structural form, optimization level (section, member, structure, system, topological), loading (static or dynamic), materials (steel, concrete, composite steel-concrete), limit states (ultimate, serviceability), constraints (stress, deflection, cracking, fatigue, buckling, dynamics), objective function (single or multiple), computational method (e.g. analytic, branch and bound, dynamic programming, optimality criteria, gradient, sequential linear or quadratic programming, etc.). It is concluded that the number of actual engineering applications was very small and in order to broadening the practical range of application, more actual examples, easy-to-use software and expert systems should be available for designers.

Suruga and Maeda [6] have compared the cost and weight of more floor systems of suspension bridges concluding that the conventional reinforced floor system is cheaper but heavier than the steel plate deck. Steel plate floor system is advantageous especially in aspect of erection and overall economy.

Konishi and Maeda [7] have worked out the optimum design of simply supported welded I-section girders using the sequential linear programming method. In addition to material cost also the costs of drawing, machining, shop welding, shop assembly and shop painting have been considered. More welded splices of web and flanges have been calculated. Span lengths were between 16 and 30 m. The fabrication costs influence the number of different sections considerably. If the fabrication costs are higher, the number of different sections should be decreased.

In the *Farkas'* book [3] the minimum cost design of a simply supported welded I-beam with one welded splice on the flanges is treated by a numerical example solved by the backtrack programming method. The beams are subject to a static, uniformly distributed load. The objective function expresses the cost of materials, welding and painting. Constraints on maximal bending stress and on local buckling of web and compressed flange are considered. Checks for shear stress are also performed. Hybrid I-beams are constructed from two types of steels with different yield stresses. Constraints on lateral buckling and deflection are not taken into account.

Ferscha [8] has defined a cost function containing material and fabrication costs and, in addition, welding costs of butt welds of web splices and fillet welds connecting the flanges and the longitudinal stiffeners to the web. A systematic search method is used to obtain the optimal 4 variables (web height and thickness as well as cross-sectional areas of the two flanges).
According to *Durfee* [9] the triangular cross-section truss is an effective structural alternate to a rectangular truss of four main chord members (*Fig.1*).. An actual numerical example of a highway bridge is investigated. The truss depth is optimized to minimize the weight of the structure. The specifications of the American Association of State Highway and Transportation Officials (AASHTO) are used. Square hollow sections (SHS) are applied. The span length is 45.7 m, the width is 9.14 m. Welded joint details are discussed.

Adeli and Balasubramanyan [10] have developed an expert system for structural optimization and applied to different trusses. Three types of simply supported truss bridges have been treated (*Fig.2*). Computerized determination of influence lines and their evaluation for AASHTO moving loads is worked out. Minimum weight design is treated considering constraints on stress, buckling, deflection and size limitations. Standard rolled and tubular profiles are applied. The mathematical method of feasible directions is used.

As mentioned above, the fabrication cost plays an important role in the optimization of bridge structures. There are very few publications about the costs of robotic welding, therefore the article of *Touran and Ladick* [11] may be of interest. A detailed study on costs of robotic fabrication of an orthotropic steel bridge deck panel has been worked out on the basis of data obtained from five steel fabricators in USA. The investigated panel has had dimensions of 2.44*12.2 m and has been stiffened by trapezoidal ribs. The costs have been compared for conventional and robotic welding. Some characteristics are as follows:

traditional welding time/module	26.4 man-hours
robot welding time/module	9.0 man-hours
modules fabricated per year	667 modules
welding man-hours saved per year	11606 hours
labour rate including overhead and profit	$31.84/hour
wage savings	$369535/year

The welding operation, including consumables, makes up on average 14% of the total module fabrication cost, 38% is the material cost and 48% the additional labour cost (handling, preparation of material, stress relieving, grinding, coating, administration and equipment costs).

Taking into account the robot investment cost, it was concluded that 5.6% reduction in the fabrication cost resulted from the use of robotics in welding operation as compared to conventional approach.

In the article of *Memari et al* [12] the finite element method (FEM) has been used to generate influence surfaces. The CONMIN non-linear programming software is used for optimization of continuous highway bridges. This software developed by Vanderplaats is based on the feasible direction method. As objective function only the material cost is defined. A numerical example of a three-span (30.5-42.7-30.5 m) highway bridge with three welded steel plate girders and with a concrete deck plate is treated. The cost of the conventionally designed continuous steel plate girder bridges can be decreased by 25%, i.e. 20% by the use of FEM and 5% by optimization, since the calculation using FEM does not need to use the wheel load distribution factors.

Ohkubo and Taniwaki [13] have worked out the optimum design of cable-stayed bridges. The following parameters have been treated as constants: span length, number of cables, height and width of the main box girder and pylon as well as materials. The variables have been as follows: cross-sectional areas of cables, the reduced thicknesses of upper and lower flanges of each main girder element and pylon element, the distance from pylon to each cable anchor position in the main girder as well as the height of the lowest cable in the pylon from the axis of main girder. The reduced thicknesses consider also the longitudinal stiffeners. Design constraints on stresses in cables, in elements of the main girder and pylon have been considered according to the Japan Road Association specifications for highway bridges. The structure was analysed by FEM as a plane frame. The maximal and minimal bending moments, axial and shear forces due to traffic and impact loads have been calculated using influence lines. As objective function the material cost was defined.

The mathematical method was as follows. Constraints have been approximated by partial derivatives and reciprocal variables have been used. The approximate subproblem was solved using dual method with Lagrangian functions developed by Fleury et al. As a numerical example a three-span bridge has been treated. The number of variables and constraints has been 67 and 158, respectively. It has been shown that the cable positions (topology) have a great influence on the minimum material cost.

In the study of *Negrao and Simoes* [14] the dimensions of box-section pylons and I-section girders as well as the anchor positions of cables are selected as design variables. The optimum design is treated as a multiobjective optimization with goals of minimum cost of material, minimal stresses and displacements. The optimization method is combined with FEM code. Cable-stayed bridges are statically indeterminate and their structural behaviour is significantly affected by the cable arrangement and stiffness distribution among the cables, deck and pylons. Four numerical examples are worked out. It is shown that the minimum structural volume can be achieved by the modification of cable positions (*Fig.3*).

In the article of *Jármai and Farkas* [15] a parallel-chord belt-conveyor bridge is investigated to find the optimal topology for minimum volume. The number of columns and the truss height is varied. SHS members are designed for tension and overall buckling.

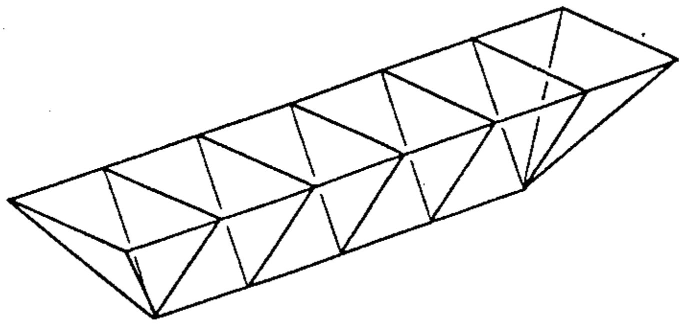

Fig. 1. Triangular truss optimized in [9]

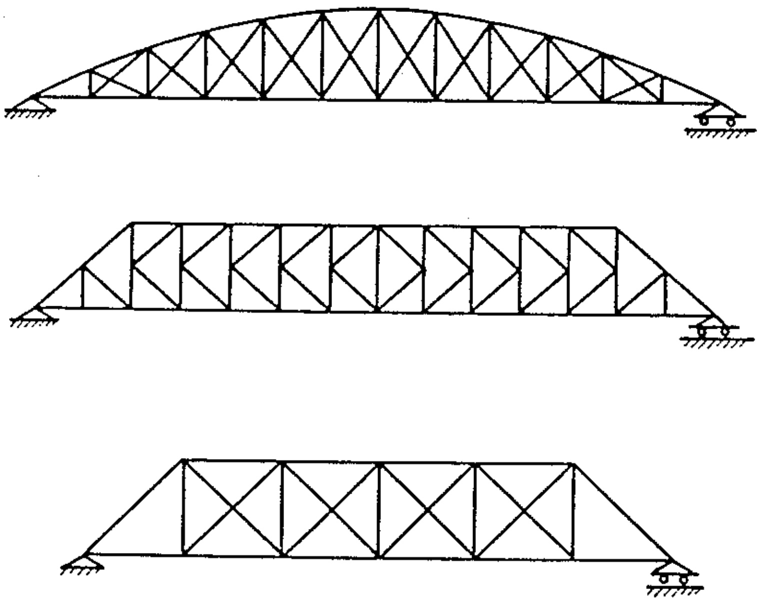

Fig.2. Parker-, K- and Pratt-type trusses optimized in [10]

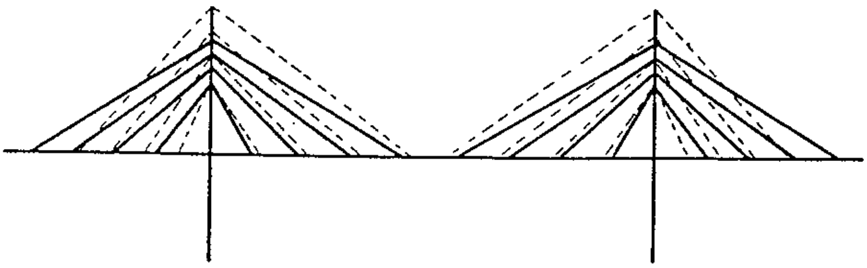

Fig.3. The initial (dashed) and optimal (continuous lines) cable topology of a cable-stayed bridge [14]

Farkas and Jármai [16] have worked out a detailed numerical example for the optimization of the height of a parallel-chord simply supported truss welded from circular hollow section (CHS) members with gap joints. The optimal height minimizes the total volume of the structure considering the stress, overall buckling and joint strength constrains. The truss is subject to a static, uniformly distributed load. It was pointed out that the stability constraints have a significant effect on the optimal version.

The minimum cost design of a simply supported SHS Vierendeel girder is worked out for uniformly distributed static loading by *Farkas and Jármai* [17]. Constraints on stress, local buckling and deflection are considered. The optimal number of bays are determined to minimize the material and welding costs.

3 OPTIMUM DESIGN OF WELDED I-AND BOX-BEAMS SUBJECT TO BENDING AND SHEAR

3.1 *Introductory remarks*

Simply supported welded I-and box-beams are predominantly loaded by bending and shear and, in most cases, it is sufficient to optimize their cross-section for pure bending and perform a check for shear. In cantilevers and continuous multispan girders the maximum bending moment and shear force can occur in the same section. In these cases the beam should be optimized for bending and shear. This problem has been treated already in the book [3], but now it is solved taking into account the Eurocode 3 (EC3) [18]. The problem is treated only for unstiffened I-and box-beams.

Note that the bending moment-shear force (*M-V*) interaction diagram given in the EC3 (*Fig.4*) contains a point A characterized by the plastic bending moment M_p, but we use the elastic bending moment M_e, since we treat here the elastic design of sections of class 3. As it will be shown the difference between M_p and M_e is small.

3.2 *Characteristics of optimal welded I-and box-sections subject to pure bending*

We treat the problem for welded I-sections, but, as it will be shown later, the derived formulae are valid also for box-sections, only small changes should be performed in the calculations.

The cross-sectional area is (*Fig.5*)

$$A = ht_w + 2bt_f \tag{1}$$

The approximate formula for the moment of inertia is

$$I_x = h^3 t_w / 12 + 2bt_f (h/2)^2 \tag{2}$$

The elastic section modulus is

$$W_{xe} \cong 2I_x / h = h^2 t_w / 6 + bt_f h \tag{3}$$

The plastic section modulus is expressed by

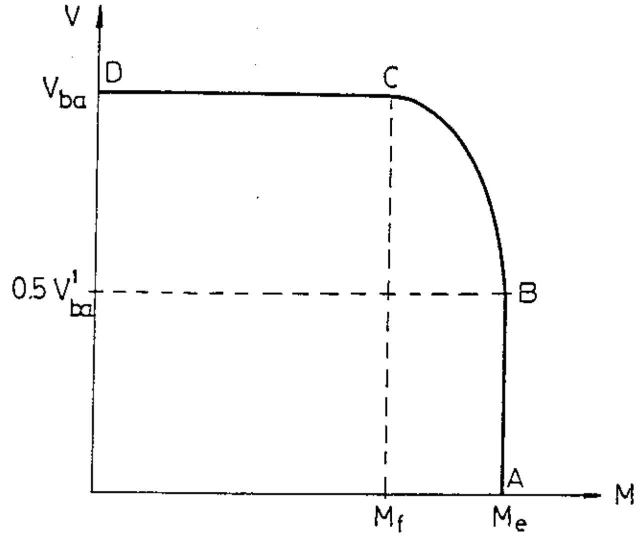

Fig.4. Bending moment-shear force interaction diagram according to EC3

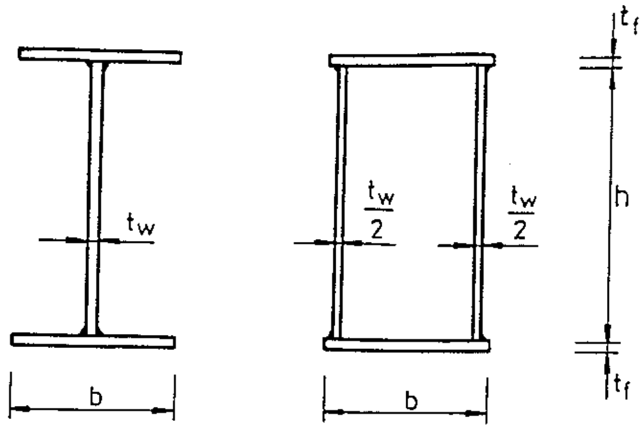

Fig.5. Cross-section of welded I-and box-beams

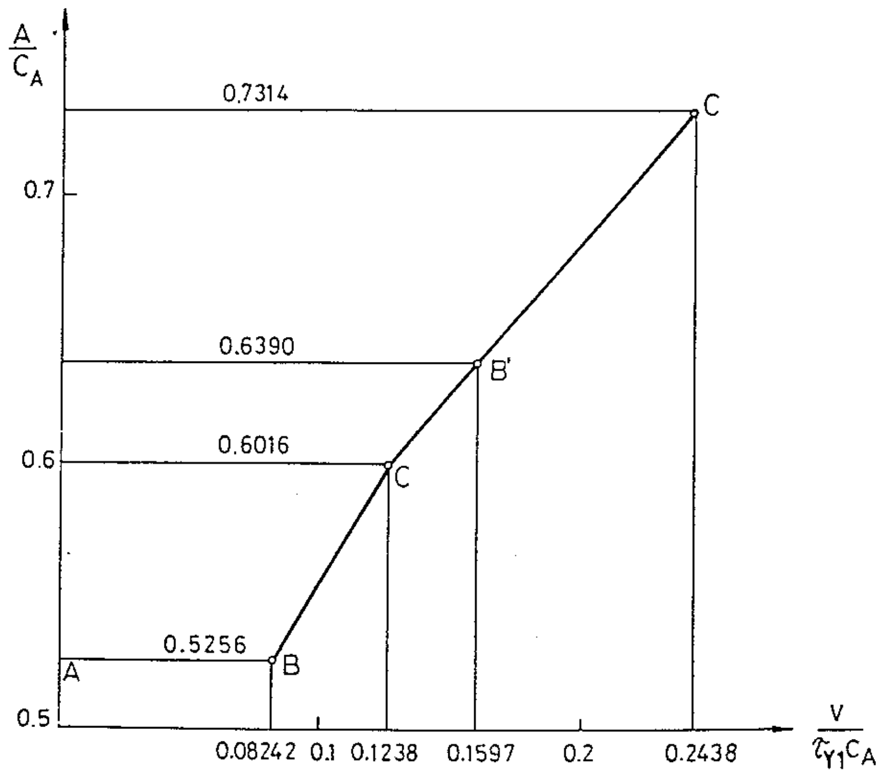

Fig.6. Minimal cross-sectional area of welded I-(and box-) beams in function of the shear force

$$W_{xp} \cong h^2 t_w / 4 + b t_f h \qquad (4)$$

We need also the section modulus of flanges

$$W_{xf} \cong b t_f h \qquad (5)$$

From Eq.(1) one obtains

$$b t_f = A/2 - h t_w / 2 \qquad (6)$$

Substituting Eq.(6) into (3), (4) as well as (5) a generalized formula for the section modulus can be written as

$$W_x = Ah/2 - \varsigma h^2 t_w / 2 \qquad (7)$$

where $\varsigma_e = 2/3, \varsigma_p = 1/2, \varsigma_f = 1$.

The stress constraint can be expressed as

$$W_x \ge W_0 = M / f_{y1} \qquad (8)$$

where W_0 is the required section modulus, $f_{y1} = f_y / \gamma_{M1}$, f_y is the yield stress, $\gamma_{M1} = 1.1$ is the partial safety factor.

The local web buckling constraint is expressed by

$$t_w \ge \beta h \qquad (9)$$

where the limiting web slenderness has the following values

for elastic design $1/\beta_e = 124\varepsilon$ (10a)

for plastic design $1/\beta_p = 72\varepsilon$ (10b)

Treating Eq.(9) as active, from Eq.(7) we get

$$A = 2W_0 / h + \varsigma \beta h^2 \qquad (11)$$

and the optimal web height to minimize the cross-sectional area can be calculated from the condition $dA/dh = 0$

$$h_{opt} = \sqrt[3]{\frac{W_0}{\varsigma \beta}} \qquad (12)$$

where $\varepsilon = \sqrt{235/f_y}$ (f_y in MPa).

Expressing W_0 from Eq.(12) and substituting it into (11) we get

$$A_{\min} = 3\varsigma \beta h_{opt}^2 = \sqrt[3]{27 \varsigma \beta W_0^2} \qquad (13)$$

This formula can be used for *comparison of elastic and plastic minimal cross-sectional areas*

$$\frac{A_{min.p}}{A_{min.e}} = \sqrt[3]{\frac{\varsigma_p \beta_p}{\varsigma_e \beta_e}} = 1.089 \quad (14)$$

i.e. the plastic design results in cross-sections by 8.9% larger than the elastic design. The plastic design is advantageous only in the optimum design of statically indeterminate structures.

The other optimal dimensions can be calculated from Eq.(6)

$$bt_f = \delta b^2 = 3\varsigma\beta h^2/2 - \beta h^2/2 = \beta h^2(3\varsigma - 1)/2 \quad (15)$$

$$b_{opt} = h_{opt}\sqrt{\beta/(2\delta)}\sqrt{3\varsigma - 1}\,, \qquad \delta = t_f/b, \qquad (1/\delta)_L = 28\varepsilon \quad (16)$$

3.3 *Minimum cross-sectional area design of welded I-and box-beams subject to bending and shear according to EC3*

We determine the optimal cross-sections for the different points or regions of the bending moment-shear force interaction diagram shown in *Fig.4* considering increasing values of the shear force acting in addition to a constant given value of the bending moment (or required section modulus). Note that the notation $0.5V'_{ba}$ is used since the actual numerical value of this shear force is not the half of V_{ba} (see the numerical example).The corresponding A_{min} -values are given in *Fig.6* in function of the shear force.

In the interaction diagram $V_{ba} = ht_w\tau_{ba}/\gamma_{m1} = ht_w\tau_Y$ (17)

The reduced web slenderness is

$$\bar{\lambda}_w = \frac{h/t_w}{37.4\varepsilon\sqrt{k_\tau}} = \frac{1}{86.4256\varepsilon\beta} \quad (18)$$

since for unstiffened webs $k_\tau = 5.34$. The web buckling diagram has three regions as follows:

$$\tau_Y = \tau_{Y1} = f_{y1}/\sqrt{3} \qquad \text{for} \qquad \bar{\lambda}_w \le 0.8 \quad (19a)$$

$$\tau_Y = \tau_{Y1}(1.5 - 0.625\bar{\lambda}_w) \qquad \text{for} \qquad 0.8 < \bar{\lambda}_w < 1.2 \quad (19b)$$

$$\tau_Y = 0.9\tau_{Y1}/\bar{\lambda}_w \qquad \text{for} \qquad \bar{\lambda}_w \ge 1.2 \quad (19c)$$

For elastic design with Eq.(11), according to (18) we get $\bar{\lambda}_w = 1.4348 > 1.2$ thus $\tau_Y = 0.6273\tau_{Y1}$. The region A-B is defined by

$$V \le 0.5V'_{ba} = 0.5\beta h^2 * 0.6273\tau_{Y1} \quad (20)$$

Using Eqs (10) and (11) and introducing the notation

$$C_A = \sqrt[3]{W_0^2/\varepsilon} \quad (21)$$

Eq.(20) can be written in the form

$V/(\tau_{Y1}C_A) \le 0.08242$ (22)

The corresponding A/C_A - value can be obtained from Eq.(13)

$A/C_A = 0.5256$ (23)

For the point C, where $V = V_{ba}$, using Eq.(20) and (10) with the value of $\varsigma_f = 1$ one obtains $V/(\tau_{Y1}C_A) = 0.1258$ (24)

and the corresponding A/C_A -value from (13) is $A/C_A = 0.6016$ (25)

Between points B and C a linear interpolation can be used according to *Fig.6*.

When $V/(\tau_{Y1}C_A) > 0.1258$ the value of $1/\beta = 69\varepsilon$ can be used instead of 124ε. In this case Eq.(18) gives the value of $\bar{\lambda}_w = 0.8$, thus, according to (19a), $\tau_Y = \tau_{Y1}$ and we calculate in Eq.(20) with τ_{Y1} instead of $0.6273\tau_{Y1}$. We get the points B' and C'. Between points C and B' a linear interpolation can also be used. The value of $1/(\beta\varepsilon)$ can also be linearly interpolated between 124 and 69. Since Eq.(13) can be written in the form

$$A/C_A = \sqrt[3]{27\varsigma\beta\varepsilon} \quad (26)$$

therefore, knowing A/C_A and $\beta\varepsilon$, the corresponding ς -value can be calculated from Eq.(26).

When $V/(\tau_{Y1}C_A) > 0.2438$, then the shear will be governing, thus from Eq.(17) we obtain

$$h_V = \sqrt{V/(\beta\tau_{Y1})} \quad (27)$$

and, with $1/(\beta\varepsilon) = 69$ and $\varsigma = 1$, using Eq.(13) we get

$$A_{min} = 3\varsigma\beta h_V^2 = 3V/\tau_{Y1} \quad \text{or} \quad A/C_A = 3V/(\tau_{Y1}C_A) \quad (28)$$

which means a straight line in the diagram. The corresponding flange width can be obtained from (16): $b_f = h_V\sqrt{\beta/\delta}$ (29)

The above derived formulae are valid also for a doubly symmetric welded box section with two webs of equal thickness $t_w/2$ with two exceptions as follows:
1) instead of β we should calculate in all formulae with 2β since the local web buckling constraint (9) relates to one web $t_w/2 \ge \beta h$ or $t_w \ge 2\beta h$; 2) in the local buckling

constraint of the compressed flänge the limiting flange slenderness in Eq.(16) is 42ε instead of 28ε. In *Fig.6* all values should be multiplied by 1.26.

Numerical example

Optimize a welded box beam with the following data: the bending moment is M = 630 kNm, f_y = 355 MPa, $\gamma_{M1} = 1.1$. W_0 = 1.9505*10^6 mm^3, $\varepsilon = \sqrt{235/355} = 0.8136$. $\tau_{Y1} = 186$ MPa

Point B in *Figs 4* and *6*: $1/\beta = 124\varepsilon, \varsigma = 2/3$. In this case, using Eqs (18) and (19) $\bar{\lambda}_w = 1.4348 > 1.2, \tau_Y = 0.9\tau_{Y1}/\bar{\lambda}_w = 116.7$ MPa.

$h = \sqrt[3]{0.75W_0/\beta} = 528$, rounded 530 mm, $t_w/2$ = 5.2, rounded 6 mm.

$b = h\sqrt{\beta/\delta}\sqrt{3\varsigma - 1} = 307$, rounded 310 mm, t_f = 9 mm.

The maximum shear force is $V_{max} = 0.5ht_w\tau_Y = 371$ kN.

Point C: the only change is that $\varsigma = 1$. $h = \sqrt[3]{W_0/(2\beta)} = 462 \to 470$, $t_w/2$ = 5, $b = h\sqrt{2\beta/\delta} = 380$, t_f = 12 mm. The maximum shear force is $V_{max} = ht_w\tau_Y = 548$ kN.

Point B': $1/\beta = 69\varepsilon = 56.14, \bar{\lambda}_w = 0.8, \tau_Y = \tau_{Y1} = 186$ MPa, $\varsigma = 2/3$.
$h = 435$, $t_w/2 = 8$, $b = 340$, t_f = 10 mm. V_{max} = 647 kN.

Point C': $\varsigma = 1$. $h = 380$, $t_w/2 = 7$, $b = 420$, t_f = 13 mm. V_{max} = 990 kN.

If the shear force is larger than 990 kN, e.g. V = 1200 kN, then we use Eq.(27) modified for box section: $h_V = \sqrt{V/(2\beta\tau_{Y1})} = 426 \to 430$, $t_w/2 = 8$, $b = h\sqrt{2\beta/\delta} = 470$, t_f = 14 mm. V_{max} = 1280 kN.

4. CONCLUSIONS

The research results cited from the selected literature show that the application of optimum design to bridge structures can result in significant cost and weight savings. To widen the application fields it is necessary to work out more easy-to-use software and expert systems. It is shown that the welded I-beams subject to bending and shear can be optimized according to EC3 by relatively simple closed formulae.

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