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Minimum cost design for fire resistance of welded steel structures

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Abstract
The optimum design is applied to cost minimization of two types of welded steel structures in fire. Both unprotected and protected structures are investigated. The compressed rod of welded square box cross-section is designed to overall and local buckling. The bent beam of welded box section should fulfil the stress, deflection and local buckling constraints. The cost function consists of cost of material, assembly, welding, painting and fire protection. In the unprotected case the critical temperature method is used with formulae given in Eurocode 3. At both structures the protected one is cheaper than the unprotected. This difference is caused by the significant difference in thicknesses.

Keywords
Structural optimization, welded structure, fire resistant design, stability, cost calculation

1. INTRODUCTION

Requirements for modern load-carrying structures are the safety, fitness for production and economy. In the optimum design procedure the safety and fitness for production are guaranteed by fulfilling of design and fabrication constraints, and the economy is achieved by minimization of a cost function.

It is possible to design a lot of structural versions. The most suitable version can be selected by cost comparison. For the purpose of economic design of welded steel structures a relatively simple cost calculation method is developed [1, 2, 3]. The cost function consists of cost of material, assembly, welding and painting.

Since the fire resistance of steel structures needs protection, the cost of various protection methods is also calculated using numerical data from industry.

The search for better solutions is performed by change of structural characteristics such as material, type of structure, profiles, main dimensions, fabrication technology and connections.

In general, the optimum design needs the solution of a constrained minimization of one or more objective nonlinear multivariable functions. Therefore the problems can only be treated numerically and the results are valid in general. In spite of this, when the numerical data of problems are selected as near as possible to industrial application, the results are very useful for designers to find the most economic and competitive structural versions.

In our research work we have worked out a lot of numerical problems of various metal structures. Our aim is to show how to apply the economic design for fire-resistant welded steel structures. The catastrophic damages and failures show that steel structures are very sensitive to high temperatures. Therefore special design rules have been elaborated in relevant Eurocodes [4, 5, 6], which are applied in the present paper.

Two numerical problems are solved as follows: (1) a centrally compressed rod of welded square box cross-section, (2) a welded box beam loaded in bending and shear.

Costs of unprotected and protected versions are compared to each other. Since only optimized versions can be compared, the both versions are optimized for minimum cost.

2. THE CRITICAL TEMPERATURE METHOD

Figure 1 shows the temperature versus time for fire gas and for a steel structure. The gas temperature can be calculated as

$$\Theta_g = 20 + 345 \log\left(\frac{8T}{60} + 1\right) \tag{1}$$

where T is the time in s.

The temperature of steel structure in a time interval is given by

$$\Delta\Theta_a = \frac{A_m}{V}\frac{h_{netd}}{c_a \rho_m}\Delta T \tag{2}$$

where c_a is the specific heat of steel,

$$c_a = 425 + 7.73x10^{-1}\Theta_a - 1.69x10^{-3}\Theta_a^2 + 2.22x10^{-6}\Theta_a^3 \tag{3}$$

ρ_m is the unit mass of steel, A_m/V is for rods of constant cross-section the ratio of perimeter/cross-section area, for a square box section

$$A_m/V = 1/t \tag{4}$$

The design value of the net heat flux per unit area is

$$h_{netd} = h_{netc} + h_{netr} \tag{5}$$

where the net convection heat flux is

$$h_{netc} = 25(\Theta_g - \Theta_a) \tag{6}$$

and the net radiative heat flux is

$$h_{netr} = 0.8x5.67x10^{-8}\left[(\Theta_g + 273)^4 - (\Theta_a + 273)^4\right] \tag{7}$$

$5.67x10^{-8}$ is the Boltzmann constant.

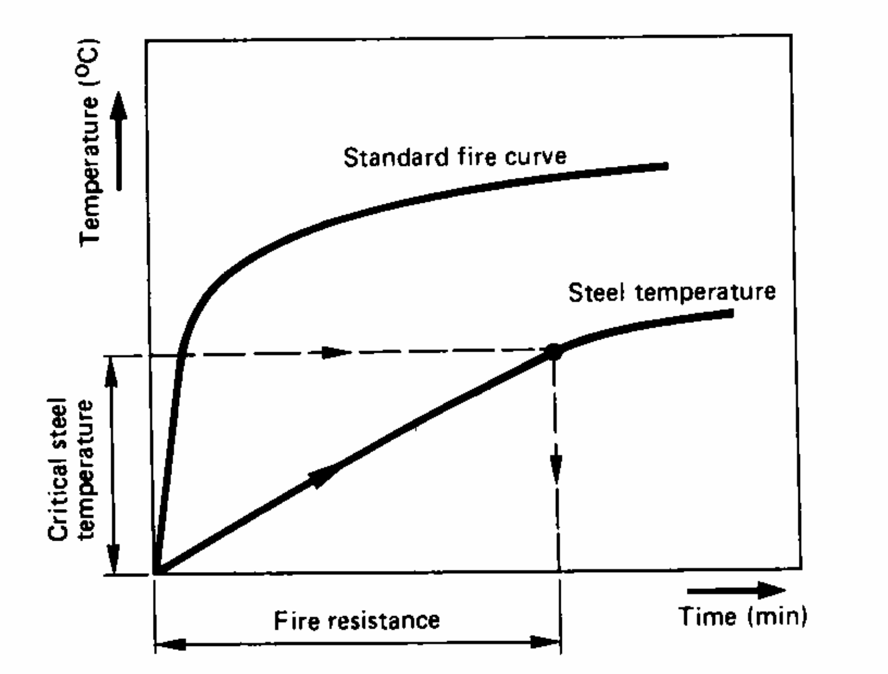

Figure 1. The critical temperature method

The critical temperature is given by

$$\Theta_{cr} = 39.19\,ln\left(\frac{1}{0.9674\mu_0^{3.833}} - 1\right) + 482 \tag{8}$$

where

$$\mu_0 = N_{fi}/N_0 \tag{9}$$

is the utilization factor, N_{fi} and N_o are the limiting compression forces in the case of fire and for ambient temperature, respectively.

The fire resistance time R corresponding to the critical temperature can be obtained by step-by-step using eqns. (1)-(9). Since until 600^0C the parameters in eqn. (2). can be approximated by three linear intervals, we use intervals of

$$\Theta_{a1} = \Theta_{cr}/3, \Theta_{a2} = 2\Theta_{cr}/3, \Theta_{a3} = \Theta_{cr} \tag{10}$$

In this case the final $R = \sum R_i$ is calculated by three iterations using a MathCad program.

$$\Delta R_i = \frac{\Theta_{ai} c_{ai} \rho_m}{6x10^4 h_{netdi}}, \; i = 1,2,3 \tag{11}$$

3. A CENTRALLY COMPRESSED ROD WITH PINNED ENDS OF WELDED SQUARE BOX CROSS-SECTION

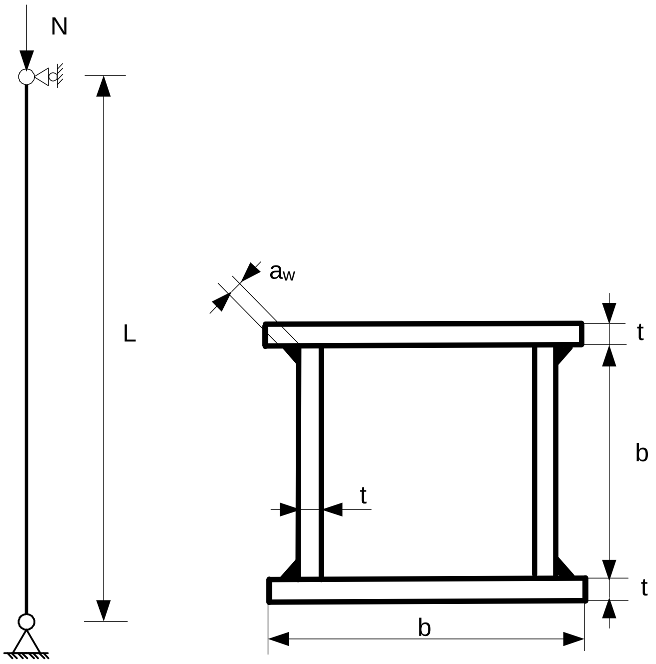

Figure 2. Compressed rod of welded square box section

3.1. Overall buckling constraint for ambient temperature

$$N \le N_0 \tag{12}$$

$$N_0 = \chi f_y A \tag{13}$$

the buckling factor

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}}, \phi = \frac{1}{2}\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right] \tag{14}$$

where

$$\bar{\lambda} = \frac{\lambda}{\lambda_E}, \lambda = \frac{L}{r}, r = \sqrt{\frac{I}{A}}, \lambda_E = \pi\sqrt{\frac{E}{f_y}} \tag{15}$$

In the case of a square box section

$$A = 4bt \tag{16}$$

$$I = \frac{2b^3 t}{3} + \frac{bt^3}{6} \tag{17}$$

For fire design $\alpha = 0.49$.

3.2. Overall buckling constraint in fire

$$N \leq N_{fi.t} \tag{18}$$

$$N_{fi.t} = \chi_{fi} A k_{y\Theta i} f_y / \gamma_{Mfi} \tag{19}$$

$\gamma_{Mfi} = 1$

$$\chi_{fi} = \frac{1}{\varphi_\Theta + \sqrt{\varphi_\Theta^2 - \overline{\lambda}_\Theta^2}}, \varphi_\Theta = \frac{1}{2}\left(1 + \alpha\overline{\lambda}_\Theta + \overline{\lambda}_\Theta^2\right) \tag{20}$$

$$\alpha = 0.65\sqrt{\frac{235}{f_y}}, \overline{\lambda}_\Theta = \overline{\lambda}\sqrt{\frac{k_{y\Theta i}}{k_{E\Theta i}}} \tag{21}$$

Factors of $k_{y\Theta i}$ and $k_{E\Theta i}$ can be approximated by linear intervals of

$$k_{y\Theta 0} = 1 \quad \text{if } 20^0\text{C} < \Theta_a < 400^0\text{C} \tag{22}$$

$$k_{y\Theta 1} = \frac{500 - \Theta_a}{100} 0.22 + 0.78 \quad \text{if } 400^0\text{C} < \Theta_a < 500^0\text{C} \tag{23}$$

$$k_{y\Theta 2} = \frac{600 - \Theta_a}{100} 0.31 + 0.47 \quad \text{if } 500^0\text{C} < \Theta_a < 600^0\text{C} \tag{24}$$

and

$$k_{E\Theta 0} = 1 \quad \text{if } 20^0\text{C} < \Theta_a < 100^0\text{C} \tag{25}$$

$$k_{E\Theta 1} = \frac{500 - \Theta_a}{400} 0.4 + 0.6 \quad \text{if } 100^0\text{C} < \Theta_a < 500^0\text{C} \tag{26}$$

$$k_{E\Theta 2} = \frac{600 - \Theta_a}{100} 0.29 + 0.31 \quad \text{if } 500^0\text{C} < \Theta_a < 600^0\text{C} \tag{27}$$

3.3. Local buckling constraint

For ambient temperature

$$b/t \leq 42\varepsilon, \varepsilon = \sqrt{235/f_y} \tag{28}$$

For fire Eurocode 3 proposed a decreased value of

$$b/t \leq 0.8x42\varepsilon = 33.6\varepsilon \tag{29}$$

According to the experiments of Knobloch [7] and calculations of Heidarpour & Bradford [8]

$$b/t \leq 0.6x42\varepsilon = 25.2\varepsilon \tag{30}$$

3.4. Cost function

The general formula for the welding cost is as follows [2, 3,4]:

$$K_w = k_w\left(C_1\Theta\sqrt{\kappa\rho V} + 1.3\sum_i C_{wi}a_{wi}^n C_{pi}L_{wi} \right) \tag{31}$$

where k_w [\$/min] is the welding cost factor, C_1 is the factor for the assembly usually taken as C_1 = 1 min/kg$^{0.5}$, Θ is the factor expressing the complexity of assembly, the first member calculates the time of the assembly, κ is the number of structural parts to be assembled, ρV is the mass of the assembled structure, the second member estimates the time of welding, C_w and n are the constants given for the specified welding technology and weld type, Cp is the factor of welding position (for downhand 1, for vertical 2, for overhead 3).

Furthermore C_{pi} is the factor for the welding position (download 1, vertical 2, overhead 3), L_w is the weld length, the multiplier 1.3 takes into account the additional welding times (deslagging, chipping, changing the electrode).

Material cost

$$K_m = k_m\rho V, V = AL, k_m = 1.0\$/kg \tag{32}$$

Welding cost for 4 fillet welds of GMAW-C (Gas metal arc welding with CO_2) [4]

$$K_w = k_w\left(C_1\Theta_c\sqrt{\kappa\rho V} + 1.3C_w a_w^2 L_w \right), k_w = 1.0\$/min : C_1 = 1.0 \text{ min/kg}^{0.5} \tag{33}$$

The factor of complexity of assembly is Θ_c = 2, number of assembled parts is κ = 4, fillet weld size a_w = 0.3t, welding coefficient C_w = 0.3394x10^{-3}, length of welds L_w = 4L.

Painting cost

$$K_p = k_p S, k_p = 28.8x10^{-6} \text{ \$/mm}^2, S = 4bL \tag{34}$$

Total cost

$$K = K_m + K_w + K_p \tag{35}$$

3.5. Numerical data and results

Centric compression force for fire N = 10^7 [N]. This load is calculated from the actual ones using a reduction factor η_{fi}. Rod length L = 6 m. Yield stress of steel f_y = 235 MPa.

The optimization is performed by a systematic search using a MathCad program. Results are given in Tables 1. and 2.

Table 1. Results for the unprotected structure for a fire resistance time R = 30 min. Optimum is marked by bold letters

b mm	t mm	$10^{-3}A$	K $	Θ_{cr} ^{0}C	R min	$10^{-7}N_{fi.T}$ [N]
500	38	76.00	5541	556	31.2	1.013
500	37	74.00	5372	551	30.2	0.977
510	**37**	**75.48**	**5451**	**555**	**30.5**	**1.003**
520	36	74.88	5359	554	29.9	0.856
530	35	74.20	5265	553	29.4	0.857

Table 2. .Results for protected structure for fire resistance time R = 60 min. Optimum is marked by bold letters. K is the cost according to eqn. (35) without the cost of protection. The result in the last row does not fulfil the local buckling constraint

b mm	t mm	$10^{-3}A$	K $	$10^{-7}N_{fitT}$ [N]	b/t
630	20	50.40	3385	1.012	31.5
660	**19**	**50.16**	**3357**	**1.014**	**34.7**
700	18	50.40	3361	1.028	38.9
720	17	48.96	3271	1.003	42.4

3.6. Cost including protection

The following approximate cost data are from Hungarian industry.

(a) Intumescent paint „Polylack" [9]

Cost factor k_{p1} = 60 $/m^2, superficies : S = 4x0.66x6 = 15.84 m^2

$K_{p1} = k_{p1}S = 950$ $

$K_1 = K - K_p + K_{p1} = 3357 - 456 + 950 = 3851$ $

Cost without protection K = 5451 $, thus, the cost savings is 29%.

(b) fire resistant plasterboard „Rigips" of thickness 12.5 mm [10]

Cost factor k_{p2} = 5.0 $/m^2, $K_{p2} = k_{p2}S$ = 79.0 $, labour cost K_L = 70 $

$K_2 = 3357 - 456 + 79 + 70 = 3050$ $

Cost without protection $K = 5451$ \$, thus, the cost savings is 44%.

4. A SIMPLY SUPPORTED UNIFORMLY LOADED WELDED BOX BEAM

Optimum design of this structure is treated for four cases as follows: unprotected and protected beam with stress or deflection constraint. In Equations the following subscripts are used: unprotected stress constraint σ, unprotected deflection constraint *w*, protected stress constraint σ1, protected deflection constraint *w1*.

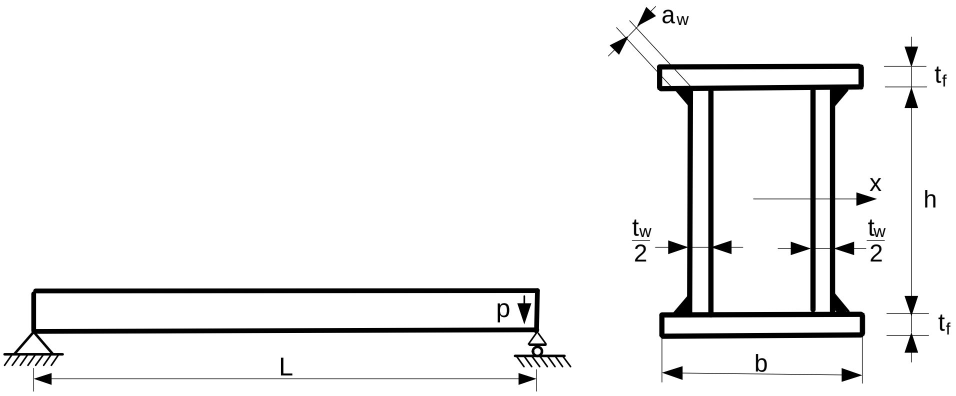

Figure 3. A simply supported welded box beam

4.1. Optimum design

It is sufficient to solve the optimization problem for minimum cross-section area instead of minimum cost, since the welding cost of four longitudinal fillet welds has no significant effect on the optimum beam dimensions.

The formulation of the optimum design of a box beam is as follows: find the optimum values of the dimensions h, t_w, b, t_f to minimize the whole cross-section area

$$A = ht_w + 2bt_f \tag{36}$$

and fulfil the following constraints:

(a) stress constraint

$$\sigma_{max} = \frac{M}{W_x} \leq f_{y1} \quad \text{or} \quad W_x \geq \frac{M}{f_{y1}} = W_0 \tag{37}$$

$$I_x = \frac{h^3 t_w}{6} + 2bt_f \left(\frac{h}{2}\right)^2 ; W_x = \frac{I}{h/2} = \frac{h^2 t_w}{3} + bt_f h \tag{38}$$

The bending moment is expressed as

$$M = p_s L^2 / 8, \quad (39)$$

the self mass of the sectorial plate is also taken into account, so

$$p_s = 1.5p + 1.1\rho_1 A, \ \rho_1 = 7.85\text{x}10^{-5} \text{ N/mm}^3. \quad (40)$$

(b) deflection constraint

$$w_{max} = \frac{C_w}{I} \le w_{adm} = \frac{L}{\phi}; \ C_w = \frac{5p_d L^4}{384E}; \ \phi = 300 \quad (41)$$

or

$$I \ge I_0 = \frac{5p_w L^4}{384Ew_{adm}} \quad (42)$$

For the deflection constraint the load intensity is calculated without safety factors, thus

$$p_w = p + \rho_1 A \quad (43)$$

(c) constraint on local buckling of webs

$$\frac{h}{t_w/2} \le \frac{1}{\beta}; \text{ or } \ t_w \ge 2\beta h \quad (44)$$

where $1/\beta = 69\varepsilon; 1/\beta_{fi} = 69\varepsilon\alpha_{fi}, \varepsilon = \sqrt{\frac{235}{f_y}}$ (45)

For unprotected beam $\alpha_{fi} = 0.6$, for protected one $\alpha_{fi} = 1$

(d) constraint for local buckling of compressed upper flange

$$\frac{b}{t_f} \le \frac{1}{\delta} = 42\varepsilon, \frac{1}{\delta_{fi}} = 42\varepsilon\alpha_{fi}, \ \alpha_{fi} = 0.6 \ \text{ or } \ t_f \ge \delta b \quad (46)$$

Considering the local buckling constraint as active the stress constraint can be written as

$$W = \frac{\beta h^3}{3} + bt_f h \ge W_0 \quad (47)$$

substituting bt_f from eqn. (36) into one obtains

$$A = \frac{2W_0}{h} + \frac{4\beta h^2}{3} \quad (48)$$

From the condition

$$\frac{dA}{dh} = 0 \quad (49)$$

one obtains the optimum value of h from the stress constraint

$$h_\sigma = \sqrt[3]{\frac{3W_0}{4\beta}} \quad (50)$$

Similarly from deflection constraint

$$h_w = \sqrt[4]{\frac{3I_0}{\beta}}; I_0 = \frac{\phi C_w}{L} \tag{51}$$

The advantage of this optimization method is that the other characteristics of the optimum cross-section can be expressed by h_σ or h_w. These characteristics are summarized in Table 3.

Table 3. Characteristics of optimum box sections

Stress constraint	Deflection constraint
$h_\sigma = \sqrt[3]{0.75 W_0 / \beta}$	$h_w = \sqrt[4]{3I_0 / \beta}$
$t_{w\sigma} / 2 = \beta h_\sigma$	$t_{ww} / 2 = \beta h_w$
$A_\sigma = 4\beta h_\sigma^2 = \sqrt[3]{36 \beta W_0^2}$	$A_w = 8\beta h_w^2 / 3 = \sqrt{64 \beta I_o / 3}$
$b_\sigma = h_\sigma \sqrt{\beta / \delta}$	$b_w = h_w \sqrt{\beta / (3\delta)}$
$t_{f\sigma} = \delta b_\sigma$	$t_{fw} = \delta b_w$
$I_{x\sigma} = 2\beta h_\sigma^4 / 3$	$I_{xw} = \beta h_w^4 / 3$
$W_{x\sigma} = 4\beta h_\sigma^3$	$W_{xw} = 2\beta h_w^3 / 3$

Numerical data

p = 90 N/mm, L = 15 m, f_y = 235 MPa, $f_{y1} = f_y/1.1$ = 213.6 MPa.

4.2. Optimum design of unprotected beam with stress constraint

Factored load in ambient temperature

$$p_\sigma = 1.5p + 1.1\rho_1 A_\sigma \tag{52}$$

Factored load in fire

$$p_{\sigma fi} = 1.5p + 1.1\rho_1 A_{\sigma fi}; A_{\sigma fi} = 4\beta_{fi} h_\sigma^2 \tag{53}$$

Bending moment for fire

$$M_{fi} = p_{\sigma fi} L^2 / 8 \tag{54}$$

Bending moment capacity in ambient temperature

$$M_0 = W_{x\sigma} f_{y1} \tag{55}$$

The utilization factor

$$\mu_0 = \frac{M_{fi}}{M_0} \tag{56}$$

The ratio of perimeter/cross-section area for a box beam

$$\frac{A_m}{V} = \frac{2(h_\sigma + b_\sigma)}{A_\sigma} = \frac{1+\sqrt{\frac{\beta}{\delta}}}{2\beta_{fi} h_\sigma} = \frac{61.41\varepsilon\alpha_{fi}}{h_\sigma} \tag{57}$$

The search for the optimum h_σ is performed according to section 2 using the critical temperature method. The result for fire resistance time R = 30 min is h_σ = 1230 mm. The other data for the beam are given in Table 4.

The maximum stress

$$\sigma_{\max\sigma} = \frac{M_{fi}}{W_{x\sigma}} \tag{58}$$

where $W_{x\sigma}$ is calculated according to eqn. (38).

The maximum deflection

$$w_{\max\sigma} = \frac{5 p_{wfi\sigma} L^4}{384 k_{E\Theta} E I_{x\sigma}} \tag{59}$$

where $k_{E\Theta}$ is calculated according to eqn. (27),

$$p_{wfi\sigma} = p + \rho_1 A_\sigma \tag{60}$$

For the cost calculation eqns. (32)-(35) are used with the following changes:

$$V_\sigma = A_\sigma L; S_\sigma = 2L(h_\sigma + b_\sigma), \quad a_{w\sigma} = 0.3\frac{t_{w\sigma}}{2} \tag{61}$$

The costs are calculated similar to eqns. (32, 33, 34) with the following differences:

$$K_m = k_m \rho V_\sigma, \quad K_p = k_p S_\sigma \tag{62}$$

4.3. Optimum design of the protected beam with stress constraint

The optimization is performed using Table 3**.** Thus, subscripts of σ1 are used. The optimum height of the beam is $h_{\sigma 1}$ = 990 mm.

$$\sigma_{\max\sigma 1} = \frac{p_{\sigma 1} L^2}{8 W_{x\sigma 1}}; p_{\sigma 1} = 1.5p + 1.1\rho_1 A_{\sigma 1} \tag{63}$$

$$w_{\max\sigma 1} = \frac{5 p_{w\sigma 1} L^4}{384 E I_{x\sigma 1}}; \quad p_{w\sigma 1} = p + \rho_1 A_{\sigma 1} \tag{64}$$

The costs are calculated similar to section 3.6.

It should be mentioned that the self mass of the protection can be neglected. (The specific mass of plasterboard protection Rigips of thickness 12.5 mm is 10.5 kg/m^2, and that of an intumescent painting of thickness 2 mm is 3.5 kg/m^2.)

The results are given in Table 4.

Table 4. Results for unprotected and protected beams with stress constraint. Dimensions in mm, stresses in MPa, costs in $

Unprotected	Protected
h_σ =1230	$h_{\sigma 1}$ = 990
b_σ =960	$b_{\sigma 1}$ = 775
$t_{w\sigma}$ = 60	$t_{w\sigma 1}$ = 30
$t_{f\sigma}$ = 38	$t_{f\sigma 1}$ = 19
$\sigma_{max\sigma}$ = 69	$\sigma_{max\sigma 1}$ =202
$w_{max\sigma}$ = 22	$w_{max\sigma 1}$ = 31
K_m = 17280	K_{m1} = 6965
K_w = 2670	K_{w1} = 870
K_p = 1892	K_{pro} = 3177, K_{pro1} = 476
K = 21840	K_1 = 11010, K_2 = 8311

Results in Table 4 show that the protected beam is much more cheaper than the unprotected one. The protection with plasterboard Rigips is cheaper than the Polylack painting.

4.4. Optimum design of unprotected beam with deflection constraint

Formulae in the right side column of Table 3 with subscript *w* are used. Eqns. (52)-(55) are used with subscript *w* instead of σ. eqn. (56) is changed to

$$\mu_0 = \frac{p_{wfi}}{p_w} = \frac{E_{fi} I_{0fi}}{EI_0} = \frac{k_{E\Theta}(600^0)}{\beta_1} = \frac{0.31}{0.6} = 0.517 \tag{65}$$

Eqn. (57) is changed to

$$\frac{A_m}{V} = \frac{2(h_w + b_w)}{A_w} = \frac{3\left(1+\sqrt{\frac{\beta}{3\delta}}\right)}{\beta h_w} = \frac{75.06\varepsilon\alpha_{fi}}{h_w} \tag{66}$$

The critical temperature according to eqn. (8) is 579^{0}C.

The optimum design procedure according to section 2 for fire resistant time R = 30 min results in h_{wopt} = 1500 mm.

In eqns. (58)-(62) the subscripts σ are changed to *w*.

The optimum beam dimensions and characteristics are summarized in Table 5.

4.5. Optimum design of the protected beam with deflection constraint

The optimization is performed using Table 3**.** Thus, subscripts *w1* are used. The optimum height of the beam is h_{w1} = 1050 mm. In eqns. (63) and (64) subscripts σ1 are changed to *w1*.
Results are given in Table 5.

Table 5. Results for unprotected and protected beams with deflection constraint. Dimensions in mm, stresses in MPa, costs in $

Unprotected	Protected
h_w =1500	h_{w1} = 1050
b_w =680	b_{w1} = 475
t_{ww} = 74	t_{ww1} = 32
t_{fw} = 27	t_{fw1} = 19
σ_{maxw} = 75	σ_{maxw1} =255
w_{maxw} = 21	w_{maxw1} = 37
K_m = 17390	
K_w = 3789	
K_p = 1884	
K = 23070	

It can be seen that the protected beam does not fulfil the stress constraint (255>213 MPa), thus the costs are not calculated for this case.

Comparison of Tables 4 and 5 shows that the deflection constraint results in a more expensive beam than that with the stress constraint.

Conclusions

A compressed rod of welded square box cross-section is designed for overall and local buckling. In the case of a simply supported beam of welded box section loaded by bending and shear the stress, deflection and local buckling constraints are considered.
In the optimization procedure the systematic search method is used with MathCad program. In the case of bent box beam the optimum dimensions are derived by an analytical method.
The cost function consists of the cost of material, assembly, painting and fire protection. Two types of protection are considered: intumescent painting Polylack and plasterboard Rigips.

The fire resistance time is 30 min for unprotected and 60 min for protected structures. The critical temperature method is suitable for the design using formulae given by Eurocode 3.
In the case of bent beam the structure for stress constraint is cheaper than that for deflection constraint.
In both structures the protected ones are much more cheaper than the unprotected ones. This difference is caused by the significant difference of section thicknesses.

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