DESIGN, FABRICATION AND ECONOMY OF WELDED STRUCTURES

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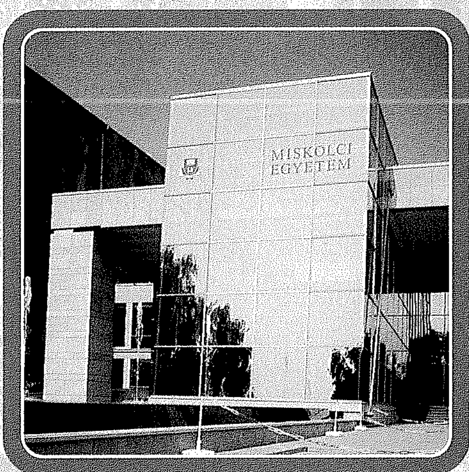

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2.4 Optimization of a Steel Frame for Fire Resistance with and without Protection

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Abstract

The main aim of the paper is to show the calculation and optimization of a steel frame according to Eurocode 1 and 3 with and without fire resistance. A comparison is made using square hollow section (SHS) columns and SHS or rectangular hollow section (RHS) for beams at a pressure vessel supporting frame (Figure 1). Optimizing for fire resistance for a given time it shows the prize of safety, the relation between mass and safety. To increase fire resistance we have to put more steel into the structure. A relatively new and promising optimization technique is introduced, the particle swarm optimization (PSO). In this evolutionary technique the social behaviour of birds is mimicked. The technique is modified in order to be efficient in technical applications. It calculates discrete optima, uses dynamic inertia reduction and craziness at some particles.

Keywords: *steel frames, fire resistance, structural optimization, evolutionary technique, particle swarm optimization*

1 Introduction

Fire research has tended to lag behind other fields of scientific and technological endeavour. This is due, no doubt partly to its extreme complexity but also due to the relatively low perceived importance of the topic in man's progress towards industrial development. Safety in general and fire safety in particular, after several major disasters, has become a subject of increasing importance in recent years. A general definition for the fire resistance of construction elements can be the following: the time after which an element, when submitted to the action of a fire, ceases to fulfil the functions for which it has been designed (Kay et al. 1996, Cox 1999, Rodrigues et al. 2000).

Steel structures have been used in industrial and residential buildings because they offer a wide range of advantages. However, these structures, when unprotected, behave poorly in fire situation. The high thermal conductivity of steel, together with the deterioration of its mechanical properties as a function of temperature, can lead to large deformations of structural elements and the premature failure of the buildings. The calculation of these steel frames can be according to Eurocode 1 and 3 (2005). The steel can be protected by materials such as mineral fibres, gypsum boards, concrete, intumescent paints and water-filled structures. In this study the optimal fire design of a steel frame structure is investigated. Using a relatively simple frame model it is shown how to apply the optimum design system for the case of fire resistance of a welded steel structure. Hollow sectional columns and beams are designed for minimum volume and weight. Overall and local buckling constraints are considered.

In the first design phase the structural mass is used as an objective function. A refined objective function is the material cost. A final objective function is the total cost including the steel mass, fabrication, fire protection technique costs.

2 Calculation of the frame members

Beams are made of RHS or SHS, unknowns are h_2, b_2, t_{f2}, columns are made of SHS, unknowns are h_1, t_{f1}.
The cross-section area of a RHS beam profile with a height h, width b and thickness t, considering rounded corners of corner radius of $R = 2t$ and supposing that $b_2 = h_2/2$, using the formulae given by Eurocode 3 Part 1.3 (2005), can be calculated as

$$A_2 = 2t_2(1.5h_2 - 2t_2)\left(1 - 0.43\frac{4t_2}{1.5h_2 - 2t_2}\right), \tag{1}$$

For SHS column it is

$$A_1 = 4t_1(h_1 - t_1)\left(1 - 0.43\frac{2t_1}{h_1 - t_1}\right), \tag{2}$$

For RHS beams the second moments of area are as follows (Figure 3).

$$I_{x2} = \left[\frac{(h_2 - t_2)^3 t_2}{6} + \frac{t_2}{2}\left(\frac{h_2}{2} - t_2\right)(h_2 - t_2)^2\right]\left(1 - 0.86\frac{4t_2}{1.5h_2 - 2t_2}\right), \tag{3}$$

$$I_{y2} = \left[\frac{(0.5h_2 - t_2)^3 t_2}{6} + \frac{t_2}{2}\left(\frac{h_2}{2} - t_2\right)^2 (h_2 - t_2)\right]\left(1 - 0.86\frac{4t_2}{1.5h_2 - 2t_2}\right). \tag{4}$$

For SHS columns

$$I_{x1} = I_{y1} = \left[\frac{2(b_1 - t_1)^3 t_1}{3}\right]\left(1 - 0.86\frac{2t_1}{b_1 - t_1}\right). \tag{5}$$

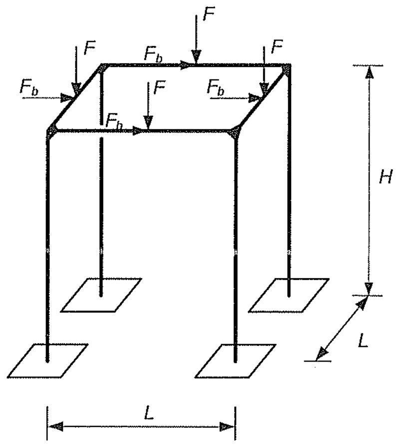

Figure 1. Supporting frame structure with vertical and horizontal forces

2.1 *Bending moments and forces from the vertical lo*ads F can be seen on Figure 2 and their calculations according to Glushkov et al. (1975) are as follows (Farkas & Jármai 1997, 2003)

$$H_A = \frac{3M_A}{H}; M_A = \frac{M_B}{2}; M_B = \frac{FL}{4(k+2)}; k = \frac{I_{y2}H}{I_{y1}L} \tag{6}$$

$$M_E = \frac{FL}{4} - M_B, \qquad M_1 = \frac{F_b H 3k}{2(6k+1)} \tag{7}$$

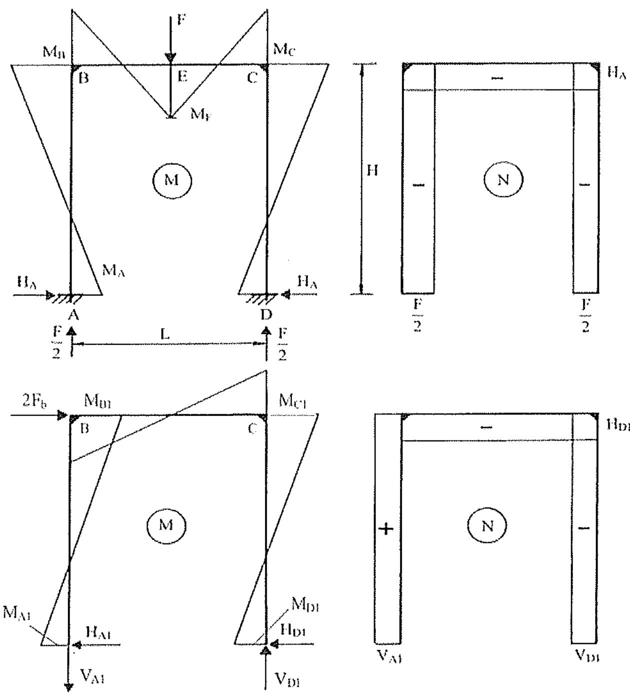

Figure 2. Diagrams for the bending moments and normal forces of a frame

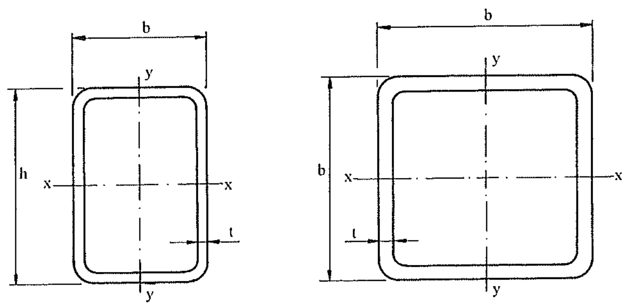

Figure 3. Dimensions of RHS and SHS profiles

Both members are made of hollow sections.

$$V_{D1} = \frac{2M_1}{L}, \qquad N_1 = F + V_{D1}, \quad H_{D1} = \frac{k+1}{k+2}\frac{F_b}{2}, \tag{8}$$

$$M_{A1} = \frac{3k+1}{6k+1}\frac{F_b}{2}H\,, \qquad M_{B1} = \frac{3k}{6k+1}\frac{F_b}{2}H\,, \tag{9}$$

$$H_2 = \frac{3k}{6k+1}H\,, \qquad M_{Bt} = M_B + M_{B1} \tag{10}$$

$$M_{At} = M_A + M_{A1} \tag{11}$$

2.2 *Bending moment in the horizontal frame due to horizontal force* F_b

The horizontal force is the tenth of the vertical one.

$$F_b = 0.1F\,, \qquad M_{Bz1} = \frac{F_b L}{4}\,, \tag{12}$$

$$M_{Bz2} = \frac{5F_b L}{64}\,, \qquad M_{Bz3} = \frac{F_b L}{64}\,, \tag{13}$$

$$M_{Bz} = M_{Bz1} - (M_{Bz2} + M_{Bz3}) \tag{14}$$

2.3 *The stress constraint for the* ***beam*** *(point E, no fire resistance) according to Eurocode 3, Part 1-2 (2005)*

$$\frac{H_A + H_{D1}}{\chi_{2.min} A_2 f_{y1}} + \frac{k_{yy2} M_E}{W_{y2} f_{y1}} + \frac{k_{yz2} M_{Bz}}{W_{z2} f_{y1}} \le 1\,, \qquad f_{y1} = \frac{f_y}{\gamma_{M1}} \tag{15}$$

The flexural buckling factor is

$$\chi_i = \frac{1}{\phi_i + \left(\phi_i^2 - \bar{\lambda}_i^2\right)^{0.5}}; \qquad \phi_i = 0.5\left[1 + 0.34\left(\bar{\lambda}_i - 0.2\right) + \bar{\lambda}_i^2\right] \tag{16}$$

$$\bar{\lambda}_{y2} = \frac{K_{y2} L}{r_{y2}\lambda_E}; \text{ the effective length factor is } K_{y2} = 0.5 \tag{17}$$

$$r_{y2} = \left(\frac{I_{y2}}{A_2}\right)^{0.5}; \lambda_E = \pi\left(\frac{E}{f_y}\right)^{0.5}; \ E \text{ is the elastic modulus} \tag{18}$$

$$\bar{\lambda}_{z2} = \frac{K_{z2} L}{r_{z2}\lambda_E}; \text{ the effective length factor is } K_{z2} = 0.5 \tag{19}$$

$$r_{z2} = \left(\frac{I_{z2}}{A_2}\right)^{0.5} \tag{20}$$

$\chi_{2.\min}$ is calculated from $\bar{\lambda}_{2.\max} = \max(\bar{\lambda}_{y2}, \bar{\lambda}_{z2})$.

$$C_{my2} = 0.4 \tag{21}$$

$$k_{yy2} = \min\left(C_{my2}\left(1 + \frac{0.6\lambda_{y2}(H_A + H_{D1})}{\chi_{y2} A_2 f_{y1}}\right), C_{my2}\left(1 + \frac{0.6(H_A + H_{D1})}{\chi_{y2} A_2 f_{y1}}\right)\right) \tag{22}$$

$$C_{mz2} = 0.4 \tag{23}$$

$$k_{zz2} = \min\left(C_{mz2}\left(1 + \frac{0.6\lambda_{z2}(H_A + H_{D1})}{\chi_{z2} A_2 f_{y1}}\right), C_{mz2}\left(1 + \frac{0.6(H_A + H_{D1})}{\chi_{z2} A_2 f_{y1}}\right)\right) \tag{24}$$

$$k_{yz2} = 0.8 k_{yy2} \tag{25}$$

2.4 *The stress constraint for the* ***beam*** *(point E, with fire resistance) according to Eurocode 1, Part 1-2 (2005)*

Member with Class 3 cross-sections, subject to combined bending and axial compression

$$\frac{H_A + H_{D1}}{\chi_{2.min} k_{y,\Theta} A_2 f_{y1}} + \frac{k_{yy2} M_E}{W_{y2} k_{y,\Theta} f_{y1}} + \frac{k_{yz2} M_{Bz}}{W_{z2} k_{y,\Theta} f_{y1}} \le 1, \quad (26)$$

The value of $\chi_{i,min\,fi}$ ($i = 1,2$) should be taken as the lesser of the values of $\chi_{y,fi}$ and $\chi_{z,fi}$ determined according to:

$$\chi_{fi} = \frac{1}{\varphi_\Theta + \sqrt{\varphi_\Theta^2 - \overline{\lambda}_\Theta^2}} \quad (27)$$

with $\varphi_\Theta = \frac{1}{2}\left(1 + \alpha\overline{\lambda}_\Theta + \overline{\lambda}_\Theta^{\,2}\right)$, and $\alpha = 0.65\sqrt{\frac{235}{f_y}}$ (28)

The non-dimensional slenderness for the temperature Θ_a, is given by:

$$\overline{\lambda}_\Theta = \overline{\lambda}\left(\frac{k_{y,\Theta}}{k_{E,\Theta}}\right)^{0.5} \quad (29)$$

Due to the application of hollow section we need not consider the lateral torsional buckling.

$$k_y = 1 - \frac{\mu_y N_{fi,Ed}}{\chi_{y,fi} A k_{y,\Theta} \frac{f_y}{\gamma_{M,fi}}} \le 3 \quad (30)$$

with $\mu_y = (1.2\beta_{M,y} - 3)\overline{\lambda}_{y,\Theta} + 0.44\beta_{M,y} - 0.29 \le 0.8$ (31)

for beam $\beta_{M,y} = 1.4$, $k_z = 1 - \frac{\mu_z N_{fi,Ed}}{\chi_{z,fi} A k_{y,\Theta} \frac{f_y}{\gamma_{M,fi}}} \le 3$ (32)

with $\mu_z = (1.2\beta_{M,z} - 5)\overline{\lambda}_{z,\Theta} + 0.44\beta_{M,z} - 0.29 \le 0.8$, $\overline{\lambda}_{z,\Theta} \le 1.1$, $\beta_{M,z} = 1.4$ (33)

2.5 *Stress constraint for* ***columns*** *(point C, no fire resistance) according to Eurocode 3, Action on structures, Part 1-2*

$$\frac{N_1}{\chi_{1.min} A_1 f_{y1}} + \frac{k_{yy1}(M_C + M_{B1})}{W_{y1} f_{y1}} + \frac{k_{zz1}(M_C)}{W_{z1} f_{y1}} \le 1, \qquad C_{my1} = 0.4, \quad (34)$$

$$k_{yy1} = min\left(C_{my1}\left(1 + \frac{0.6\lambda_{y1}(H_A + H_{D1})}{\chi_{y1} A_1 f_{y1}}\right), C_{my1}\left(1 + \frac{0.6(H_A + H_{D1})}{\chi_{y1} A_1 f_{y1}}\right)\right), \qquad C_{mz1} = 0.4, \quad (35)$$

$$k_{zz1} = min\left(C_{mz1}\left(1 + \frac{0.6\lambda_{z1}(H_A + H_{D1})}{\chi_{z1} A_1 f_{y1}}\right), C_{mz1}\left(1 + \frac{0.6(H_A + H_{D1})}{\chi_{z1} A_1 f_{y1}}\right)\right), \qquad k_{yz1} = 0.8 k_{yy1}, \quad (36)$$

$$r_{y1} = \left(\frac{I_{y1}}{A_1}\right)^{0.5}; r_{z1} = \left(\frac{I_{z1}}{A_1}\right)^{0.5}; \overline{\lambda}_{y1} = \frac{K_{y1} H}{r_{y1}\lambda_E}; K_{y1} = 2.19; \overline{\lambda}_{z1} = \frac{K_{z1} H}{r_{z1}\lambda_E}; K_{z1} = 0.5 \quad (37)$$

$$\bar{\lambda}_{1.max} = max(\bar{\lambda}_{y1}, \bar{\lambda}_{z1}), \qquad \chi_{i.min} = \frac{1}{\phi_i + \left(\phi_i^2 - \bar{\lambda}_{i.max}^2\right)^{0.5}}; \tag{38}$$

$$\phi_i = 0.5\left[1 + 0.34\left(\bar{\lambda}_{i.max} - 0.2\right) + \bar{\lambda}_{i.max}^2\right] \tag{39}$$

2.6 *Stress constraint for* ***columns*** *(point C, with fire resistance) according to Eurocode 1, Action on structures, Part 1-2*

Member with Class 3 cross-sections, subject to combined bending and axial compression

$$\frac{N_1}{\chi_{1.min.fi} A_1 k_{y,\Theta} f_{y1}} + \frac{k_y(M_C + M_{B1})}{W_{y1} k_{y,\Theta} f_{y1}} + \frac{k_z M_C}{W_{z1} k_{y,\Theta} f_{y1}} \le 1 \tag{40}$$

Calculation of the parameters is according to Eqs. (27-33).

for column $\beta_{M,\psi} = 1.8 - 0.7\psi$, $\psi = -1$ (41)

Due to the application of hollow section we need not to consider the lateral torsional buckling.

3 Local buckling of plates

For the local buckling calculation we use limit slendernesses, given by *Eurocode 3, Action on structures, Part 1-2* (2005).

For the beam flange $\frac{b_2}{t_{f2}} - 3 \le 42\varepsilon$ (42)

For the beam web $\frac{h_2}{t_{w2}} - 3 \le 69\varepsilon$ (43)

For the column flange $\frac{b_1}{t_{f1}} \le 42\varepsilon$ (44)

For the column web $\frac{h_1}{t_{w1}} - 3 \le 42\varepsilon$ (45)

where for fire resistance design $\varepsilon = 0.85\sqrt{\frac{235}{f_y}}$ (46)

4 Calculation of temperature

For unprotected structure the calculation of temperature is as follows:

The time at the beginning of the fire is $t_i = 0$, and every time period: $\Delta t_i = 5$ we calculate it $t_i = t_i + \Delta t_i$ [sec], (47)

Chancing the time from $0 \le t_i \le t_{max}$ [sec], (48)

where t_{max} can be ½, 1, 1 ½, 2 , 4 hours, means 1800, 3600, 5400, 7200, 14400 [sec].

The temperature of the steel can be between 20 [°C] $\le \Theta_a \le$ 1200 [°C] (49)

Starting values for temperature and density are as follows:

$$\Theta_a = 20\,[^\circ C],\ \Delta\Theta_a = 0\,[^\circ C],\ \rho_m = 7850,\ \rho = 7.85x10^{-6} \tag{50}$$

The specific heat of steel can be calculated as a function of different temperature according to Eurocode.

The gas temperature in the vicinity of the fire exposed member (standard temperature-time curve)

$$\Theta_g = 20 + 345 \log\left(8\frac{t_i}{60} + 1\right) \text{ [°C]}, \quad (51)$$

The net *convection* heat flux

$$\dot{h}_{netc} = \alpha_c(\Theta_g - \Theta_a), \quad (52)$$

Where the coefficient of heat transfer by convection $\alpha_c = 25$ [W/m^{2}K]

The net *radiative* heat flux

$$\dot{h}_{netr} = \Phi \varepsilon_m \varepsilon_f \sigma \left[(\Theta_g + 273)^4 - (\Theta_a + 273)^4\right] \text{ [W/m}^2\text{]} \quad (53)$$

where the configuration factor $\Phi = 1$, the surface emissivity of the member $\varepsilon_m = 0.8$, the emissivity of the fire $\varepsilon_f = 1.0$, the Stephan Boltzmann constant $\sigma = 5.67x10^{-8}$ [W/m^{2}K^4],

The total net heat flux can be calculated as the sum of convection and radiative heat fluxes

$$\dot{h}_{netd} = \dot{h}_{netc} + \dot{h}_{netr} \quad (54)$$

$$A_m V_m = \frac{1}{10^{-3} t_2} \quad (55)$$

The temperature changing

$$\Delta\Theta_a = k_{sh} \frac{A_m V_m h_{netd} \Delta t_i}{c_a \rho_m}, \quad \text{where } k_{sh} = 1 \quad (56)$$

The surface temperature of the steel member

$$\Theta_a = \Theta_a + \Delta\Theta_a \quad (57)$$

7. Calculation of material properties

The calculation of the yield stress and Young modulus on higher temperature is according to Eurocode 1 (2005). Figure 4 shows the reduction factors in the function of temperature between 20 and 1200 C°.

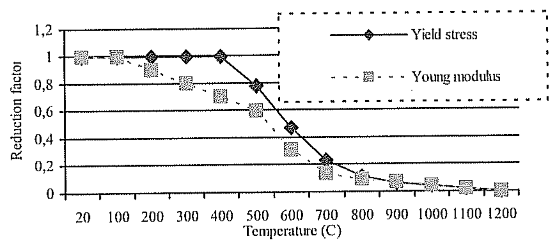

Figure 4. The yield stress and the Young modulus reduction factors in the function of temperature

7.1 *Calculation of yield strength*

The yield strength at a given temperature can be calculated by $k_{y,\Theta}$ reduction factor

$$f_{y,\Theta} = k_{y,\Theta} f_y \quad (58)$$

7.2 Calculation of Young modulus

The yield strength at a given temperature can be calculated by $k_{E,\Theta}$ reduction factor

$$E_{a,\Theta} = k_{E,\Theta} E_a \tag{59}$$

values of $k_{y,\Theta}$ and $k_{E,\Theta}$ can be calculated according to Table 1 and Figure 4.

The objective function is the mass of the frame to be minimized.

$$M = \rho(4HA_1 + 4LA_2) \tag{60}$$

Particle Swarm Optimization (PSO) techniques is used (Kenedy & Eberhardt 1995).

9 Numerical optimization results

9.1 Numerical data

The sizes of the frame are $H = 4000$, $L = 4000$ mm. The vertical and horizontal loads are $F = 75$ kN, $F_b = 0.1F$ for the normal design and $F = 0.74$x75 kN, $F_b = 0.1$ F for the fire resistant design. The Young modulus and the shear modulus and the yield stress $E = 2.1\text{x}10^5$ MPa, $G = 0.8\text{x}10^5$ MPa, $f_y = 355$ MPa respectively. The frame is a sway one with class 3 section.

The objective function is the structural mass M according to Eq. (60). *The unknowns* are the dimensions of SHS columns (b_1, t_1) and those of RHS beams (h_2, t_2). If SHS beams are taken into account, the formulae for SHS columns should be used with subscript 2 and their unknowns are b_2 and t_2.

Fabrication limitation

$$b_2 = \frac{h_2}{2} \le b_1 \tag{61}$$

To ease the fabrication, the solution of $b_2 = b_1$ is recommended. In this case the number of unknowns is 3.

9.2 Optimization results

Table 1 shows the optimum sizes of the frame, when we consider the same SHS section for the column and beam members, means 3 variables (SHS 3v), or different SHS sections for columns and beams, with 4 variables (SHS 4v), or different SHS and RHS sections for columns and beams, with 4 variables, assuming that the width of RHS section is the half of its height. We have used the tables of Dutta (1999) to get the available SHS and RHS sections. Both continuous (unrounded) and discrete optima have been calculated. The two different SHS sections version gives the best solution.

Table 1. Optimization results for the frame (no fire resistance has been taken into account)

Section		**h_1 (mm)**	**t_1 (mm)**	**h_2 (mm)**	**t_2 (mm)**	**K (kg)**
SHS 3v	discrete	180	5	-	4	775.57
SHS 4v	**discrete**	**200**	**5**	**150**	**4**	**765.53**
SHS-RHS 4v	discrete	180	5	200	5	782.24

For the frame with the same SHS section at columns and beams we have calculated the optima considering fire resistance. The fire resistance time vary from 225 sec up to 4500 sec. Both continuous and discrete optima have been calculated. Optima show that increasing the time of fire resistance, considerable increment of mass can be detected. If increase the time from 450 sec to 4500 sec (10 times more) we get an increment of mass from 1561 up to 4703 kg (3 times more). One more hour safety means three times more steel in the structure (Table 2).

Table 2. Optimization results for the frame (with fire resistance considerations)

Fire resistance time (sec)	h_1 (mm)	t_1 (mm)	t_2 (mm)	K (kg)
225	250	8	6.3	1699.19
450	250	8	6.3	1699.19
900	250	8	6.3	1699.19
1800	250	12	8	2317.63
2700	220	20	12	3028.55
3600	220	25	18	3865.90
4500	220	35	22	4703.10

Using intumescent painting, we can compare the efficiency of the painting. If we calculate the cost of the structure and consider only the material and the painting costs, than we can calculate on the following way:

$$K = K_m + K_p \tag{62}$$

$$K_m = k_m M, \tag{63}$$

where $k_m = 1$ \$/kg.

$$K_p = k_p A_p, \tag{64}$$

where $k_p = 14$ \$/m^2 which is the normal painting cost in two layers. The additional intumescent painting cost is either 20 \$/m^2, or 60 \$/m^2 depending on the fire safety time, which is half an hour, or one hour. A_p is the total painted surface

$$A_p = 16\, h_1 H + 16\, h_2 L \tag{65}$$

Table 3 shows the cost optimization results for the frame with and without intumescent painting. It shows, than in case of half an hour fire resistance the cost saving is about 5 %. In case of one hour resistance the cost saving can be about 27 %.

Table 3. Cost optimization results for the frame with and without intumescent painting

	Fire resistance time (sec)	h_1 (mm)	t_1 (mm)	t_2 (mm)	K (\$)
no protection	1800	250	12	8	2317.6
no protection	3600	220	25	18	3865.9
intumescent painting	1800	250	7	7	2210.8
intumescent painting	3600	230	10	6	2811.0

9 Conclusion

Optimization of steel frames for fire safety is a relatively new area. We have calculated the members of a high pressure vessel supporting frame without fire resistance. Using different cross sections (SHS, RHS) the mass of the frame is also different. The best solution occurred, when both columns and beams were made of SHS sections, with four variable sizes. When we consider fire resistance, the time after which its elements still work, needs more material (steel) to be built into the structure. The present example shows, that about 1 hour increment in fire safety needs 3 times more material in the structure. For a designer it is important to know the relation between mass and fire safety. Using intumescent painting and considering only the material and painting cost, the cost savings can be between 5 – 27 % depending on the fire resistance time. The applied optimization technique was very robust, the modified particle swarm optimization. It calculated both the continuous and discrete optima.

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