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OPTIMUM DESIGN OF VESSEL SUPPORTING FRAME FOR FIRE

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ABSTRACT: This article deals with the optimization of a vessel supporting frame for fire. A structure is economical, if its mass is the smallest as possible. Mass optimization with and without fire protection are discussed. Nowadays a lot of fire protection solutions are available, increase of the mass of steel, using higher yield stress steel, intumescent coating or fireboard protection are presented.

Keywords: optimum design, SOLVER-Excel, steel structures, fire protection, cost.

1. INTRODUCTION

Structures designed by engineer needs to be economical, and designers have to pay attention to the safety and manufacturability. Optimization helps satisfy the design criteria with the smallest expenses.

The viewpoint of fire protection, steel structures are a major challenge, because the yield stress of the warming steel decreases, affected by heat. Above 500 °C this decrease can cause the damage of the structure. This is why is so important to deal with fire protection of structures.

2. OPTIMIZATION OF A VESSEL SUPPORTING FRAME WITHOUT FIRE

2.1. Introducing the frame

We can see the build-up and the forces on the schematic figure of the frame (Fig. 1.) [1]. In our example height of the column is marked with *H*, and the width of the beam is marked with *L*, and their value are equally 4 meters.

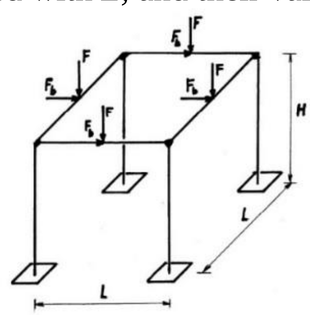

Figure 1. The build-up of the frame [1]

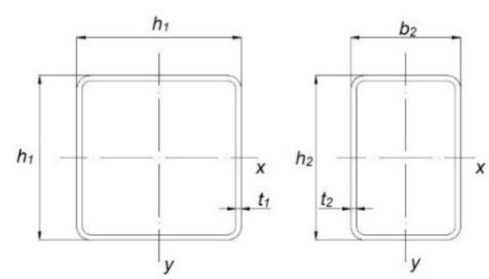

Figure 2. The cross sections of SHS and RHS hollow sections

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Each vertical force effecting on beams is 75 [kN], so the total load is 300 [kN]. The values of horizontal forces are the tenth of the vertical's (F_b=0,1F), namely 7500 [N]. In the base case beams are made of rectangular, and columns are made of square hollow sections. The cross-section area of the SHS column depends on the side length (h_1) and on the wall thickness (t_1). In the case of RHS beams the cross-section area depends on the height (h_2), on the width (b_2) and also on the wall thickness (t_2) (Fig. 2). To facilitate the optimization, we introduced a new variable, called side ratio (a), which equals h_2/b_2. Subsequently, when the fire protection will be achieved with mass increase, there is no opportunity to use standard cross sections (because large sizes are not available from manufacturers), so in this case we will use profiles welded from thicker plates.

2.2. Minimizing the mass of the frame

The optimization task is made with Solver, which is an extension of Microsoft Excel. The principle of operation is, that the software analyzes the possible solutions and chooses the most favourable, while the limiting criteria are fulfilled.

The Microsoft Excel Solver uses the Generalized Reduced Gradient (GRG2) algorithm for optimization in case of non-linear problems. The algorithm is developed by Leon Lasdon (University of Texas at Austin) and Allen Warren (Cleveland State University). The basic concept of the method is, that it search the solution with the expansion of Taylor-series besides non-linear criteria. The reduced gradient method separates two specified subset of the variable, a fundamental and a non-fundamental part. The effective method search the unconditional NLP problems with approximation. The process is repeated, until the optimization criterion is fulfilled. [9]

We worked all in all with five variables, which are the measurements of the cross sections, namely h_1, h_2, t_1, t_2, and a.
The function, which is needed to be minimized is the mass of the frame M:

$$M=\rho\,(4HA_1+4LA_2) \quad (1)$$

We can find the cross sectional areas (A_1, A_2) in this formula (1), which depend on the defined variables.

The stress of beams is two-way bending, in opposite to columns, where we need to design for compressive stress. We calculated the limiting criteria based on Eurocode 3. These are the local buckling of the flange (2) and the web plates (3) of the column, as well as the flange (4) and the web plates (5) of the beam.

$$\frac{b_1}{t_1} \leq 42\varepsilon \quad (2) \qquad \frac{h_1}{t_1} \leq 42\varepsilon \quad (3)$$

$$\frac{b_2}{t_2} \leq 42\varepsilon \qquad (4) \qquad\qquad \frac{h_2}{t_2} \leq 69\varepsilon \qquad (5)$$

Another limiting criteria are the global stability constraint of the column (6) and the beam (7).

$$\frac{H_A + H_{D1}}{\chi_{2.min} A_2 f_{y1}} + \frac{k_{yy2} M_E}{W_{y2} f_{y1}} + \frac{k_{yz2} M_{Bz}}{W_{z2} f_{y1}} \leq 1 \qquad (6)$$

$$\frac{N_1}{\chi_{1.min} A_1 f_{y1}} + \frac{k_{yy1}\,(M_c + M_{B1})}{W_{y1} f_{y1}} + \frac{k_{zz1} M_C}{W_{z1} f_{y1}} \leq 1 \qquad (7)$$

Where:

- $H_A, H_{D1}, M_E, M_{Bz}, N_1, M_c, M_{B1}, M_C$ compressive forces and bending moments
- $k_{yy1}, k_{yy2}, k_{yz2}, k_{zz1}$ – stability parameters
- $\chi_{1.min}, \chi_{2.min}$ – bending-denting factors
- A_1, A_2 – cross sections of the column and beam
- f_{y1}- reduced yield point ($f_{y1} = \frac{f_y}{\gamma_{M,1}}$)
- $W_{y1}, W_{z1}, W_{y2}, W_{z2}$ – cross section factors

3. OPTIMISATION OF THE FRAME FOR FIRE

As the temperature rises, the strength and stiffness of the steel are constantly decreasing. Thus the relation between the temperature and material properties of the steel have to be defined, which depends on the time in standard fire. Definition of fire resistance of structural elements: the time, when the structure can’t perform his function. The yield stress and the Young modulus on higher temperature is calculated based on Eurocode 3 1.2. [4]

Considering the fire protection the formulas of the global stability constraint are changing, the new forms for the beam (8) and for the column (9) are:

$$\frac{H_A + H_{D1}}{\chi_{2.min\,fi}\, k_{y,\theta} A_2 f_{y1}} + \frac{k_y\, M_E}{W_{y2} k_{y,\theta} f_{y1}} + \frac{k_z M_{Bz}}{W_{z2} k_{y,\theta} f_{y1}} \leq 1 \qquad (8)$$

$$\frac{N_1}{\chi_{1.min\,fi}\, k_{y,\theta} A_1 f_{y1}} + \frac{k_y\,(M_c + M_{B1})}{W_{y1} k_{y,\theta} f_{y1}} + \frac{k_z M_C}{W_{z1} k_{y,\theta} f_{y1}} \leq 1 \qquad (9)$$

Where:

- k_y, k_z – interaction factors;
- $k_{y,\theta}$ - reduction factor of yield stress at θ temperature

These parameters are calculated based on [1, 3, 4, 13].

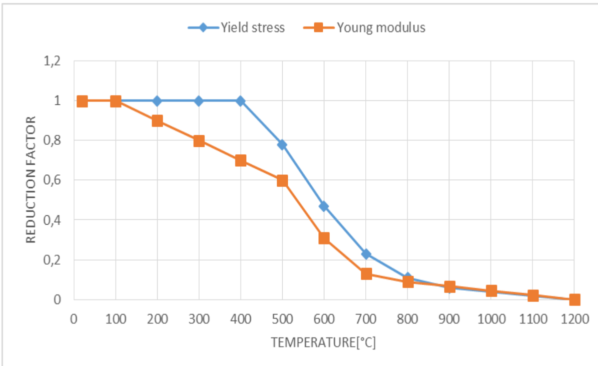

Figure 3. Reduction factor of yield stress and Young modulus versus temperature

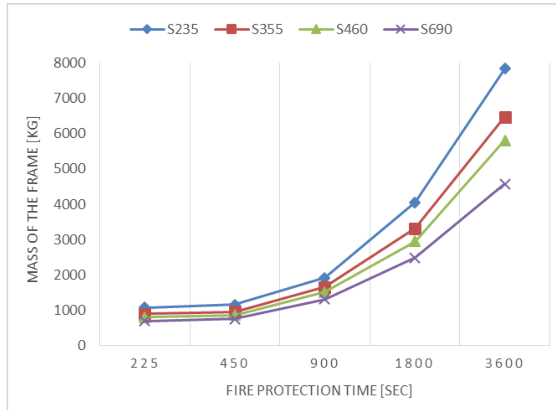

Figure 4. Mass of the frame in case of different types of steel versus fire protection time

The ε parameter (10) in the formula of local buckling criterion is also changed:

$$\varepsilon = 0{,}85\sqrt{\frac{235}{f_y}} \qquad (10)$$

We made calculations for four different types of steel (f_y=235 MPa; 355 MPa, 460 MPa, 690 MPa). The criteria mentioned above were calculated as the temperature increased (values reported at 225, 450, 900, 1800, and 3600 sec.).

In this case the function which is needed to be minimized was also the mass of the structure, therefore our variables were the same, too. At the Figure 4 we can see the mass of the frame of the four different types of steel versus fire protection time. It is clear from the graph, that there is a significant difference between using S235 and S690 steel. Using the S690 steel, 32-42 percent mass decrease can be achieved.

The values calculated by Solver are summarized in the following tables, where the rounded values (for the standard hollow sections) are also shown based on [2, 5, 6].

Since hollow section sizes are available in standard sizes manufactured by companies, therefore, usually there is a remarkable difference between the constant and discrete values. The mass of the frame increased by 6-12% in case of S235, 4-12% in the case of S355, with 4-11,5% in case of S460 and with 4-14% in case of S690.

Table 1. Results for the S235 steel

S235		Column (mm)		Beam (mm)			
Fire prot. Time (s)		h1	t1	h2	b2	t2	Mass (kg)
225	continous	220,4	6,17	252,82	153,89	4,31	1069,7
	standard	220	6,3	250	150	5	1136,52
450	continous	221,05	6,19	258,24	172,16	4,82	1149,81
	standard	220	6,3	260	180	6	1286,23
900	continous	135,93	14,37	388,75	259,16	7,26	1917,17
	standard	150	16	400	200	8	2107,99
1800	continous	390,99	12,19	402,9	268,6	11,4	4049,06
	standard	350	16	450	250	12	4539,61
3600	continous	571,2	16	560,19	495,91	13,89	7843,6

Table 2. Results for the S690 steel

S690		Column (mm)		Beam (mm)			
Fire prot. Time (s)		h1	t1	h2	b2	t2	Mass (kg)
225	continous	143,14	6,87	149,48	90,98	4,37	694,62
	standard	140	8	150	100	5	791,76
450	continous	145,69	6,99	151,88	101,25	4,86	751,46
	standard	140	8	150	4	5	791,76
900	continous	142,68	12,13	230,15	140,09	6,72	1312,15
	standard	150	12	220	120	8	1388,71
1800	continous	256,18	12,3	242,9	161,93	11,58	2484,26
	standard	260	12,5	250	150	12,5	2585,79
3600	continous	355,65	17,07	404,84	246,43	11,83	4572,21

The table shows also, after 3600 s (where the temperature is above 900°C), standard dimensions are not available. So in this case we have to use cabinet profiles welded from plate.

In case of welded elements 6 variables were defined, because different wall thickness can be used at the flange and at the web plates. We can see in Figure 5 the result of optimization for welded plate elements versus fire protection time. The difference between the constant and the standard values is 36-42%. The standard values of hollow sections and welded plate were compared. We can see significant differences at 1800 sec. For instance, in case of S235 steel, the mass difference reached 550 kg (Fig.6).

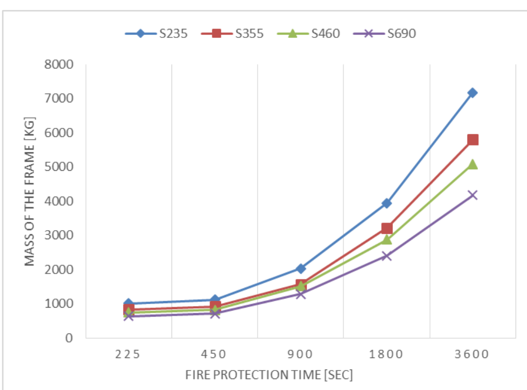

Figure 5. Mass of the frame in case of different types of welded plate versus fire protection time

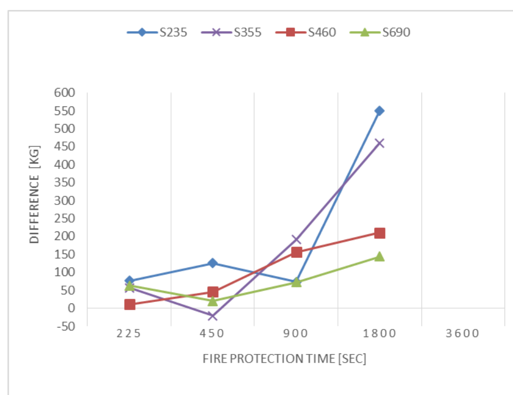

Figure 6. Mass difference between hollow section and welded plate

In case of S355 there is a negative value at 450 s, because the mass of the structure was smaller, when we calculated with hollow sections.

4. THE COST CALCULATIONS

4.1. The hollow structure without fire exposure

The costs of the design were calculated with the methods included in [1, 2, 10] and the following formulae on an optimized model, S235 steel hollow section without any fire protection. The dimensions of the column $h_1=b_1=180$ mm, $t_1=5$ mm and the dimensions of the beam are $h_2=180$ mm, $b_2=100$mm and $t_2=4$ mm.

The formula of the structure's total costs without any fire protection is (11):

$$K = K_M + K_f + K_r + K_p = k_M \rho V + k_f \sum_i T_i + k_r A_r + k_p A_p = 355\,550 \text{ [Ft]} \qquad (11)$$

where:

$-\ K_M$ =188771 [Ft] - Material cost

- $k_M = 270$ [Ft/kg] – Cost factor
- $\rho = 7.85\text{x}10^{-6}$ [kg/mm^3] - Material density
- V=89 006 400 [mm^3] - Structure's volume

At the volume calculation, beside the 4 columns and the beam, the authors regarded also the four plates (b=l=180 mm, t=5 mm), which were welded on the columns' top and serve as a support for the beams.

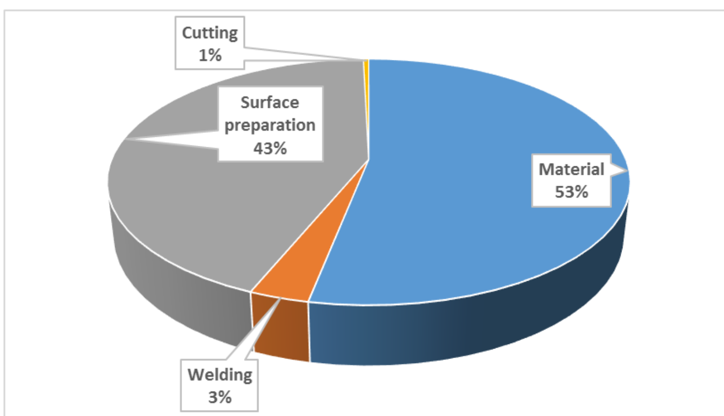

Figure 7. Distribution of cost types in case of no fire protection

Figure 7 shows the ratio of different types of costs. It is clear from the graph, that the material cost and the surface preparation (corrosion protection painting and blasting) cost is the significant. Welding and cutting cost is negligible in case of hollow sections. On the other hand these costs would be much more remarkable, if we would use profiles welded from thicker plates instead of hollow sections.

- ***K_f* - Fabrication cost**

Because the fabrication cost factor (k_f) varies from a manufacturer to another the production cost's formula will be the following. (12):

$$K_f = k_{f1}(T_{w1} + T_{w2} + T_{w3}) + k_{f2}T_{cp} + k_{f3}(T_{sp} + T_p) = 83\ 660 \text{ [Ft]} \qquad (12)$$

- k_{f1} =37 [Ft/min] - Welding cost
- $k_{f2} = 135$ [Ft/min] - Cutting cost
- $k_{f3} = 170$ [Ft/min]; - Painting Cost
- $T_{w1}, T_{w2,} T_{w3}, T_{cp}, T_{sp}, T_p$ [min] - Preparation, welding, cutting and painting times

a) *Calculation of the time of preparation*, assembling (13):

$$T_{w1} = C_1\theta_{dw}\sqrt{\kappa\rho V}=183{,}19 \text{ [min]} \qquad (13)$$

- $C_1 = 1$ - a parameter which depends on the welding technology;
  $\theta_{dw} = 2$ - fabrication difficulty factor, which depends on $\kappa = 12$, number of assembled parts;

b) *Calculation of a real welding time and of an additional fabrication action times* (14):

$$T_{w2} + T_{w3} = 1.3\sum C_{wi}a_{wi}^n L_{wi}C_{pi} = 117{,}49 \text{ [min]} \qquad (14)$$

where:

- The welding of the plate and the column (SMAW welding technology), with a butt welding:

  - $C_{w1} = 3{,}13 \times 10^{-3}$, welding time parameter
  - $n_1 = 1$; $a_{w1} = t_1 = 5$ mm, weld dimension
  - $L_{w1} = 4 \times 4 \times h_1 = 2880$ [mm]
  - $C_{p1} = 1$, welding position factor (horizontal 1, vertical 2, downhand 3)

- Welding of the column's flange (SMAW), V welding:

  - $C_{w2} = 2{,}7 \text{ x } 10^{-3}$
  - $n_2 = 1$
  - $a_{w1} = t_2 = 4$ [mm]
  - $L_{w2} = 4\sqrt{2b_2^2} = 565{,}69$ [mm]
  - $C_{p2} = 1$

- Welding of the column's webs (SMAW), with corner welding:

    - $C_{w3} = 0{,}7889 \times 10^{-3}$
    - $n_3 = 2$
    - $a_{w3} = 0{,}75t_2 = 3$ [mm]
    - $L_{w3} = 4 \times 2 \times h_2 = 1440$ [mm]
    - $C_{p3} = 2$

- The welding of the plate and the column (SMAW), with corner welding:

    - $C_{w4} = 0{,}7889 \times 10^{-3}$
    - $n_4 = 2$
    - $a_{w4} = 0{,}75t_2 = 3$ [mm]
    - $L_{w4} = 4\left[2\left(b_1 - \frac{b_1 - b_2}{2}\right) + \left(2\frac{b_1 - b_2}{2}\right)\right] = 1440$ [mm]
    - $C_{p4} = 1$

- The welding of the plate/column and beam (SMAW), with corner welding:

    - $C_{w5} = 0{,}7889 \times 10^{-3}$
    - $n_4 = 2$
    - $a_{w4} = 0{,}75\mathrm{t}_2 = 3$ [mm]
    - $L_{w4} = 4b_2 = 400$ [mm]
    - $C_{p4} = 3$ - downhand

c) *Plate Cutting and Edge Grinding Times* (15):

$$T_{cp} = \sum_i C_{CPi} t_i^n L_{ci} = 10{,}45 \text{ [min]} \quad (15)$$

- Cutting time of the column:

    - $C_{CP1} = 1.1388 \times 10^{-3}$
    - $t_1 = 5$ $[mm]$
    - $n_1 = 0{,}25$
    - $L_{c1} = L_{w1} = 4 \times 4 \times h_1 = 2\,880$ [mm]

- Cutting time of the beam:

    - $C_{CP2} = 1.1388 \times 10^{-3}$
    - $t_2 = 4$ $[mm]$
    - $n_2 = 0{,}25$
    - $L_{c2} = L_{w2} + L_{w3} + L_{w4} = 3\,445{,}7$ [mm]

d) Surface preparation time (16):

$$T_{SP} = \theta_{ds} a_{sp} A_s = 124 \text{ [min]} \tag{16}$$

Where:

- $\theta_{ds} = 2$ difficulty parameter
- $a_{sp} = 3 \times 10^{-6}$ [min/mm^2]
- $A_s = 4H(h_1 + b_1)2 + 4L(h_2 + b_2)2 + 4hb = 20\ 609\ 600$ [mm^2]

e) Anti-corrosion painting time (17) – (ground and top coat):

$$T_p = \theta_{dp}\left(a_{gc} + a_{tc}\right)A_s = 294{,}72 \text{ [min]} \tag{17}$$

Where:

- $\theta_{dp} = 2$
- $a_{gc} = 3 \times 10^{-6}$ [min/mm^2]
- $a_{tc} = 4{,}15 \times 10^{-6}$ [min/mm^2]
- $A_s = 2{,}06 \times 10^7$ [mm^2]

- ***K_r* - Surface cleaning with blasting Sa 2.5 (HULP airpistol):**

$$K_r = k_r A_r = 65435 \text{ [Ft]} \tag{18}$$

- $k_r = 3\ 175$ [Ft/m^2] - m^2 cost
- $A_p = 20{,}61$ [m^2] - surface area of the frame

- ***K_p* - Anti-corrosion paint costs:**

$$K_p = k_p A_p = 17683{,}04 \text{ [Ft]} \tag{19}$$

- $k_p = 858$ [Ft/m^2] - m^2 cost
- $A_p = 20{,}61$ [m^2] - surface area of the frame

4.2. Fire protection with intumescent coating

Cost of intumescent Painting- for a 30 minute fire protection:

$$K_{ip} = k_{ip} A_{ip} + k_{f3} T_{ip} = 176\ 542 \text{ [Ft]} \tag{20}$$

- $k_{ip} = 6\ 135$ [Ft/m^2]
- $A_{ip} = 20{,}61$ [m^2];

- $k_{f3} = 170$ [Ft/s];
- $T_{ip} = \theta_{dp}\left(a_{gc} + a_{tc}\right)A_s = 294{,}72$ [min]

Total cost:

$$K_{intumescent\ coating} = K - K_p + K_{ip} = 514\ 409 \text{ [Ft]} \quad (21)$$

4.3. *Fireboard protection*

Cost of fireboard protection (with 25mm): - for a 30 minute fire protection:

$$K_{fp} = (k_{Mfp} + k_{fp})A_{fp} = 304\ 008 \text{ [Ft]} \quad (22)$$

- $k_{Mfp} = 11\ 709$ [Ft/m^2] - m^2 cost of material
- $k_{fp} = 3042$ [Ft/m^2] - m^2 cost of work
- $A_{fp} = 20{,}61$ [m^2] - surface area of the frame

Total cost:

$$K_{fireboard} = K + K_{fp} = 355\ 550 + 304008 = 659\ 556 \text{ [Ft]} \quad (23)$$

With the help of a cost estimator and the following document the authors received the fireboard protection cost [11, 12 ,13].

4.4. Fire resistance (R30) obtained by the volume increase of the steel S 235

The following calculations will be performed on the hollow section with steel S235, present in the Table 1 optimized for 1800 seconds (R30) fire resistance. The column's dimensions are h_1=b_1=350 mm, t_1=16 mm and of the beam h_2=450 mm, b_2=250 mm and t_2=12 mm.
The total costs were calculated in this case similar to the situation in chapter 4.1 (11) equation for fire resistance like in the following part (24):

$$K_{S235} = K_M + K_f + K_r + K_p = 1\ 639\ 683 \text{ [Ft]} \quad (24)$$

Where:

- $K_M = 1\ 230\ 886$ [Ft]
- $K_f = 226\ 142$ [Ft]
- $K_r = 143\ 795$ [Ft]
- $K_p = 38\ 858$ [Ft]

4.5. Fire resistance (R30) obtained by the volume increase of the steel S690:

The following calculations will be performed on the hollow section with S690 present in the Table 2. optimized for 1800 seconds (R30) fire resistance. The dimensions of the column are: $h_1=b_1=260$ mm, $t_1=12{,}5$ mm and of the beam $h_2=250$ mm, $b_2=150$ mm and the $t_2=12.5$ mm.

$$K_{S690} = K_M + K_f + K_r + K_p = 1\ 006\ 404\ [\mathrm{Ft}] \tag{25}$$

Where:

- $K_M = 736\ 080\ [\mathrm{Ft}]$ - the steel's material cost is 5% greater, than in the case of the steel with S235
- $K_f = 150\ 502\ [\mathrm{Ft}]$
- $K_r = 94\ 330\ [\mathrm{Ft}]$
- $K_P = 25\ 491 [\mathrm{Ft}]$

The results of the cost calculations are summed up in Table 3 and are shown in Fig. 7.

Table 3. Results of cost calculation

<table>
<tr><th rowspan="3">Structure</th><th colspan="3">Column</th><th colspan="3">Beam</th><th rowspan="2">Total Price</th></tr>
<tr><th>h_1</th><th>b_1</th><th>t_1</th><th>h_2</th><th>b_2</th><th>t_2</th></tr>
<tr><th colspan="6">[mm]</th><th>[Ft]</th></tr>
<tr><td>Without fire protection S235, no fire</td><td rowspan="3">180</td><td rowspan="3">180</td><td rowspan="3">5</td><td rowspan="3">180</td><td rowspan="3">100</td><td rowspan="3">4</td><td>355 550</td></tr>
<tr><td>With intumescent fire protection R30 min. fire</td><td>514 409</td></tr>
<tr><td>With Fireboard protection R30 min. fire</td><td>659 556</td></tr>
<tr><td>Without fire protection S235, R30 min. fire</td><td>350</td><td>350</td><td>16</td><td>450</td><td>250</td><td>12</td><td>1 639 683</td></tr>
<tr><td>Without fire protection S690, R30 min. fire</td><td>260</td><td>260</td><td>12.5</td><td>250</td><td>150</td><td>12.5</td><td>1 006 404</td></tr>
</table>

Figure 8 shows the prices and their distribution of different types of fire protection methods namely mass increase, yield stress and mass increase, intumescent coating and fireboard protection.

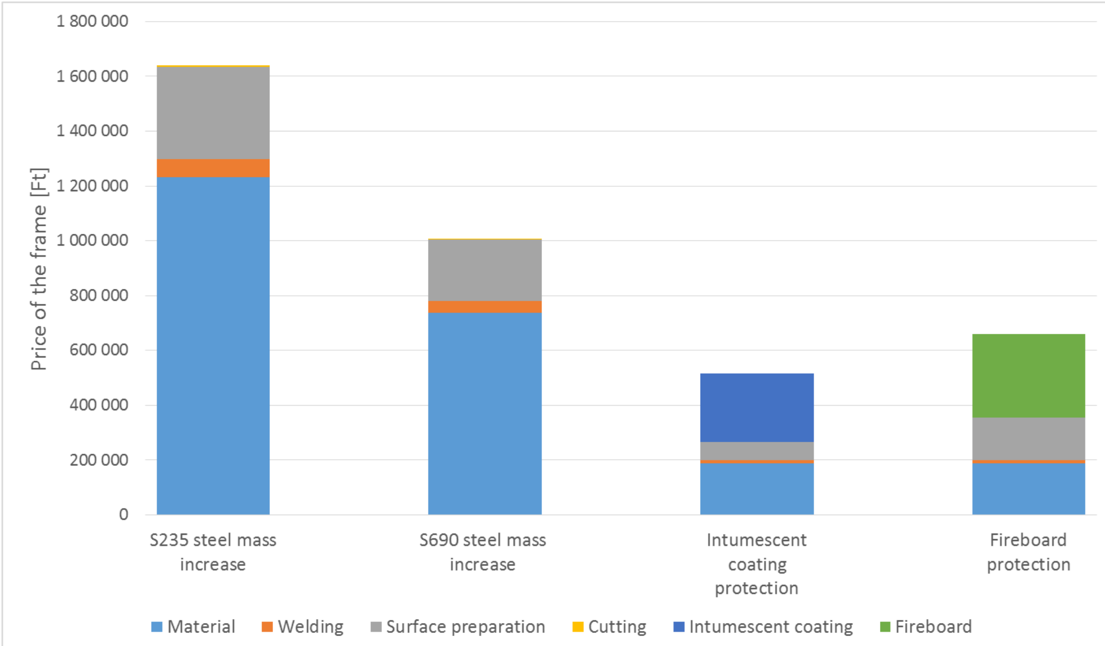

Figure 8. Results of cost calculation (Equal fire protection levels)

5. CONCLUSIONS

From the results presented in Table 3 one can conclude that if the fireproofing should be guaranteed with the structure's volume increase and not by coating of the elements the costs will be much higher. From the example must be highlighted: in the case of the structure's volume increase method the S235 steel costs increased by 3 times, to be more exact by 319% compared to the paint-coating method. Throughout the optimization process the costs of the intumescent paint proved themselves to be the most efficient protection.

It must be also mentioned that the authors observed a great difference between the hollow section structure of the S235 and of the S690 steel. When the hollow section structure of the S690 steel was used for fireproofing purposes the cost decreased by 39%.

6. ACKNOWLEDGEMENT

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