

Economic analysis of Stainless Steel- Reinforced Concrete members designed for a Fire Resistance of 2 hours

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I. INTRODUCTION

While Stainless steel is used primarily for its corrosion resistance, there is a growing interest in other properties, such as mechanical, high temperature resistance, aesthetics and others.

The properties at high temperatures (400 – 650°C) of stainless steels are known and put to use in a number of applications such as steam turbines and equipment for the chemical industry. Some research on the fire resistance of stainless steels has been carried out for welded I beams and for concrete-filled tubes, with positive results. However, there are no published results on the behavior of concrete reinforced with stainless steel rebar.

In a preliminary research program conducted by the University of Liege for ISSF, a preliminary assessment of the performance of stainless steel reinforced concrete has been carried out, using computer simulation methods. This preliminary study has shown that stainless steel reinforced concrete members can behave better than carbon steel reinforced concrete members in fire situation. However, an economic analysis is still needed to determine whether stainless steel reinforcement can be interesting for practical applications.

This report is divided in two parts. The first part presents the assumptions and the results of the numerical simulations performed by University of Liege. The numerical simulations aim to design stainless and carbon steel -reinforced concrete members satisfying to a fire resistance of 2 hours (Standard fire conditions). The two structural members considered in this study are a reinforced concrete beam and a composite slab (with the so-called « membrane » behavior). The numerical simulations are performed with the SAFIR 20011.a.3 software developed in University of Liege.

The second part presents the assumptions and the results of the economic analysis of the structural members designed in the first part. The stainless and carbon steel –reinforced solutions for a fire resistance of 2 hours are compared on an economic basis.

II. TECHNICAL ANALYSIS

II.1. General data

II.1.1. Presentation of the studied cases

Two structural members are considered in this study:

- A reinforced concrete beam
- A composite slab

The objective of the study is to design R 120 solutions (fire resistance higher than 120 minutes in Standard fire conditions) for these two types of structural members, with stainless and with carbon steel reinforcement. In the second part, an economic analysis of the designed solutions is performed to compare the stainless steel solution with the carbon steel solution.

The fire that is considered for the analysis is the Standard ISO-834 fire.

II.1.2. Stainless Steel properties

The present work has been performed considering the SLS1.4311 grade.

II.1.2.1. Thermal properties

According to Eurocode 3 (EN 1993-1-2, clause C.1 (1)), the thermal properties are the same for all stainless steel types that comply with EN 1993-1-4. The thermal properties are taken from Annexe C of the Eurocode.

The thermal conductivity is given by the following formula:

$$\lambda = 14.6 + 1.27 \times 10^{-2} T$$

where T is the temperature in °Celsius. The thermal conductivity of stainless steel is significantly lower than that of conventional carbon steel.

The specific heat is given by the following formula:

$$c = 450 + 0.28T - 2.91 \times 10^{-4} T^2 + 1.34 \times 10^{-7} T^3$$

where T is the temperature in °Celsius.

The specific mass has been taken equal to: $\rho = 7850 \text{ kg/m}^3$.

Other properties, such as emissivity or coefficient of convection, are not relevant in this study as no stainless steel part is directly exposed to the effect of the fire.

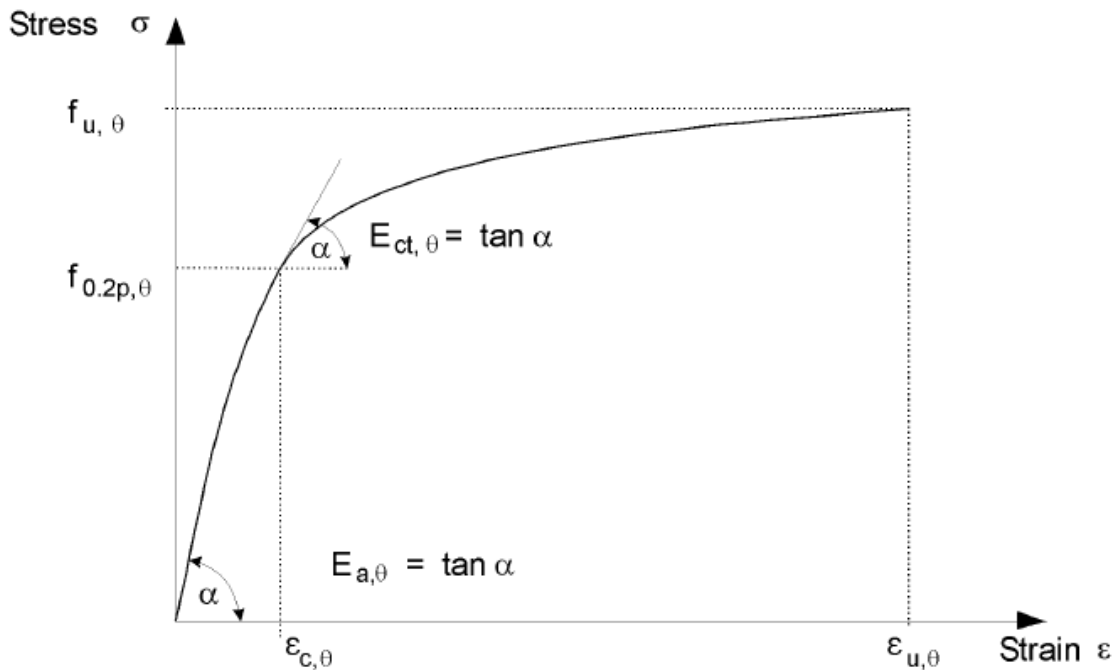
II.1.2.2. Mechanical properties

Thermal elongation of austenitic stainless steel is given by Eq. C1 of Eurocode EN 1993-1-2:

$$\varepsilon_{th} = (16 + 4.79 \times 10^{-3} T - 1.243 \times 10^{-6} T^2) \times (T - 20) 10^{-6}$$

The thermal expansion of stainless steel is significantly higher than that of conventional carbon steel.

For heating rates between 2 and 50 K/min, the stress strain relationship is given by Figure C.1 of EN 1993-1-2.



- Key:**
- $f_{u,\theta}$ is tensile strength;
 - $f_{0.2p,\theta}$ is the proof strength at 0.2% plastic strain;
 - $E_{a,\theta}$ is the slope of the linear elastic range;
 - $E_{ct,\theta}$ is the slope at proof strength;
 - $\epsilon_{c,\theta}$ is the total strain at proof strength;
 - $\epsilon_{u,\theta}$ is the ultimate strain.

Figure 1: Stress-strain relationship for stainless steel

The evolution of the parameters that define this stress strain relationship is not given in the Eurocode for the grade 1.4311. The values of the parameters given in Table I have been considered in this analysis.

Temp	$f_{0.2}$	$k_{f,0.2}$	UTS	k_{UTS}	ϵ_{su}	$k_{E,01}$	$k_{E,02}$
°C	MPa	-	MPa	-	-	-	-
20	530	1,000	750	1,000	0,3	1.000	0.110
100	481	0.908	683	0.911	0.3	0.960	0.050
200	420	0,792	600	0,800	0,3	0.920	0.020
400	380	0,717	550	0,733	0,3	0.840	0.020
600	330	0,623	470	0,627	0,3	0.760	0.020
700	265	0.482	365	0.487	0.3	0.710	0.020
800	180	0,340	260	0,347	0,3	0.630	0.020
900	126	0,140	113	0,150	0,3	0.450	0.020
1000	32	0.060	53	0.070	0.3	0.200	0.020
1100	16	0.030	23	0.030	0.3	0.100	0.020
1200	0	0.000	0	0.000	0.3	0.000	0.020

Table 1: Parameters for the stainless steel stress-strain relationship

In Table 1,

- The values of the cells marked as xxx have been taken from the ISSF PowerPoint presentation
- The values of the cells marked as xxx have been obtained by linear interpolation
- Other values have been taken equal to the corresponding values given in Eurocode EN 1993-1-2 for stainless steel grade 1.4301

The strength of 1.4311 stainless steel grade is higher than the strength of 1.4301 grade, at least for temperatures up to 800°C. It has to be emphasized that the strain of stainless steel has to reach very high levels (0.30 considered here for 1.4311) for the stainless steel to reach the ultimate tensile strength (UTS), whereas a strain of 0.02 is sufficient for the carbon steel to reach its effective yield strength.

II.1.3. Carbon Steel properties

II.1.3.1. Thermal properties

The thermal properties that are considered for the carbon steel reinforcement are the properties given in the Eurocode EN 1992-1-2 in which:

- convection coefficient on hot surfaces is 25.0 W/m²K
- convection coefficient on cold surfaces : 4.0 W/m²K
- relative emissivity is 0.7

In the slab analysis, steel profiles are also modeled and the properties for the steel profiles are taken from the Eurocode EN 1993-1-2. The values of the parameters used the steel profiles thermal analysis are the same as for the reinforcement.

II.1.3.2. Mechanical properties

The mechanical properties for the carbon steel reinforcement are taken from the EN 1992-1-2, in which:

- yield strength is 500 MPa
- Young modulus is 210.000 MPa

For the steel profiles considered in the slab analysis, the mechanical properties are taken from the EN 1993-1-2 and the values of the main parameters are:

- yield strength is 355 MPa
- Young modulus is 210.000 MPa

II.1.4. Concrete properties

II.1.4.1. Thermal properties

The thermal properties of concrete are taken from Eurocode EN 1992-1-2. A siliceous concrete is considered with a specific mass of 2400 kg/m³, a moisture content of 72 kg/m³, a convection coefficient on hot surfaces of 25 W/m²K, a convection coefficient on cold surfaces of 4 W/m²K, a relative emissivity of 0.8 and a parameter for thermal conductivity of 0.5 (average between upper limit and lower limit).

II.1.4.2. Mechanical properties

The mechanical properties of concrete are taken from Eurocode EN 1992-1-2. A compressive strength of 30 MPa is considered for the concrete used for the reinforced beam whereas a compressive strength of 50 MPa is considered for the concrete of the composite slab.

II.2. Beam analysis

II.2.1. Data

A simply supported beam with a 6 m span is considered in this analysis. The reference section is 160 mm wide and 400 mm high. The beam is reinforced with 3 Φ 20 mm rebars in its lower part. In the upper part, the section is reinforced with 3 Φ 10 mm rebars (i.e. 236 mm²). This is superior to the minimum amount of rebars to limit cracking, that is given by:

$$0.15\% \times b \times h = 0.0015 \times 160 \text{ mm} \times 400 \text{ mm} = 96 \text{ mm}^2$$

The cover is defined in this report as the distance between the edge of the section and the edge of the bar (not to the axe of the bar). The maximum cover that is possible for the considered section, in the lower part, considering the minimum distance of 20 mm between the Φ 20 mm rebars, is given by the following formula:

$$(160 \text{ mm} - (3 + 2) \times 20 \text{ mm}) \times \frac{1}{2} = 30 \text{ mm}$$

The cover is thus taken equal to 30 mm.

The shear reinforcement is made of Φ 6 mm carbon steel stirrups with 200 mm spacing, i.e. 283 mm²/m stirrups. This is superior to the minimum amount of 0.11 % for the shear reinforcement.

II.2.2. Load bearing capacity at room temperature

It is assumed that the beam is submitted to the following loads:

Description	Characteristics kN/m	Room temperature factor	Fire factor
Permanent loads	8.00	1.35	1.00
Live loads	6.15	1.50	0.30

Table 2: Loads for the beam analysis

Consequently, the applied load considered for the ELU analysis at room temperature is equal to $p_{ELU} = 8.00 \times 1.35 + 6.15 \times 1.50 = 20.02 \text{ kN} / \text{m}$. It has also to be verified that the beam satisfies to the serviceability criterion, i.e. that the maximum deflection under the ELS combination of loads is lower than the maximum acceptable deflection. The load considered for the ELS analysis is given by: $p_{ELS} = 8.00 \times 1.00 + 6.15 \times 1.00 = 14.15 \text{ kN} / \text{m}$. The maximum acceptable deflection is taken as L/250, i.e. 2.4 cm.

The load bearing capacity at room temperature is established for the carbon steel reinforced beam and the section 160 mm X 400 mm. It can be calculated analytically as follows:

- Maximum resistant moment:

$$M_{Rd} = 0.9 \times d \times A_s \times f_{yc} = 0.9 \times 360 \text{ mm} \times 942.48 \text{ mm}^2 \times \left(\frac{500}{1.15} \text{ N} / \text{mm}^2 \right) = 132.8 \text{ kNm}$$

- Maximum distributed load:

$$M_{sd,max} = p_{max} \times l^2 / 8 \rightarrow p_{max} = M_{Rd} \times 8 / l^2$$

$$\rightarrow p_{max} = 29.50 \text{ kN} / \text{m}$$

As a verification, the SAFIR numerical analysis at room temperature gives a maximum distributed load: $p_{max} = 32.35 \text{ kN} / \text{m}$. The ELU criterion is thus satisfied since the maximum load bearing capacity $p_{max} = 32.35 \text{ kN} / \text{m}$ of the reference beam is higher than the ELU applied load $p_{ELU} = 20.02 \text{ kN} / \text{m}$.

Finally, it is verified that the central deflection of the beam under the ELS combination of load is lower than 2.4 cm. The SAFIR analysis at room temperature gives a central vertical deflection of 2.0 cm for a distributed load of $p_{ELS} = 14.15 \text{ kN} / \text{m}$.

II.2.3. Data for the fire analysis

The applied load for the fire analysis is equal to $p_{fire} = 8.00 \times 1.00 + 6.15 \times 0.30 = 9.85 \text{ kN} / \text{m}$. Finally, a distributed load of $p_{fire} = 10 \text{ kN} / \text{m}$ is considered for the fire analysis, which corresponds to 50% of the load considered for the ultimate load analysis at room temperature.

The objective is to design the beam in order to reach a fire resistance of 120 minutes; the section and/or the reinforcement of the beam can thus be adapted. A stainless and a carbon steel –reinforced section are considered and compared. As explained in the following parts, for the carbon steel reinforced section, the R 120 condition will lead us to consider a section of 180 mm width by 410 mm height to allow for an increased cover equal to 40 mm.

The fire resistance is taken as the time at which the structure collapse is reached. In this study, the structure collapse is considered to be reached when the structure maximum deflection reaches $L/20 = 0.30 \text{ m}$.

II.2.4. Thermal analysis

The thermal analysis is performed with the SAFIR software. The beam is subjected to ISO fire on three faces. The thermal properties for the stainless steel, carbon steel and concrete materials have been given previously.

The following figures give the temperature distribution in the section of 160 mm width by 400 mm height after 60 minutes and after 120 minutes.

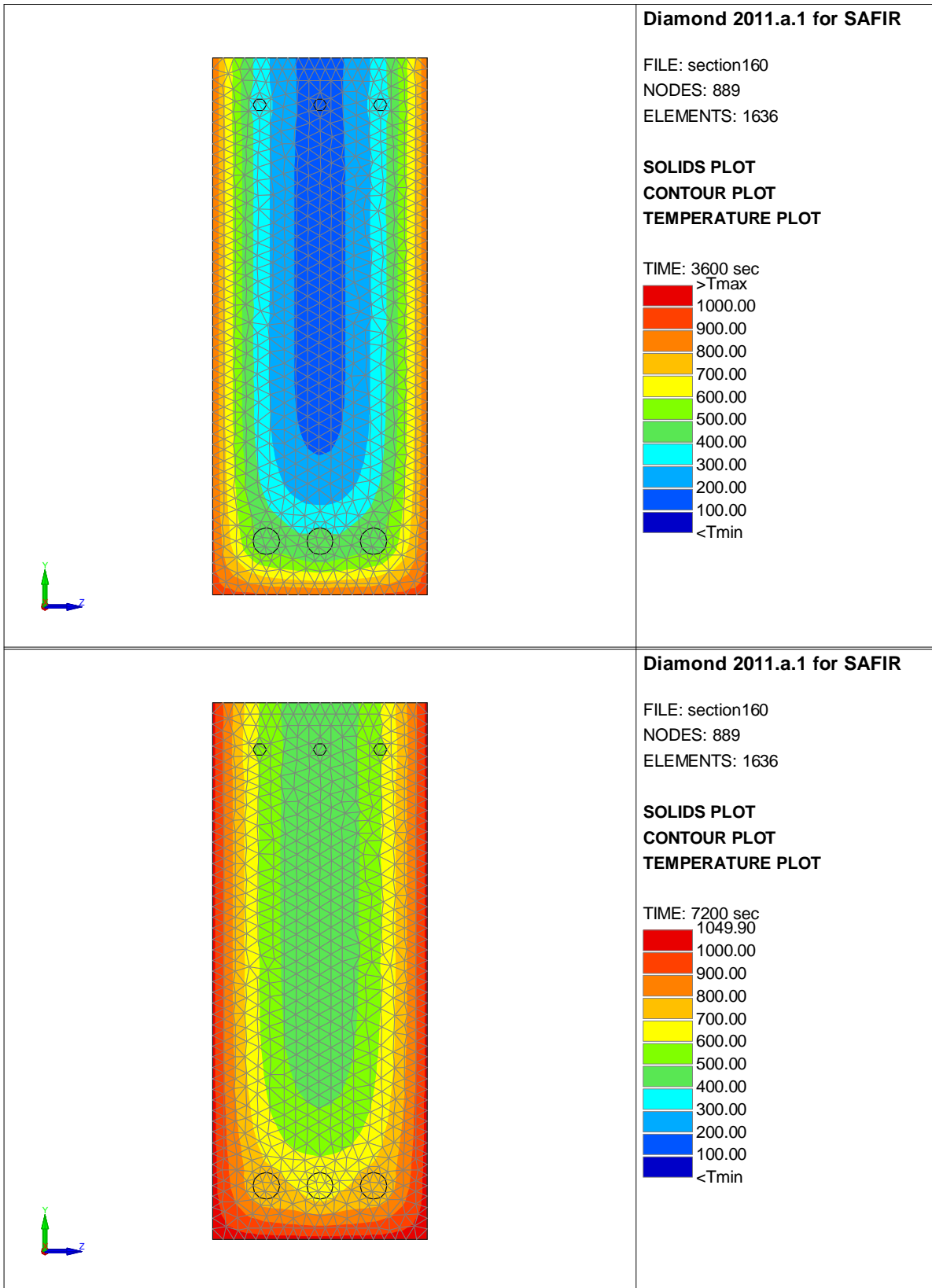


Figure 2: Temperature distribution in the beam section after 60 minutes and after 120 minutes

The evolution of the temperature in the lower corner rebars for two different section geometries can be seen in Figure 3. In the 180 X 410 mm² section, the temperature in the lower corner rebars increases slower than in the 160 X 400 mm² section because the concrete cover is increased from 30 mm to 40 mm.

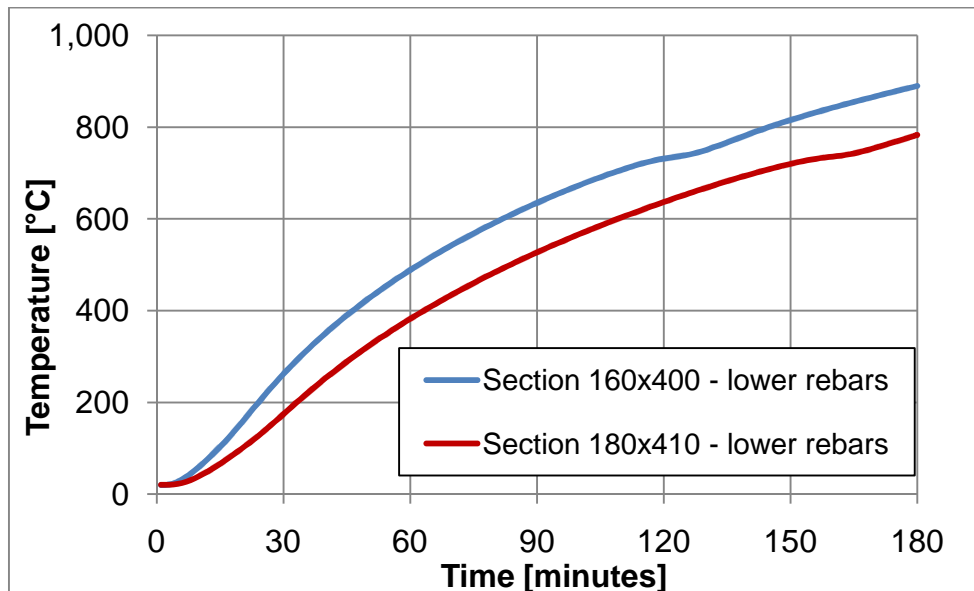


Figure 3: Temperature evolution in the lower corner rebars

II.2.5. Structural analysis

II.2.5.1. Case I : carbon steel reinforced concrete

A finite element model was built in the SAFIR software. The structure is modeled using BEAM elements.

The main data for the reference case, with carbon steel reinforced concrete, are summarized in Table 3:

Section		Case 1	Case 2
	Width	160 mm	180 mm
	Height	400 mm	410 mm
Reinforcement			
	Upper part	3 Φ 10 carbon	3 Φ 10 carbon
	Lower part	3 Φ 20 carbon	3 Φ 20 carbon
Cover			
	Lower part	30 mm	40 mm
Load (fire analysis)			
	Distributed load	10.0 kN/m	10.0 kN/m
Span			
	Isostatic beam	6.0 m	6.0 m

Table 3: Data for the carbon steel reinforced beam analysis

The fire resistance that is obtained using the reference section (Case 1) of 160 mm by 400 mm is 95 minutes (R 90).

In order to increase the fire resistance of the beam, a solution is to increase the concrete cover of the lower rebars. However, there is no sufficient space to move the rebars towards the inside of the section as the minimum spacing between the rebars is 20 mm. To increase the concrete cover, it is thus necessary to increase the dimensions of the section. An additional 10 mm of concrete is considered on the three sides attacked by the fire, which gives a section of 180 mm width and 410 mm

height with a concrete cover of 40 mm for the lower rebars. The other parameters are the same as in the first studied case. With this increased section of 180 mm X 410 mm, the fire resistance is increased up to 126 minutes, and the objective of R 120 is reached.

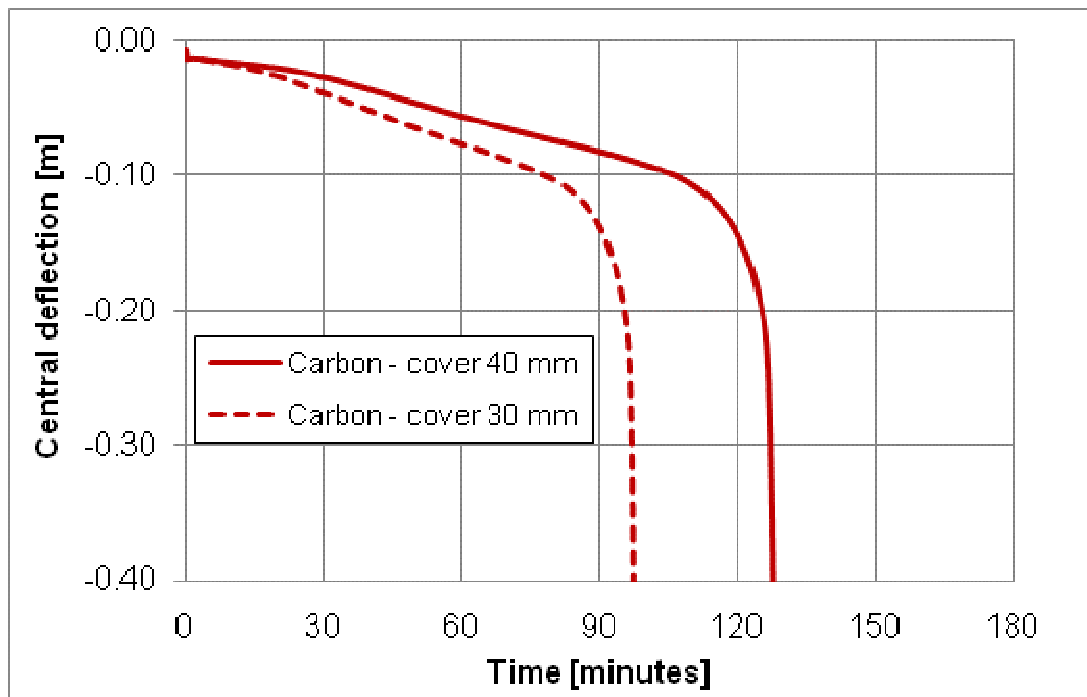


Figure 4: Deflection VS Time for the carbon steel reinforced beams

II.2.5.2. Case II : stainless steel reinforced concrete

The analysis of the reference case (Case 1) where the 3 Φ 20 carbon steel lower rebars have been replaced by stainless steel rebars gives a fire resistance of 162 minutes. The objective of 120 minutes is thus overreached (see Figure 5).

In fact, if stainless steel reinforcement is used for the three lower rebars, not only is the cover of 30 mm sufficient to reach R 120, but it would also be possible to use 3 Φ 16 stainless steel rebars instead of the 3 Φ 20 carbon steel rebars. The beam with a 30 mm cover and 3 Φ 16 stainless steel rebars has a fire resistance of 126 minutes. Moreover, it is interesting to notice that the fire resistance of 126 minutes is the value obtained because of the restriction on the maximum deflection $L/20$, but in fact the collapse of the beam with stainless steel reinforcement is much more ductile than the collapse of the beam that has carbon steel reinforcement as can be observed in Figure 5. Consequently, even though the 180 mm width beam with 3 Φ 20 carbon steel rebars and the 160 mm width beam with 3 Φ 16 stainless steel rebars have almost the same fire resistance, the stainless steel reinforced beam has a ductile mode of failure which has always to be preferred in practical applications.

Of course, it should be verified that the behavior of the stainless steel reinforced concrete beam at room temperature is still satisfactory. Considering a reinforcement made of 3 Φ 16 stainless steel rebars, the maximum distributed load that the beam can sustain at room temperature is equal to $p_{max} = 26.18 \text{ kN/m}$. This load bearing capacity is thus higher than the maximum ELU load, which is $p_{ELU} = 20.02 \text{ kN/m}$. The ELU requirement is thus satisfied with the stainless steel reinforcement made of 3 Φ 16.

Under service conditions, the maximum vertical deflection of the stainless steel reinforced beam is 2.9 cm under a load of $p_{ELS} = 14.15 \text{ kN/m}$. This deflection corresponds to $L/207$, which is higher than

the limit of $L/250$. This means that the $3 \Phi 16$ reinforcement would not be enough to satisfy with ELS requirements; however in this study case, it was chosen not to take into account this criterion, because the study case considers an isostatic reinforced concrete beam whereas hyperstatic beams are more often used in practical applications. The present study case focus on the fire analysis and the boundary conditions that have been considered for the ELS analysis may be too unfavorable. Yet, it is very important to keep in mind that, if the stainless steel properties at high temperature may allow the designer to reduce the sections that are used, it should always be verified that the behavior at room temperature still satisfies with the ELU and ELS requirements at room temperature.

The main data for the reference case, with stainless steel reinforced concrete, are summarized in Table 4. The behavior of the stainless steel reinforced beams is showed in Figure 5 next to the behavior of the carbon steel reinforced beams.

Section		Case 1	Case 2
	Width	160 mm	160 mm
	Height	400 mm	400 mm
Reinforcement			
	Upper part	3 Φ 10 carbon	3 Φ 10 carbon
	Lower part	3 Φ 20 SLS1.4311	3 Φ 16 SLS1.4311
Cover			
	Lower part	30 mm	30 mm
Load (fire analysis)			
	Distributed load	10.0 kN/m	10.0 kN/m
Span			
	Isostatic beam	6.0 m	6.0 m

Table 4: Data for the stainless steel reinforced beam analysis

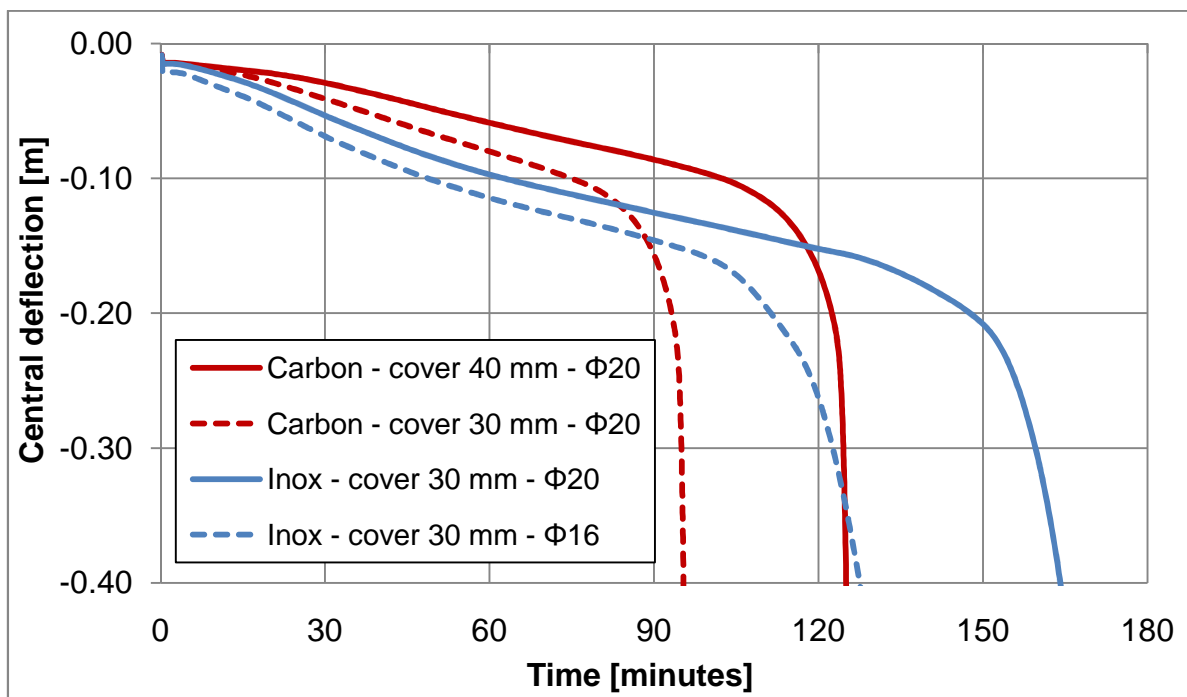


Figure 5: Deflection VS Time for the stainless steel reinforced beams

II.2.6. Final design

The main data of the two R 120 carbon and stainless –steel reinforced beams that will be compared in the economic analysis are summarized in Table 5:

Section		Case 1	Case 2
	Width	180 mm	160 mm
	Height	410 mm	400 mm
Reinforcement			
	Upper part	3 Φ 10 carbon	3 Φ 10 carbon
	Lower part	3 Φ 20 carbon	3 Φ 16 SLS1.4311
Cover			
	Lower part	40 mm	30 mm
Load (fire analysis)			
	Distributed load	10.0 kN/m	10.0 kN/m
Span			
	Isostatic beam	6.0 m	6.0 m
Room temp. analysis			
	Load bearing capacity	32.35 kN/m	26.18 kN/m
	Maximum deflection ELS	2.0 cm	2.9 cm

Table 5: Data for the final R 120 beams in carbon and in stainless -steel

It should be noted that the load bearing capacity at room temperature has been reduced considering the stainless steel reinforced beam, since the 3 Φ 20 carbon steel rebars have been replaced by 3 Φ 16 stainless steel rebars. However, this reduction of load bearing capacity has no influence in this study case as the maximum applied load at ELU, $p_{ELU} = 20.02 \text{ kN/m}$, is still lower than the load bearing capacity of the stainless steel reinforced beam, which is $p_{\max} = 26.18 \text{ kN/m}$. In fact, the ULS in the fire situation is more critical than the ULS at room temperature and it has governed the dimension of the bars.

II.3. Slab analysis

II.3.1. *Data*

The studied slab is 9.0 m width by 15.0 m long. It is supported by four peripheral steel beams and by two additional central steel beams (see Figure 6). The four peripheral beams are protected with thermal insulation whereas the two central beams are unprotected. The slab is vertically supported on its four corners, the vertical supports representing the columns of the building.

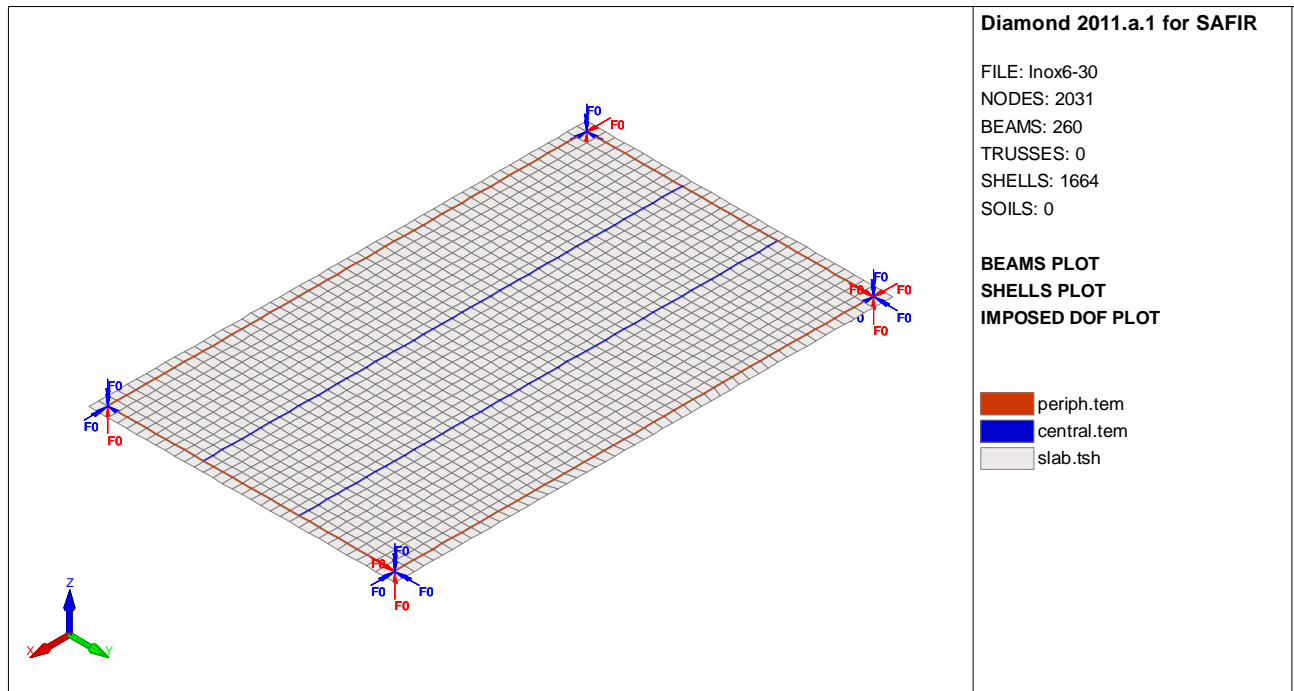


Figure 6: 3D view of the geometry of the slab

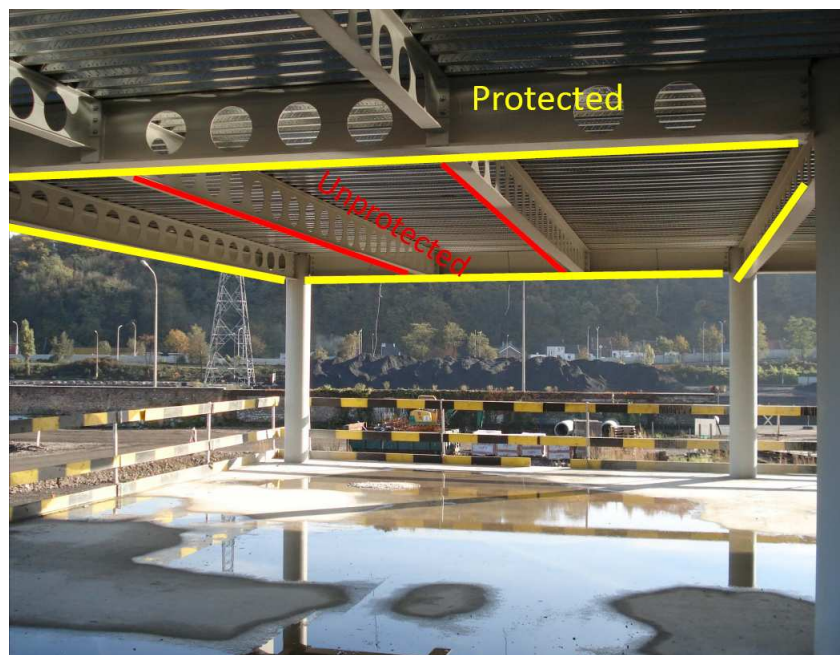


Figure 7: Example of actual building designed according to the same principles

The slab is a composite slab made of a steel deck and a concrete cover. The steel deck is a 51 mm deep profile of the Kingspan Multideck 50 of 1.0 mm thickness made of 350 N/mm² steel grade (Figure 8). The steel deck is not taken into account in the mechanical analysis because its adherence to the concrete at elevated temperature cannot be ensured. Moreover, the steel deck quickly reaches very high temperatures as it is not protected, its structural contribution thus being dramatically reduced in case of fire.

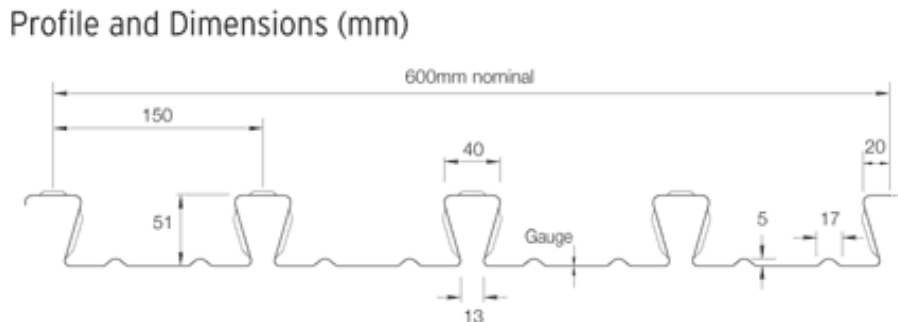


Figure 8: Steel deck used for the composite slab

The concrete layer above the profile is 69 mm high, which means that the total height of the composite slab is 120 mm. The concrete compressive strength at room temperature is 50 MPa.

The steel profiles are all IPE 600 made of S355 steel grade. Full interaction between the slab and beam is considered in the analysis.

The steel mesh has to be determined in order to ensure a fire resistance of 120 minutes in Standard fire conditions. It is considered that the steel mesh is positioned at 30 mm above the upper part of the steel deck, or in other words at 39 mm below the top of the slab. The present analysis aims to determine the minimum amount of steel mesh in carbon steel and in stainless steel that is required to reach R 120.

In this analysis, the columns are not modeled and are replaced by vertical supports.

The fire resistance is taken as the time at which the structure collapse is reached. In this study, the structure collapse is considered to be reached when the numerical simulation stops or when the structure maximum deflection reaches 1.00 m.

II.3.2. Mechanical load

The loads applied on the slab are those which are commonly used in the design of office buildings, see Table 6.

Description	Characteristics KN/m ²	Fire Factor	Design Load KN/m ²
Partition	1.00	1.00	1.00
Services & Finishes	0.50	1.00	0.50
Live Load	3.50	0.50	1.75
		Total	3.25

Table 6: Design loads

The self weight of the slab of 120 mm thickness is about 2.85 KN/m². The total load is thus 6.10 kN/m².

11.3.3. Thermal analysis

The thermal analysis is performed with the SAFIR software. The thermal properties for the stainless steel, carbon steel and concrete materials have been given previously.

The protected steel sections are affected by the ISO fire on one side and on the bottom flange, while the other side of the profile, in front of a wall, is supposed to be an adiabatic boundary, see Figure 9. For the protected sections, the insulation material is modeled. The fire protection of 2 hours for the peripheral steel beams can be reached by using silicate protective device panels, or by using a protection painting. In the thermal analysis, solid finite elements have been used for the insulation material. The insulation material has no mechanical resistance and thus it has not been considered for the mechanical analysis. The thermal properties for the insulation material have been chosen to ensure a fire protection of 2 hours for the peripheral beams:

- thermal conductivity is 0.04 W/mK
- specific heat is 1100 J/kgK
- volumic mass is 550 kg/m³
- water content is 16.5 kg/m³
- convection coefficient on hot surfaces is 25.0 W/m²K
- convection coefficient on cold surfaces : 4.0 W/m²K
- relative emissivity is 0.8

The unprotected steel sections, i.e. the central beams, are affected by the fire on the two sides and on the bottom flange, see Figure 11.

The concrete slab is modeled in order to take into account its capacity of absorbing heat. The material used for the concrete slab is siliceous concrete according to EN1992-1-2 with a specific mass of 2400 kg/m³, a moisture content of 72 kg/m³, a convection coefficient on hot surfaces of 25 W/m²K, a convection coefficient on cold surfaces of 4 W/m²K, a relative emissivity of 0.8 and a parameter for thermal conductivity of 0.5. The concrete above the upper flange of the steel profile is only considered for thermal analysis. The thermal concrete has no mechanical resistance in the beam, because the concrete slab is modeled by the shell elements. The bottom face of the slab is submitted to the ISO fire while the upper face of the slab is submitted to a frontier condition F20, i.e. that the upper face remains in contact with gas at 20°C during all the calculation, see Figure 9-11.

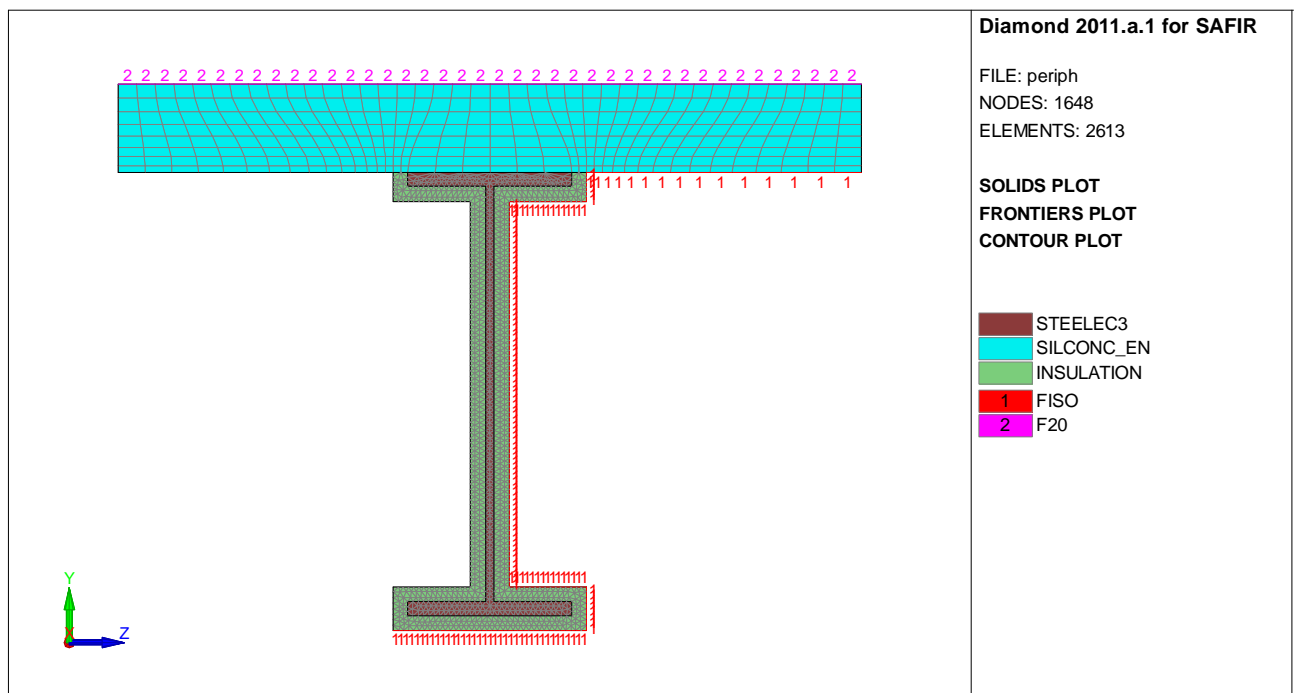


Figure 9: Model used for the thermal analysis of the protected edge beams

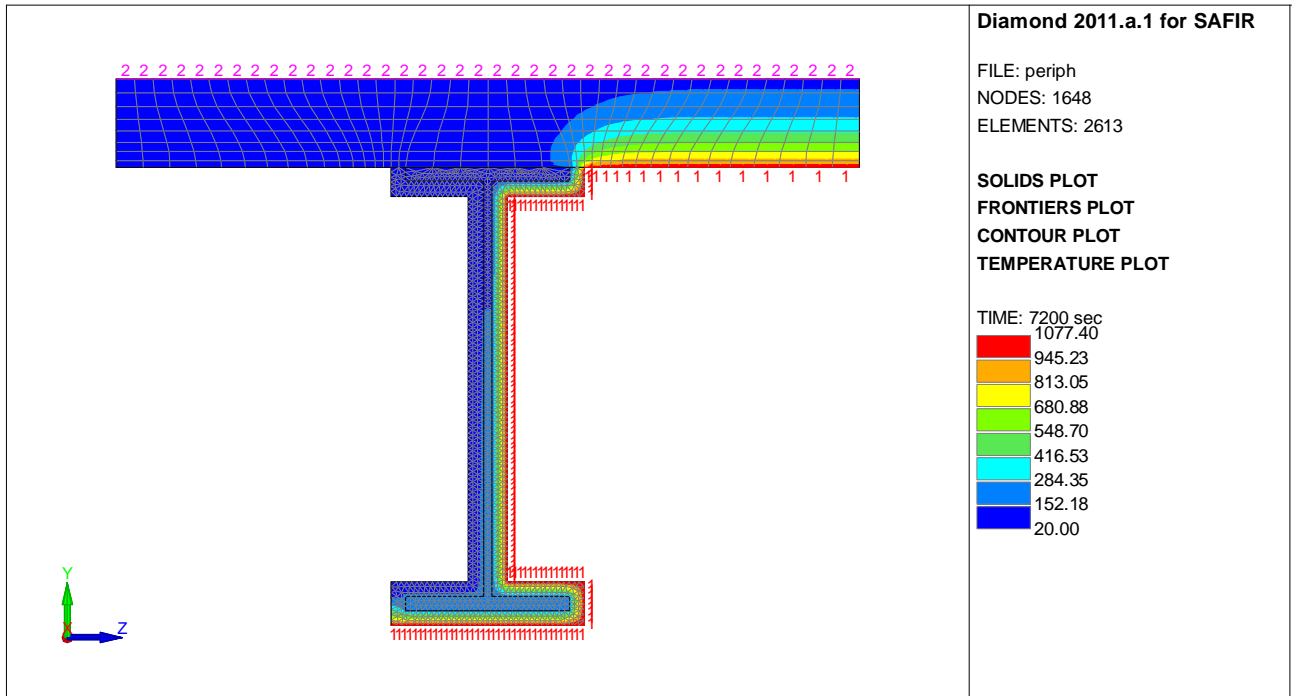


Figure 10: Temperature distribution in the protected edge beams after 120 minutes

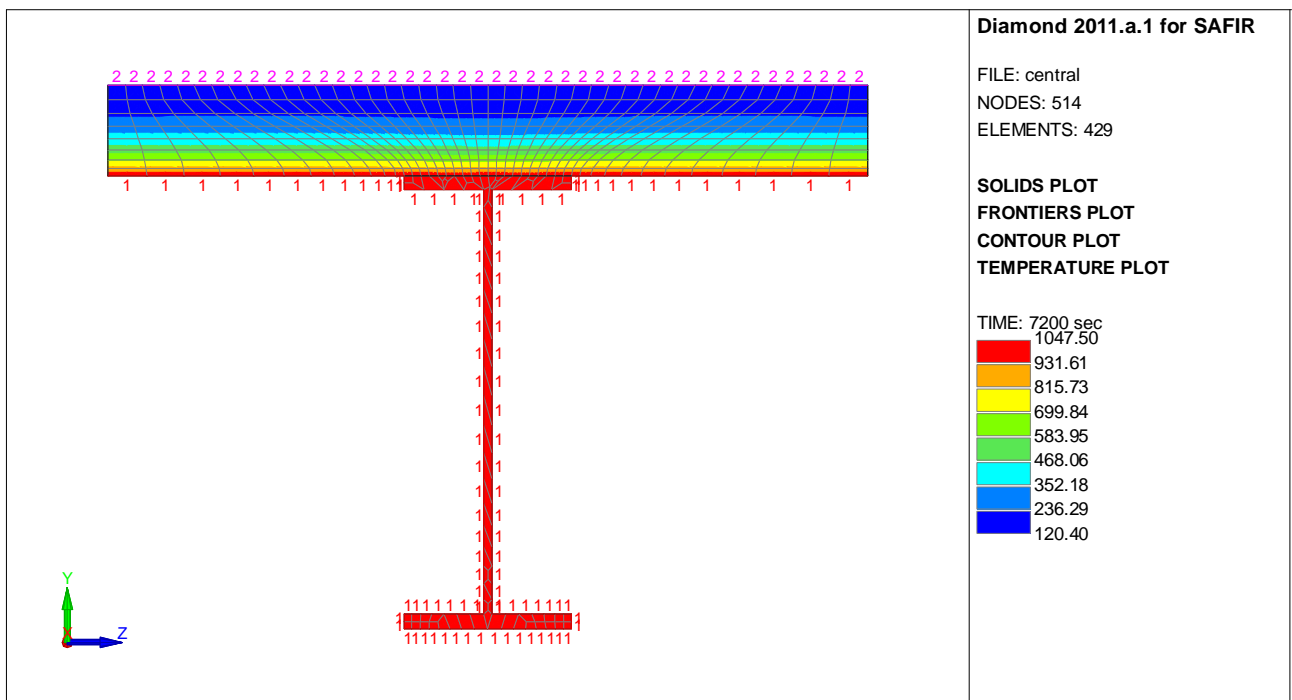


Figure 11: Temperature distribution in the unprotected central beams after 120 minutes

For the slab thermal analysis, the effective thickness model for the slab as defined in Eurocode EN1994-1-2 has been used. This effective thickness represents the height of the slab to consider for the thermal response. The height to consider for mechanical calculation is the concrete height above the steel deck. Here, the height of the structural concrete is 69.0 mm and the height of the additional thermal concrete (as defined in Annex D4 of EN 1994-1-2) is 41.6 mm. It means that, in the thermal analysis of the slab, the height of the slab is 110.6 mm plain concrete. It was verified that the geometric properties of the composite slab are in the field of application of the formula. The thermal properties of concrete have been given previously. The slab is submitted to the fire on its lower face and to a frontier condition F20 on its upper face, i.e. its upper face remains in contact with a gas at 20°C. Due to this model of the slab with an effective thickness, for the structural analysis, there is a gap of

$120.0 - 110.6 = 9.4 \text{ mm}$ between the upper flange of the steel sections and the bottom surface of the slab.

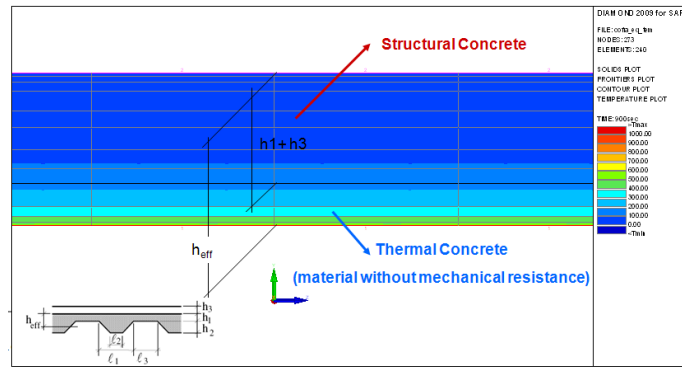


Figure 12: Model used for the thermal analysis of the slab

Figure 13 gives the temperature evolution in the steel profiles and in the reinforcement mesh.

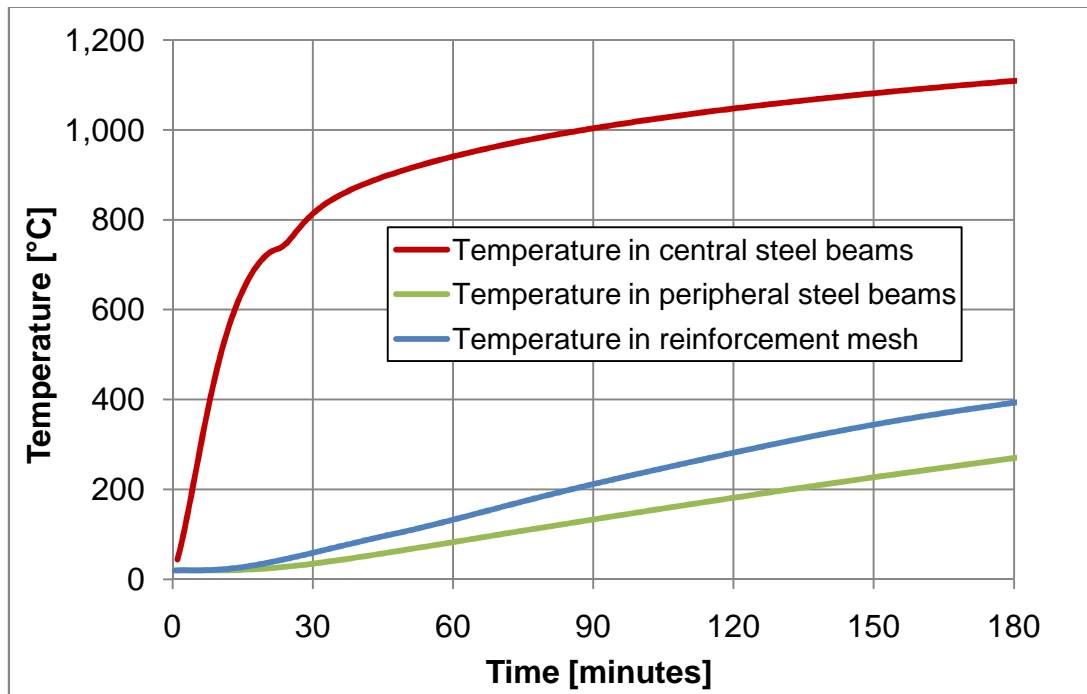


Figure 13: Temperature evolution in the steel profiles and in the reinforcement mesh

II.3.4. Structural analysis

II.3.4.1. Assumptions

A finite steel element model was built in the SAFIR software. The structure is modeled using BEAM elements for the beams and SHELL elements for the concrete slab. The edge beams are simply supported on the columns as indicated in Figure 6. The material used for the beams is steel according to EN1993-1-2 with yield strength of 355 MPa. The material used for the slab is siliceous concrete according to EN1992-1-2 with compressive strength of 50 MPa.

The structural behavior at room temperature is a flexional mode whereas during the fire, membrane action develops. Indeed, the central beams, which are unprotected, quickly lose their strength and stiffness due to the fire. In this case, the central part of the slab becomes highly cracked and the steel

mesh equilibrates the tensile forces. Large deflections occur in the slab. A compression ring develops in the concrete to equilibrate the horizontal forces, so that the membrane mode can work even though there is no lateral restraint. Figure 14 compares a slab in bending mode with a slab in tensile membrane action.

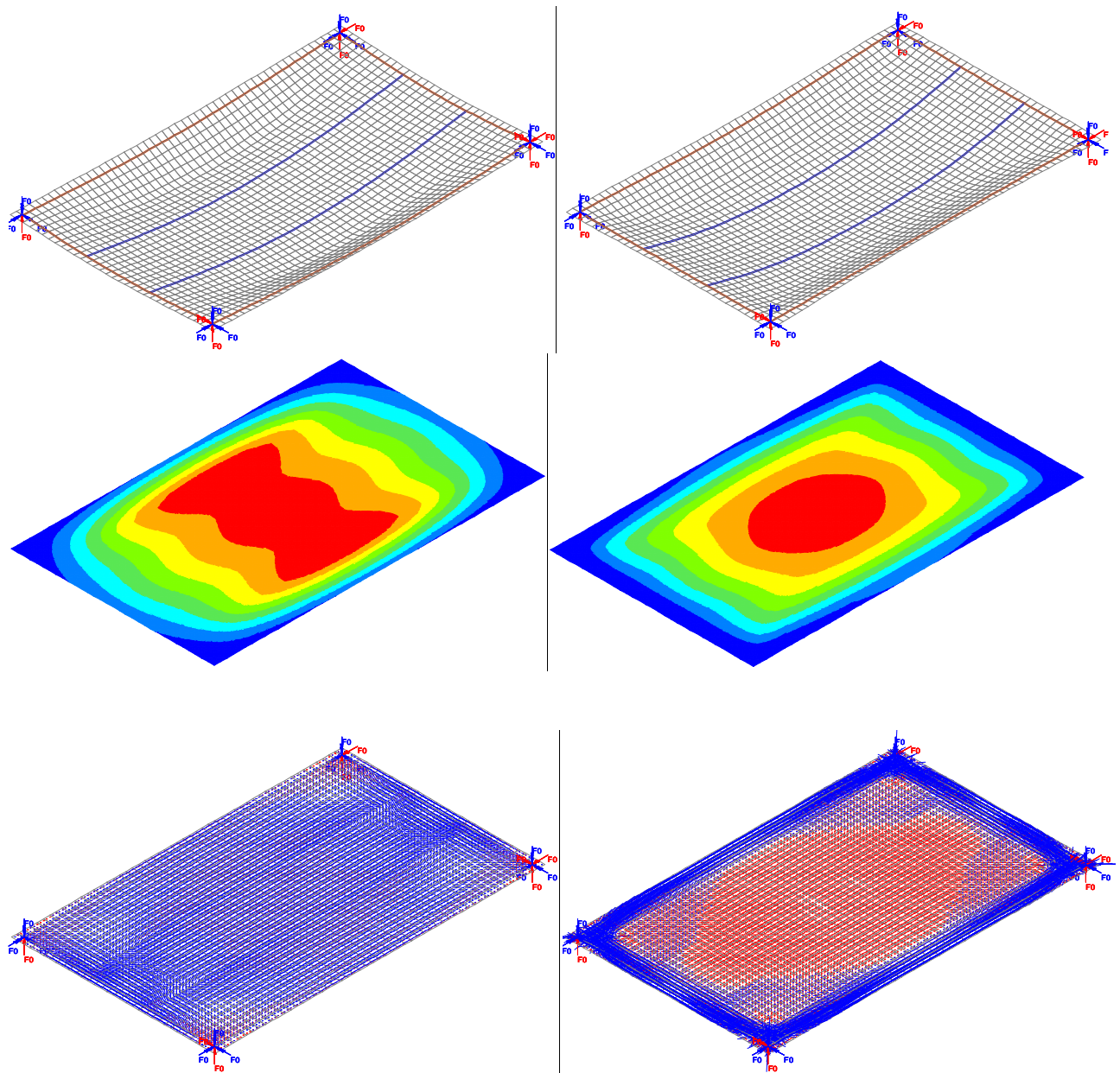


Figure 14: Comparison between bending mode (left) and tensile membrane action (right): deformed shape, vertical displacements and membrane forces for bending mode (at room temperature) and for membrane mode (at elevated temperature)

If carbon steel reinforcement is considered, no lateral restraint can be taken into account at the edges of the slab because of the low ductility of carbon steel. Indeed, due to the large rotations on the edges of the slab during the membrane action, it cannot be ensured that the carbon steel reinforcement will be able to sustain the local deformations at the edges of the slab without cracking. It is generally assumed that no lateral restraint can be considered at the edges of a slab during membrane action except if special measures are taken and it is demonstrated that the lateral restraint can sustain the large rotations.

In this study, it is thus considered that the slab and the beams are axially unrestrained when carbon steel reinforcement is used. However, with stainless steel reinforcement it can be assumed that the slab is laterally restrained as the stainless steel is much more ductile than the carbon steel and the ductility of the stainless steel is sufficient to sustain large rotations. Consequently it is considered that the stainless steel reinforced slab is laterally restrained.

II.3.4.2. Carbon steel reinforcement mesh

The carbon steel rebars yield strength is 500 MPa. The minimum carbon steel reinforcement mesh for a fire resistance of 120 minutes is a 251 mm²/m mesh, i.e. a carbon steel mesh made of 8 mm diameter reinforcement bars with 200 mm spacing between bars. The fire resistance R 120 is not reached using a carbon steel mesh made of 8 mm diameter reinforcement bars with 250 mm spacing between bars (201 mm²/m), as can be seen in Figure 15.

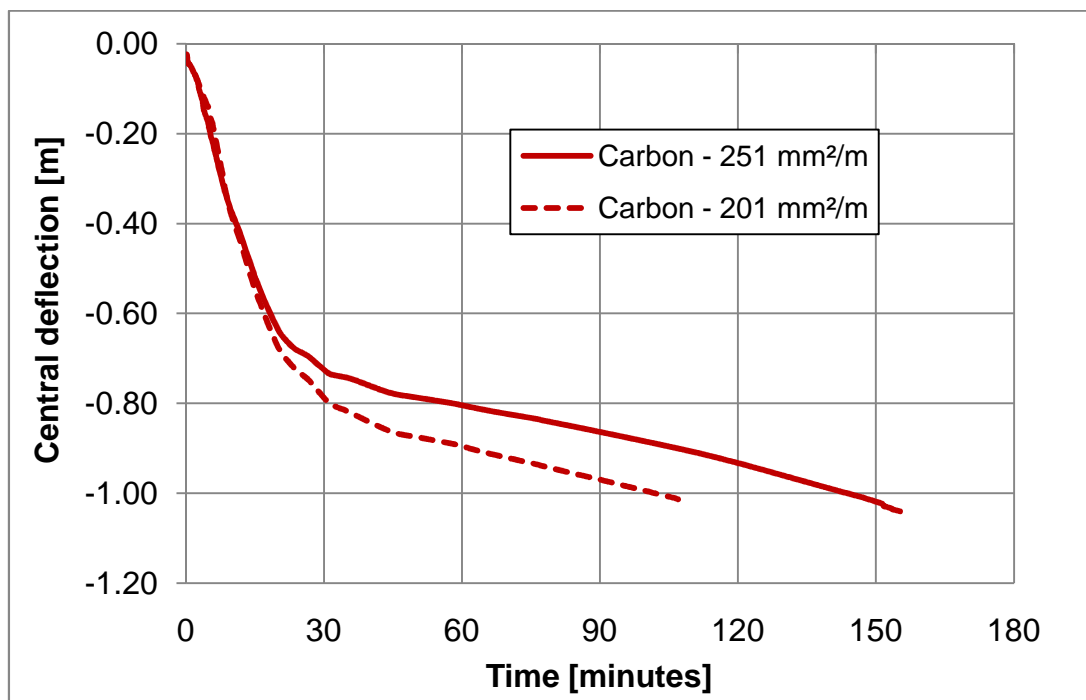


Figure 15: Evolution of the deflection at the center of the slab for carbon steel reinforcement

Note that the central vertical deflection at room temperature, considering the applied load of 6.10 kN/m and the carbon steel reinforcement mesh of 251 mm²/m, is equal to 4.2 cm.

II.3.4.3. Stainless steel reinforcement mesh

For the stainless steel reinforced slab with lateral restraint, it is possible to reach a fire resistance higher than 120 minutes considering a 141 mm²/m reinforcement, i.e. a stainless steel mesh made of 6 mm bars with 200 mm spacing between bars. However the fire resistance R 120 is not reached with a stainless steel mesh made of 6 mm bars with 250 mm spacing between bars (113 mm²/m), as shown in Figure 16.

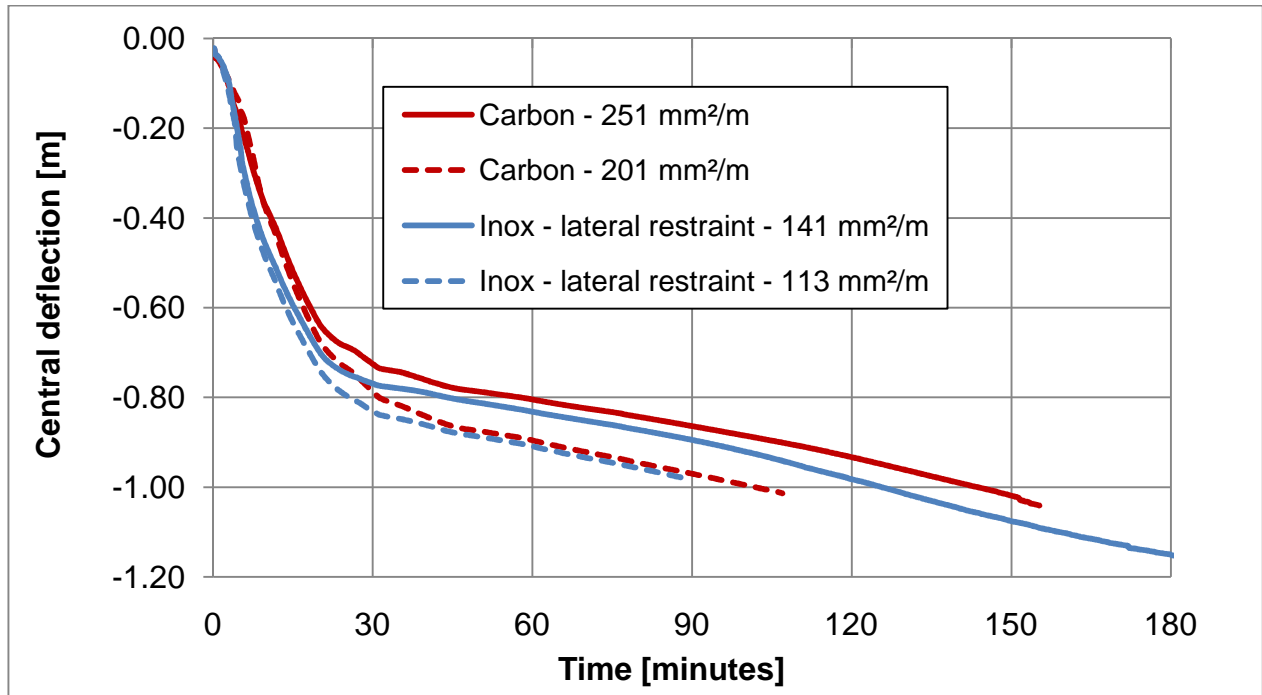


Figure 16: Evolution of the deflection at the center of the slab for stainless steel reinforcement

Note that the central vertical deflection at room temperature, considering the applied load of 6.10 kN/m and the stainless steel reinforcement mesh of 141 mm²/m, is equal to 3.5 cm. It is lower than the deflection with the carbon steel reinforcement because lateral restraints have been considered in this case.

II.3.5. Final design

The two R 120 slab solutions that will be compared in the economic analysis are summarized in the table below:

		Case 1	Case 2
Reinforcement			
	Steel mesh	Φ 8 / 200 mm carbon	Φ 6 / 200 mm stainless
Slab			
	Steel deck	51 mm deep Kingspan Multideck 50 1.0 mm thick	
	Concrete cover	69 mm C50 concrete	
Steel profiles			
		IPE 600 of S355 steel grade	
Supports			
	Vertical	Corners (modelling the four columns)	
	Lateral	unrestrained	restrained
Thermal			
	Insulation	R 120 thermal insulation on the peripheral beams	

Table 7: Data for the final R 120 slabs in carbon and in stainless -steel

III. ECONOMIC ANALYSIS

III.1. Unit costs

The data used for the economic analysis have been given by the Bureau Greisch, a design office located in Liege. The unit costs are given in the table below. These costs include the costs of the material delivered on the construction site and the direct labor costs of erecting the different components.

Unit costs (including direct labor costs)		
BEAM		
Concrete form (3 faces)	50	€/m ²
Concrete	135	€/m ³
Carbon steel rebars	1.80	€/kg
Stainless steel rebars	6	€/kg
SLAB		
Steel deck	35	€/m ²
Concrete	135	€/m ³
Carbon steel mesh	1.65	€/kg
Stainless steel mesh	6	€/kg
Laminated steel beams	3.5	€/kg
Thermal insulation: PROMAT	60	€/m ²
Thermal insulation: painting	200	€/m ²

III.2. Beam economic analysis

The following table gives the details of the calculation for the economic analysis of the carbon and stainless -steel reinforced beams.

Beam	carbon	stainless
Geometric data		
width (m):	0.18	0.16
height (m):	0.41	0.40
length (m):	6.00	6.00
Concrete data		
volume of concrete (m ³):	0.44	0.38
concrete form surface (m ²):	6.00	5.76
Rebars data		
steel density (kg/m ³):	7850	7850
Upper rebars diameter (m):	0.010	0.010
number of upper rebars:	3	3
upper rebars (m ³):	1.41E-03	1.41E-03
upper rebars (kg):	11.10	11.10
lower rebars diameter (m):	0.020	0.016
number of lower rebars:	3	3
lower rebars (m ³):	5.65E-03	3.62E-03
lower rebars (kg):	44.39	28.41
stirrups (m ² /m):	2.83E-04	2.83E-04
stirrups (kg):	7.86	7.46
carbon steel total weight (kg):	63.35	18.56
stainless steel total weight (kg):	0.00	28.41
kg steel by m ³ concrete:	143.07	122.32
Costs		
cost concrete form (€):	300.00 €	288.00 €
cost concrete (€):	59.78 €	51.84 €
cost carbon steel (€):	114.03 €	33.41 €
cost stainless steel (€):	0.00 €	170.46 €
total cost (€):	473.81 €	543.71 €
additional cost stainless steel (€):		14.8%

Table 8 : Economic analysis for the beams

As can be seen, the total cost for the carbon steel reinforced beam can be estimated to 473.81 euros, whereas it is estimated to 543.71 euros for the stainless steel reinforced beam. The additional cost for using stainless steel reinforcement instead of carbon steel reinforcement, considering a same fire resistance of 2 hours, is thus estimated to 14.8%. These costs include the material costs and the direct costs of erecting the beam.

III.3. Slab economic analysis

For the slab economic analysis, two different solutions have been studied for the thermal insulation of the peripheral beams. The thermal insulation can be performed by using an insulation painting or by using silicate panels, see Figure 17. The unit costs that have been supplied for these two solutions are approximately the following: 60 €/m² for the Promat panels and 200 €/m² for the painting.

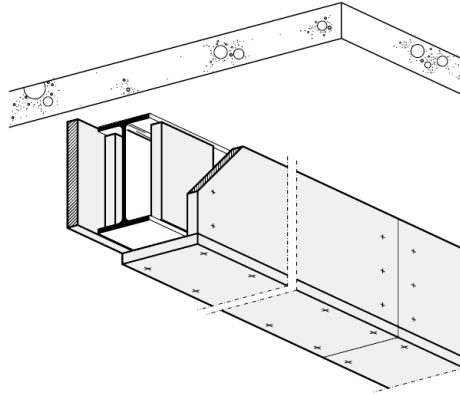


Figure 17: Thermal insulation of the beams using Promat panels

The details of the calculation for the economic analysis of the carbon and stainless steel reinforced slab are given in Table 9. It is important to notice that the slab that has been considered in the present analysis is assumed to be located in the center of a building, since lateral restraint has been assumed for this slab when using stainless steel reinforcement. Indeed, for a slab located at the edge or in the corner of a building, no lateral restraint can be considered in the structural analysis. As a consequence, in the economic analysis, only half of the peripheral beams have been considered.

As can be seen in Table 9, the additional cost of the stainless steel solution is about 6.8 €/m². The total cost for the carbon steel reinforced structure can be estimated to 32,894 euros using the Promat panels as thermal protection for the peripheral beams, whereas it is estimated to 33,813 euros (+2.8%) for the stainless steel reinforced structure also with the Promat panels. Using the unit costs for the R 120 painting, the total costs would be 39,662 euros for the carbon steel reinforced structure and 40,581 euros (+2.3%) for the stainless steel reinforced structure. These costs include the material costs and the direct costs of erecting the beam. Note that if only the composite slab and its reinforcement mesh are taken into account in the economic analysis, the additional cost of the stainless steel solution is +11.8% compared with the carbon steel reinforced solution.

slab	carbon	stainless
Geometric data		
width (m):	9	9
length (m):	15	15
slab area (m ²):	135	135
slab thickness (m):	0.12	0.12
Concrete data		
volume of concrete (m ³):	16.2	16.2
Steel deck data		
steel deck area (m ²):	135	135
Rebars data		
steel density (kg/m ³):	7850	7850
diameter of rebars (m):	0.008	0.006
spacing (m):	0.2	0.2
mesh (m ² /m):	2.51E-04	1.41E-04
mesh (m ³):	6.79E-02	3.82E-02
mesh (kg):	532.69	299.64
kg steel mesh by m ³ concrete:	32.88	18.50
Beams data		
center beams length (m):	30	30
peripheral beams length (m):	24	24
unit weight IPE 600 (kg/m):	122	122
total weight IPE 600 (kg):	6588	6588
Thermal insulation data (panels)		
height IPE 600 (m):	0.6	0.6
flange width IPE 600(m):	0.22	0.22
surface to protect (m ² box):	34.08	34.08
Thermal insulation data (painting)		
surface to paint (m ² /m):	1.84	1.84
surface to paint (m ²):	44.06	44.06
Costs		
cost steel deck (€):	4725.0	4725.0
cost concrete (€):	2187.0	2187.0
cost carbon steel mesh (€):	878.9	0.0
cost stainless steel mesh (€):	0.0	1797.8
cost IPE 600 beams (€):	23058	23058
cost promat box R 120 (€):	2044.8	2044.8
cost painting R 120 (€):	8812.8	8812.8
cost composite slab R 120 (€):	7790.9	8709.8
cost composite slab R 120 (€/m ²):	57.7	64.5
additional cost stainless steel (€):		11.8%
total cost with panels R 120 (€):	32893.7	33812.6
total cost with panels R 120 (€/m ²):	243.7	250.5
additional cost stainless steel (€):		2.8%
total cost with painting R 120 (€):	39661.7	40580.6
total cost with painting R 120 (€/m ²):	293.8	300.6
additional cost stainless steel (€):		2.3%

Table 9: Economic analysis for the slab

IV. CONCLUSION

An economic analysis has been performed to evaluate the practical interest of using stainless steel reinforcement instead of carbon steel reinforcement for applications in the field of fire structural engineering.

The structural analysis has highlighted the fact that it is possible for reinforced concrete members to reach the same performance in fire situation, i.e. in the present study a fire resistance of 2 hours, using a lower amount of rebars if stainless steel is used instead of carbon steel. This is due, on the one hand, to the higher resistance of stainless steel at elevated temperature and, on the other hand, to the important ductility of stainless steel which allows to sustain large rotations, for instance on the edges of a slab acting in membrane action.

However, the economic analysis has shown that the difference in unit cost between carbon steel and stainless steel has an important impact on the total cost of single structural members, even though the amount of reinforcement and/or the dimensions of this structural member can be reduced owing to the use of stainless steel (for the beam, the additional cost of using stainless steel reinforcement is close to 15%). The economic analysis of the slab shows that the additional cost of stainless steel reinforcement becomes less significant only if the economic analysis takes into account the entire structure, including for instance the steel beams and the thermal insulation. Three comments have to be made:

- First, it should be remembered that the unit costs that have been considered in this analysis imply that sufficient amount of the different products are used for the construction. Indeed, these unit costs include the cost of transport. Obviously, if only a very small amount of stainless steel was used (lower than 5 or 10 tons for instance), its unit cost would increase dramatically because of irreducible costs such as transport, and the solution using stainless steel would become uncompetitive compared to the carbon steel reinforced solution.
- The interesting properties of stainless steel at elevated temperature may allow for a reduction of the amount of reinforcement for a same fire resistance. However, the behavior at room temperature should always be verified; in other words, it should be verified that the stainless steel reinforced structural member has a satisfactory behavior also at room temperature.
- Finally, the higher ductility of the stainless steel reinforcement, compared with the carbon steel reinforcement, has allowed us to take into account the effect of the lateral restraint for the slab structural analysis. This lateral restraint has a positive effect on the behavior of a slab subjected to fire in which membrane action develops. However, it has to be mentioned that this lateral restraint cannot be considered for a slab located at the edge or in the corner of a building.