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ANALYTICAL METHOD FOR DESIGN OF FIRE RESISTANCE OF STEEL STRUCTURAL ELEMENTS

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Abstract *The engineering approach in the design of fire prevention allows the engineer to study fire prevention solutions that do not meet the requirements of the Rules provided that they ensure the same level of fire safety.*

To design interventions appropriate to ensure the fire resistance of structures, and in particular for the steel structures, the performance approach of the Rules requires the adoption of predefined solutions as a function only of the structural material and the required fire resistance.

The performance approach of the Norms, therefore, does not take into account the loads acting on the structural element and the critical temperature, which is the temperature at which the load that can be worn in hot equals the exercise load.

Often, during the design phase, some of the requirements of rules are difficult, if not impossible to meet. A practical example is that of a restoration project of a historic building, for which interventions are required non-invasive, reversible and compatible.

In this paper it is presented a study that enables you to design interventions appropriate to ensure the fire resistance of structural elements with an analytical method and, therefore, with an engineering approach. In particular, according to the fire scenario adopted, the analytical method proposed allows to determine the critical temperature of the structural element under study, depending on the working load, the resistant section and solicitation of project.

Subsequently, depending on the critical temperature and the required fire resistance, it is possible to design the appropriate fire-fighting interventions.

1. INTRODUCTION

With the law of 09.05.2007 Italian legislature, for the first time, introduces the engeneer approach in fire safety of building. Next to the prescriptive policy, ampiously applied in Italy, is thus present in firefighting regulatory framework introduced the criterion of performance of engineering approach [1].

With the engineering approach, the designer, using models and rigorous calculation procedures, plays what are the possible scenarios of fire that the building will face. Based on these scenarios, the designer will choose the most suitable design solutions, demonstrating the safety objectives set.

Calculation methods of structural fire resistance performance are based on the concept of the section reduced to hot, intending for reduced to heat section steel or concrete) that section able to equilibrate with its resistance to cold stress that can balance the section with its heat resistance [2].

2. CALCULATION OF THE FIRE RESISTANCE OF STEEL STRUCTURES

The design of fire-resistant steel structures with the analytical method must be checked in general that the carrying capacity last temperature θ P_θ, is greater than the working load Pe[3]:

$$P_\theta > P_e \qquad (1)$$

For the determination of P_θ it is necessary to know the temperature θ of the material as a result of exposure to fire for a time t, the reduction factor of compressive elastic modulus reduction factor Φc and Φ_E .

In the most general case, proceed with the following step:

- identifying the law of variation of ambient temperature (T) as a function of time (t) of exposure to fire: T = f (t)
- identifying the law of temperature change of material (θ) as a function of time (t) of exposure to fire: θ = f (T) = f (t)
- identifying the law of variation of resistance to heat (σ_θ) than the cold resistance (σ): $\Theta_\sigma = \sigma_\theta/\sigma = f(\theta)$
- identifying the law of variation of the elastic modulus in heat (E_θ) and the elastic modulus in cold (E):
  $\Theta_E = E_\theta / E = f(\theta)$

2.1. The law of variation of environmental temperature t

For scenarios of fire the standard defines three nominal curve: the standard curve, the curve of the hydrocarbons and the outside curve. The temperature in the surroundings of a structural member exposed to fire assumes, to vary the exposure time, the values given in table 1.

Hereafter, we will have fire scenario corresponding to the nominal standard curve:

$$T = 20+345 \log_{10}(8t+1) \qquad (2)$$

time [min]	Temperature [°C]		
	Standard nominal curve	Nominal hydrocarbon curve	Nominal outer curve
15	739	1.071	676
30	842	1.098	680
45	902	1.100	680
60	945	1.100	680
90	1.006	1.100	680
120	1.049	1.100	680
180	1.110	1.100	680
240	1.153	1.100	680
360	1.214	1.100	680

Table 1 – Ambiental temperature T

From the values given in table 1, we see that around the structure:

-the maximum temperature is reached after about 30 minutes of exposure to the fire for the scenario of fire of hydrocarbons;
-the maximum temperature is reached after about 15 minutes of exposure to an external fire scenario;
-the temperature increases rapidly in the first 45 minutes, then take a difference quotient descending for the scenario of fire standards.

2.2. THE LAW OF VARIATION OF TEMPERATURE Θ IN A MATERIAL

The law of variation of temperature θ in a steel profile is obtained by equating, at all times, the flow of heat that penetrates in the profile (proportional to the surface of the profile) to the amount of heat absorbed by the metal (proportional to the volume V of the profile).
In the case of homogeneous heating, acceptable hypothesis for value μ = S/V not less than 30 m^{-1}, the law of variation of temperature θ depending on time t, for a fixed value of μ, presents the trend referred to the curve of Figure 1.
From scientific experimentation exists in new Government logo shows that the values of θ in function of t and μ, for steel structural members exposed to fire rated standard, are those listed in table 2. The table fire exposure times not exceeding 30' because, in almost all cases, unprotected steel structures over 30' fire exposure you have the collapse of the material.
Table 2 data processing has allowed us to obtain the function θ = f (t), for fixed values of μ, is a third-order polynomial:

$$\theta = k_1 \cdot t^3 + k_2 \cdot t^2 + k_3 \cdot t + k_4 \quad (3)$$

with the values of the coefficients k_1, k_2, k_3 and k_4 reported in table 3.

t	μ=S/V [m^{-1}]														
[min]	50	75	100	125	150	175	200	225	250	275	300	325	350	375	400
2	36	45	53	61	69	76	84	92	100	107	115	123	130	136	145
4	72	97	121	144	167	189	209	230	247	267	285	302	313	334	349
6	117	161	203	241	277	310	340	368	394	417	437	457	474	489	503
8	167	231	288	338	383	421	455	485	510	531	549	564	577	587	596
10	221	302	370	428	476	516	547	572	593	609	622	632	639	646	650
12	275	371	447	507	554	589	616	636	651	662	670	676	680	684	687
14	330	436	515	574	616	646	666	681	691	698	703	707	710	712	714
16	383	496	575	629	665	688	703	714	721	726	729	732	734	735	737
18	433	550	626	673	703	721	732	740	745	748	750	752	754	755	756
20	482	598	668	709	733	747	755	760	764	767	768	770	771	772	773
22	527	641	704	739	757	768	774	778	781	783	784	785	786	787	788
24	568	678	734	763	777	785	790	793	796	797	799	800	801	801	802
26	607	710	759	783	794	801	805	807	809	811	812	813	813	814	815
28	642	738	781	800	809	814	817	819	821	823	824	825	825	826	826
30	674	762	799	815	822	826	829	831	833	834	835	835	836	837	837

Table 2 - Values of θ as a function of t and μ, for steel structural elements exposed to standard nominal fire

	μ [m^{-1}]							
	50	75	100	125	150	175	200	225
K_1	-0,0193	-0,0209	-0,0014	-0,0022	0,0118	0,0253	0,0375	0,0481
K_2	0,7894	0,5368	-0,0818	-0,8560	-1,6698	-2,4058	-3,0401	-3,5774
K_3	15,945	28,268	42,286	56,016	68,562	79,034	87,438	94,172
K_4	1,441	-14,457	-32,517	-48,766	-61,152	-69,198	-72,65	-72,284
	μ [m^{-1}]							
	250	275	300	325	350	375	400	
K_1	0,0570	0,0644	0,0704	0,0750	0,0788	0,0823	0,0840	
K_2	-4,0137	-4,3647	-4,6387	-4,8473	-5,0168	-5,1615	-5,2259	
K_3	99,403	103,300	106,090	108,040	109,630	110,640	110,700	
K_4	-69,963	-64,841	-57,904	-49,649	-43,336	-33,766	-22,815	

Table 3 - Values of coefficients k in function of μ

2.3. The law of variation of the heat resistance (σ_θ) than the cold resistance (σ).

The increase in the temperature of the steel causes an increase in the amplitude and frequency of the oscillations of the atoms around their balance position. This phenomenon causes a transformation of carbon atoms. In particular, around 700 °C steel passes from ferritic to austenitic, while at 1500 °C it becomes a liquid carbon and iron solution. These structural transformations naturally result in changes in the properties of the steels and at high temperatures, and there is a lowering of the breaking strength and the elasticity limit.

Figure 1 shows the steel tension-deformation diagram for temperatures varying from ambient temperature to 650 °C. From the figure it is noted that while the elasticity limit decreases regularly as the temperature rises, the breaking resistance increases to 200-300 °C and then decreases as the temperature rises.

Steel strength coefficient is defined as the ratio

$$\Phi_y = \frac{f_{y\theta}}{f_{yk}} \qquad (4)$$

where: f_{yk} is the characteristic tension of yield
$f_{y\theta}$ is the characteristic tension of heat yield

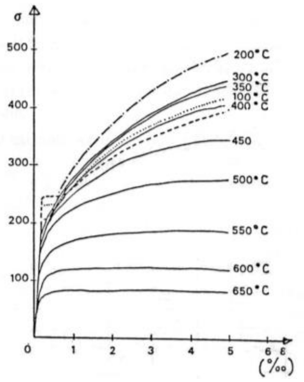


Figure 1 – Variability of the diagram σ-ε as a function of temperature θ

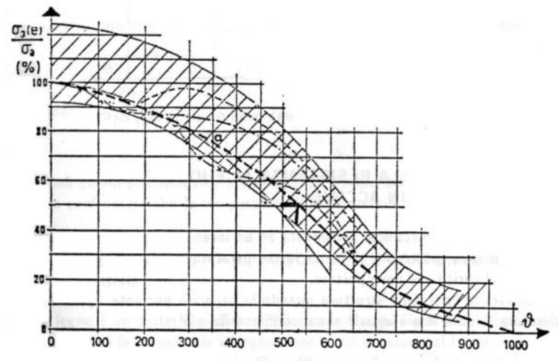


Figure 2 – Variability of Φy zone as a function of temperature θ

You can see in Figure 2, the Φy variation according to temperature θ.
The curve (a) shown in Figure 2, represents the law of variation of Φy proposed by D.T.U.
The study of the curve (a) allowed to derive the following mathematical expressions to calculate Φy as a function of θ:

for $0 < \theta \leq 600$ °C $$\Phi_y = 1 + \frac{\theta}{900 \ln \frac{\theta}{1750}} \qquad (5)$$

for $600 < \theta \leq 1000$ °C $$\Phi_y = \frac{340 - 0{,}34 \cdot \theta}{\theta - 240} \qquad (6)$$

Relationships (5) and (6) you can also write in inverse form:

for $0 < \theta \leq 600$ °C $$\theta = 745 \cdot \left(1 - (\Phi_y)^{1{,}3}\right)^{2/3} \qquad (7)$$

for $600 < \theta \leq 1000$ °C $$\theta = \frac{240 \cdot \Phi_y + 340}{\Phi_y + 0{,}34} \qquad (8)$$

Relationships (7) and (8) can be used to determine the critical temperature θ_{crit} steel, namely that temperature at which the load that can be brought to heat $P_{\theta crit}$ load cold door structure equals the Pe:

$$\theta_{crit} \quad \rightarrow \quad P_{\theta crit} = P_e$$

It should be noted, though, that while the report (8) is correct, the (7) is accurate.

2.4. The law of variation of the heat elastic modulus ($E_{y\theta}$) than the cold elastic modulus (E_y)

Defined the reduction factor of the modulus of elasticity:

$$\Phi_{Ey} = \frac{E_{y,\theta}}{E_y}$$

where E_y is the modulus of elasticity of steel
$E_{y,\theta}$ is the modulus of elasticity of heat

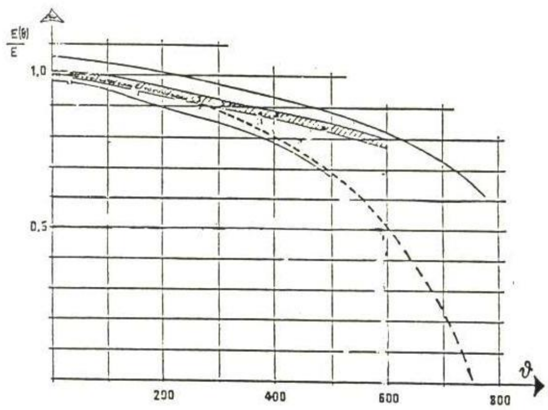


Figure 3 – Variability of ΦEy zone as a function of temperature θ

You can see in Figure 3, the time zone of elastic modulus variation as a function of temperature θ, in scientific literature. Taken as reference the curve in Figure 3 traits, has the function of θ ΦEy report below:

$$\Phi_{Ey} = 1 + \frac{\theta}{2000 \cdot \ln \frac{\theta}{1100}} \qquad (9)$$

3. APPLOCATION OF THE PROCEDURE FOR THE CALCULATION OF THE RESISTANCE OF STEEL STRUCTURES.

A metal structure subjected to a fire, loses its ability resistant as you increase the temperature generally remains constant while the operating load that must lead. You will have the structure crisis when load P_θ, that the structure is capable of bringing to the temperature θ, is less than the operating load Pe.

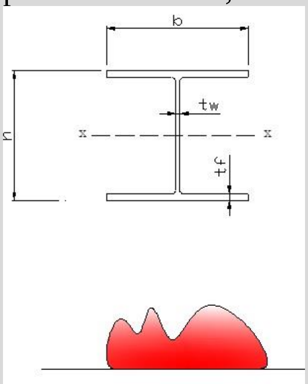
The calculation procedure generally consists of the following phases:

- calculation of θ temperature reached by the structure element exposed to fire;
- determination of the critical temperature θ_{crit};
- comparison of the critical temperature θ_{crit} and the temperature reached by the structural member.

In almost all cases the θ_{crit} is very low so the building has fire resistance time of less than 30 minutes of exposure to fire; the steel structure, therefore, must be protected and the θ_{crit} comes in handy for designing the isolation.

In Note 1 provides an application checking the fire resistance of unprotected steel for some structures.

1. Steel column

For a column formed by a profile HE A 220, are: 	N_e = 500 kN	compressive load of exercise
	Acciaio	S355
	f_{yk}=355 N/mm^2	the typical yield strength
	γ_{M0}=1,05	global partial factor
	h = 210 mm	the profile height
	b = 220 mm	the profile width
	t_w = 7 mm	the thickness of the core
	t_f = 11 mm	the thickness of the wing
	A = 64,34 cm^2	the area of the straight section
	W_x= 515,2 cm^3	the section modulus x-x

So:

$\mu = P/S = 1{,}225/(64{,}34 \cdot 10^{-4}) = 195 \approx 200\ m^{-1}$ — the mass ratio

$f_{yd} = f_{yk}/\gamma_{M0} = 355/1{,}05 = 338\ N/mm^2$ — the resistance of the steel calculation

For t = 30 min of exposure to standard fire curve nominal scenario using the (3) shows that the temperature of the steel holds:

$\theta_c^{30} = 0{,}0375 \cdot 30^3 - 3{,}0401 \cdot 30^2 + 87{,}438 \cdot 30 - 72{,}65 = 821\ °C$

Applying the (8) we get the strength reduction coefficient of steel:

$$\Phi_y^{30} = \frac{340 - 0{,}34 \cdot 821}{821 - 240} = 0{,}10475$$

Therefore, the load that can be worn in warm holds:
$N_\theta^{30} = (0{,}10475 \cdot 338 \cdot 64{,}34 \cdot 10^2 \cdot 10^{-3}) = 224$ kN in c.t.

At the end, $N_\theta^{30} < N_e$ and then for you can't take even a fire resistance of R30.

Consider the critical temperature θcrit, i.e. the temperature at which the load that can be brought to heat Nθcrit is equal to the operating load.
By imposing the condition:

$N_{\theta crit} = \Phi_{crit}\ [(f_{yd} \cdot A)] = N_e$

Therefor:

$\Phi_{crit} = N_e/[(f_{yd} \cdot A)] = 500/(338 \cdot 64{,}34 \cdot 10^2 \cdot 10^{-3}) = 0{,}233$

Applying the (7) we have:

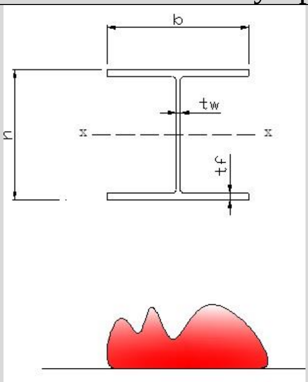
$$\theta_{crit} = 745 \cdot \left[1-(\Phi_{crit})^{1,3}\right]^{2/3} = 745 \cdot \left[1-(0{,}233)^{1,3}\right]^{2/3} = 668\ °C$$

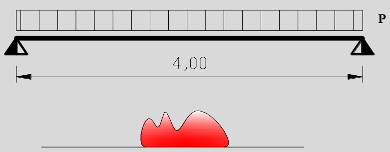
Temperature that is reached by applying (3) after 14 minutes, approximately.

Note the critical temperature you can design the frame needed to bring the fire resistance of the structure by 14 minutes fire resistance value required by the class of the building.

2. Steel beam

For a beam formed by a profile HE A 220, are





P_e = 40 kN/m	the uniformly distributed load of exercise
Acciaio	S355
f_{yk}=355 N/mm^2	the typical yield strength
γ_{M0}=1,05	global partial factor
h = 210 mm	the height of the profile
b = 220 mm	the width of the profile
t_w = 7 mm	the thickness of the core
t_f = 11 mm	the thickness of the wing
A = 64,34 cm^2	the area of the straight section
W_x = 515,2 cm^3	the section modulus x-x

It is estabished that the maximum bending moment of exercise is:
$M_e = (40 \cdot 4^2)/8 = 80$ kNm

The mass ratio is: $\mu = P/S = 1{,}225/(64{,}34 \cdot 10^{-4}) = 195 \approx 200\ m^{-1}$

the resistance of the steel calculation $f_{yd} = f_{yk}/\gamma_{M0} = 355/1{,}05 = 338$ N/mm^2

For t = 30 min of exposure to standard fire curve nominal scenario using the (3) shows that the temperature of the steel is:

$\theta_c^{30} = 0{,}0375 \cdot 30^3 - 3{,}0401 \cdot 30^2 + 87{,}438 \cdot 30 - 72{,}65 = 821$ °C

By applying the [5.45] we obtain that the coefficient of drag reduction of steel holds:

$$\Phi_y^{30} = \frac{340 - 0{,}34 \cdot 821}{821 - 240} = 0{,}10475$$

and, so, hot bending moment holds: $M_\theta^{30} = (0{,}10475 \cdot 338 \cdot 515{,}2 \cdot 10^{-3}) = 18{,}24$ kNm in c.t.

It is M_θ^{30} < Me so for you can't take even a fire resistance R30

Consider the critical temperature θcrit, i.e. the temperature at which time he can be brought to heat Mθcrit is equal to the time to exercise Me

By imposing: $M_{\theta crit} = \Phi_{crit}\,[(f_{yd} \cdot W)] = M_e$

We obtain: $\Phi_{crit} = M_e/[(f_{yd} \cdot W)] = (80*10^3)/(338 \cdot 515{,}2) = 0{,}46$

Applying the (7) we have:

$$\theta_{crit} = 745 \cdot \left[1-(\Phi_{crit})^{1,3}\right]^{2/3} = 745 \cdot \left[1-(0{,}46)^{1,3}\right]^{2/3} = 550\ °C$$

temperature that is reached by applying the [5.43], after 10 minutes.

Note the critical temperature you can design the frame needed to bring the fire resistance of the structure by 10 minutes a fire resistance value required by the class of the building.

NOTE 1

4. CALCULATION OF THE LINING NEEDED TO CLASSIFY THE STEEL ELEMENT WITH R DEFAULT.

For the structural element plan required to classify coating analytical steel with fire resistance R default, you need to know the θcrit, or the temperature of the material at which is:

$$N_{\theta cirt} = N_e$$

Given the critical temperature, using schedules you can design the cover necessary to classify the property at R default.

In the present state of scientific research were developed some 4les that can help you determine the temperature achieved by structural members protected according to the features of specific treatment.

Tab 4-7, drawn up for protective treatments with thermal resistance Rt = s/λ variable from 0,043 to 0,258 m^2 °C/W, create secure profile temperature depending on the ratio of mass and time of exposure to the standard fire rated curve.

	S/V [m^{-1}]							
[min]	50	100	150	200	250	300	350	400
5	25	49	72	94	116	136	156	175
10	62	119	171	217	258	296	330	360
15	104	192	267	330	383	428	466	499
20	147	263	354	427	485	532	570	602
25	189	329	432	510	569	615	651	678
30	231	390	501	581	639	682	713	736
35	272	446	562	641	696	734	761	779
40	311	498	616	696	744	777	798	813
45	349	545	664	738	783	811	829	840
50	385	589	707	775	815	839	854	863
55	420	629	745	808	843	863	875	882
60	453	667	778	836	866	883	893	899

65	484	701	807	860	886	900	909	914
70	514	733	834	881	904	916	923	927
75	543	762	857	899	919	929	935	939
80	571	788	878	915	933	942	947	951
85	597	813	896	930	945	953	958	961
90	623	835	913	943	957	964	968	971
95	648	856	928	955	967	973	977	980
100	671	875	942	966	977	982	986	988
105	694	892	954	976	986	991	994	996
110	715	909	966	986	994	999		
115	736	923	976	994				
120	756	937	986					

Table 4 – Thermic resistance 0,04 m^2 °C/W

t	S/V [m^{-1}]							
[min]	50	100	150	200	250	300	350	400
5	15	30	45	59	73	86	100	113
10	38	73	107	139	169	197	223	247
15	63	120	172	219	261	299	333	364
20	69	168	235	293	344	389	427	462
25	116	214	294	362	418	466	507	542
30	144	258	350	424	484	534	575	610
35	171	301	401	480	542	593	634	668
40	198	342	449	530	594	645	685	716
45	224	380	493	577	641	691	728	757
50	249	417	534	620	683	731	766	792
55	275	452	573	659	721	766	798	822
60	299	485	609	695	755	797	826	847
65	323	516	642	727	785	824	851	869
70	346	546	674	757	812	848	872	889
75	369	575	703	784	836	870	891	906
80	391	602	731	809	858	889	908	921
85	412	629	756	832	878	906	924	935
90	433	654	780	853	896	921	937	947
95	453	678	802	857	912	935	950	959
100	473	700	823	890	927	948	961	969
105	492	722	842	906	941	960	972	979
110	511	743	861	921	953	971	981	988
115	529	763	877	935	964	981	990	996
120	547	782	893	948	975	990	999	

Table 5 – Thermic resistence 0,086 m^2 °C/W

t	S/V [m^{-1}]							
[min]	50	100	150	200	250	300	350	400
5	9	17	25	34	42	50	58	66

10	21	42	62	81	99	118	135	152
15	35	69	101	131	159	186	211	235
20	50	97	141	180	217	251	285	311
25	66	126	180	228	272	312	348	380
30	82	154	218	274	324	368	407	442
35	98	183	255	318	372	420	461	498
40	114	210	291	359	417	467	510	548
45	130	237	325	398	459	511	555	594
50	146	264	358	435	498	552	597	636
55	162	289	389	469	535	590	636	674
60	178	314	419	502	570	625	671	709
65	194	339	448	534	602	658	704	741
70	210	362	476	563	633	689	734	769
75	225	385	502	592	662	718	761	796
80	240	407	528	619	690	744	787	820
85	255	429	552	645	715	769	810	842
90	270	450	576	670	740	792	832	862
95	285	470	599	693	763	814	852	880
100	299	490	621	715	784	834	870	897
105	313	509	642	737	804	853	887	912
110	327	528	663	757	823	870	903	927
115	341	546	682	776	841	886	918	940
120	355	564	701	795	858	902	931	952

Table 6 – Thermic resistence 0,172 m^2 °C/W

t	S/V [m^{-1}]							
[min]	50	100	150	200	250	300	350	400
5	6	12	18	24	29	35	41	46
10	15	29	43	57	70	84	97	109
15	24	48	71	93	114	135	154	173
20	35	68	100	130	158	185	210	234
25	46	89	129	167	201	234	264	292
30	57	110	158	202	243	280	314	345
35	68	131	187	237	283	324	361	395
40	80	152	215	271	321	365	405	441
45	92	172	242	303	357	404	446	484
50	103	193	269	334	391	441	485	523
55	115	213	295	364	424	476	521	560
60	127	232	320	393	455	508	555	595
65	139	252	344	421	485	540	587	628
70	150	271	368	447	513	569	617	659
75	162	289	391	473	540	598	646	687
80	173	308	413	497	567	625	673	714
85	185	326	435	521	592	650	699	740
90	196	343	455	544	616	675	724	763

95	208	360	476	566	639	698	746	786
100	219	377	495	588	661	721	768	806
105	230	394	515	608	683	742	789	826
110	241	410	533	628	703	762	808	844
115	252	426	551	648	723	781	826	861
120	262	441	569	667	742	799	843	878

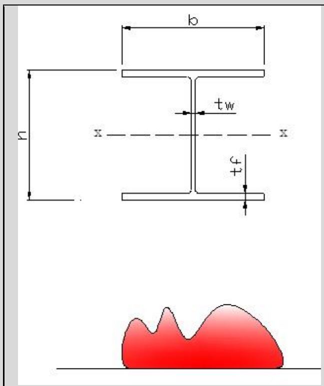
Table 7 – Thermic resistence 0,258 m^2 °C/W

As an example, we calculate the thickness of protective layer for some structural elements of steel.

Tab. 5, in particular, shows the calculation for the two structures referred to note 2.

1. Steel column

For a column formed by a profile HE A 220, are:



N_e = 500 kN	compressive load of exercise
Acciaio	S355
f_{yk}=355 N/mm^2	the typical yield strength
γ_{M0}=1,05	global partial factor
h = 210 mm	the profile height
b = 220 mm	the profile width
t_w = 7 mm	the thickness of the core
t_f = 11 mm	the thickness of the wing
A = 64,34 cm^2	the area of the straight section
W_x= 515,2 cm^3	the section modulus x-x

For the column in question, for the scenario of fire rated standard curve (see fact note 1) shows that:

the mass ratio is: $\mu = 200\ m^{-1}$

the critical temperature is: $\theta_{crit} = 668$ °C

We want to design security for having the strength class R120

From table 4 shows that we need a protection with thermal resistance

$Rt = s/\lambda = 0.258\ m^2$ °C/W

that can be done with:

<u>vermiculite panel</u> λ=0,23 s = (0,258·0,23) = 0,059 m = 6 cm

<u>perlite panel</u> λ=0,31 s = (0,258·0,31) = 0,079 m = 8 cm

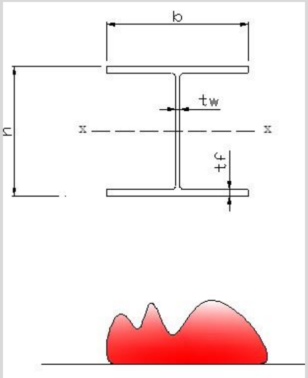
<u>gypsum panel</u> λ=0,24 s = (0,258·0,24) = 0,062 m = 6 cm

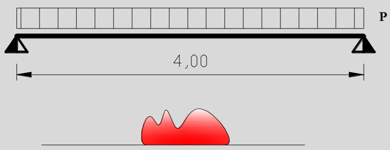
<u>concrete and clay panel</u> λ=0,30 s = (0,258·0,30) = 0,077 m = 8 cm

<u>cellular concrete panel</u> λ=0,10 s = (0,258·0,10) = 0,026 m = 3 cm

2. Steel beam

For a beam formed by a profile HE A 220, are:





P_e = 40 kN/m	the uniformly distributed load of exercise
Acciaio	S355
f_{yk}=355 N/mm^2	the typical yield strength
γ_{M0}=1,05	global partial factor
h = 210 mm	the profile height
b = 220 mm	the profile width
t_w = 7 mm	the thickness of the core
t_f = 11 mm	the thickness of the wing
A = 64,34 cm^2	the area of the straight section
W_x = 515,2 cm^3	the section modulus x-x

For the cbeam in question, for the scenario of fire rated standard curve (see fact note 1) shows that:
the mass ratio is: μ = 200 m^{-1}
the critical temperature is: θ_{crit} = 550 °C

We want to design security for having the strength class R90
From table 4 shows that we need a protection with thermal resistance

Rt = s/λ = 0.258 m^2 °C/W that can be done with:

<u>vermiculite panel</u> λ=0,23 s = (0,258·0,23) = 0,059 m = 6 cm

<u>perlite panel</u> λ=0,31 s = (0,258·0,31) = 0,079 m = 8 cm

<u>gypsum panel</u> λ=0,24 s = (0,258·0,24) = 0,062 m = 6 cm

<u>concrete and clay panel</u> λ=0,30 s = (0,258·0,30) = 0,077 m = 8 cm

<u>cellular concrete panel</u> λ=0,10 s = (0,258·0,10) = 0,026 m = 3 cm

NOTE 2

REFERENCES

[1] Fascia F. and Iovino R. "La struttura in cemento armato per l'architettura – tecnica e tecnologia". Aracne, Roma 2008

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