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Professional paper

ANALYSIS OF THE FIRE BEHAVIOR OF COMPOSITE COLUMNS OF STEEL AND CONCRETE IN MULTI-STORY BUILDINGS

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Abstract

Purpose: Unwanted fire is a destructive force that causes thousands of deaths and severe property damage each year. Fire is a spontaneous process of uncontrolled combustion with negative effects expressed through the destruction of material goods and endangering people's lives. Many of the artificial fires were caused by wars, other weather disasters, lightning, but, also by poor construction techniques used in the construction of buildings with highly flammable materials. The main factor that motivates us is to design a building that will have high fire safety with which there will be no great economic and human losses due to its premature collapse.

Method: Analysis of the fire resistance of construction structures according to technical regulations: The action of high temperature on interconnected-composite structures causes a decrease in the mechanical properties of the two components of the component (concrete and steel), and thus decreases in drastically their load capacity, then they cause major damage and breakage.

Fire resistance design of building structures is based on a certain fire scenario. Here are some of the types of fire scenarios that apply around the world. In most European countries, a standard fire time curve according to ISO 834 and a parametric curve are used, the standard curve is very close to the time-temperature curve used in the United States, ASTM E119. (American Society for Testing and Materials). Constructive behavior of composite elements in multi-story buildings exposed to high-fire temperatures, according to Eurocode 4, part 2. For composite columns, the control procedure: - bearing capacity of the cross section in compression and in moment (bending), - horizontal section, - the bearing capacity of the longitudinal section. Composite columns are considered, as unprotected steel elements, partially and completely concreted, in aspects of the structural scheme as freely supported or embedded elements. Nowadays, several software packages determine the fire resistance of the load-bearing elements of a composite structure. One of the most widely used software packages in most European countries is ArcelorMittal, which has been used in the analysis of fire resistance of structural elements-pillars of a multi-story building.

Results: The fire resistance of the pillars is calculated for three cases/ three types of pillars with a height of H = 3.3 m/ - as an unprotected pillar made only of steel profile; - protected with partial concrete cladding and fully concreted steel pillars. Fire resistance analysis of composite columns. The room temperature is taken as the initial temperature and the heating from the induced fire is done gradually according to the standard temperature time curve (ISO 834). - Complete plasticization occurred at the critical temperature of 684 °C (degrees Celsius), a temperature that, as a result of the ignition of the burning material, is reached for a time interval of At=28 minutes, in the case of the unprotected pillar that corresponds to the resistance class R15, which is smaller than what is provided by the regulations (ISO 834). For the partially concreted column, a class of fire resistance of R 120, this is a resistance that is expected for such objects, while in the case of fire resistance for completely concrete pillars, the fire resistance is R180, which is greater than the previous one, which is R120.

Conclusion: Unprotected pole - the critical temperature of collapse - collapse is Tkr = 673°C - The fire resistance of the pole in terms of time is At = 28.00min - the fire resistance class of the pole is: R 15, which is the most smaller than that provided for this type of elements Rmin120. The partially concreted column is carried out according to the rules provided by Eurocode 4 for steel and welded concrete. Constructions (Part 4). In the fire calculation, the standard temperature-time curve is used as in the previous elements. the resistance which in this case is equal to that provided for Class R 120. The fire resistance of the partially concreted pillar is R 120min and is equal to that provided for this type of element. The fully concreted pillar has a fire resistance of R 180 min and is larger than the previous one by regulation.

Keywords: composite constructions, composite columns, fire resistance, steel profile, concrete.

COMPOSITE COLUMNS

In multi-story buildings, three types of composite columns are most often used:

-composite pillars of fully wrapped steel elements

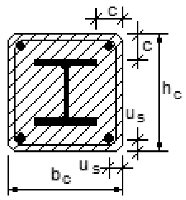
- composite poles with steel sections partially concreted and

-composite pillars made of steel profiles in cylindrical shapes filled with concrete.

1. Composite pillars of steel elements completely wrapped

Composite columns made of fully wrapped steel elements are classified according to function by the depth bc or hc, the protective layer of concrete c of the steel section, and the minimum axial distance of the reinforcing bars as given by the two alternative solutions in Table 1.1

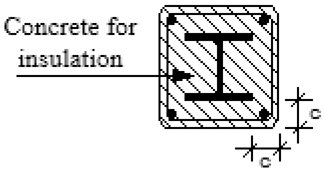
Table 1.1: Minimum dimensions of the cross-section, minimum protective layer of concrete in the steel section, and minimum axial distance of the reinforcing bars, for composite columns made of fully concreted steel sections.



		Standard fire resistance					
		R30	R60	R90	R120	R180	R240
1.1	Minimum dimensions hc and bc (mm)	150	180	220	300	350	400
1.2	Minimum protective layer of concrete on steel sections c (mm)	40	50	50	75	75	75
1.3	minimum axial distance of reinforcing bars us(mm)	20*	30	30	40	50	50
	OR						
2.1	Minimum dimensions hc and bc (mm)	-	200	250	350	400	-
2.2	Minimum protective layer of concrete on the steel section c (mm)	-	40	40	50	60	-
2.3	minimum axial distance of reinforcing bars us(mm)	-	20*	20*	30	40	-

For R30, the concrete is only placed between the steel shear bands. If the concrete wrap around the steel section has only an insulating function, when designing the column for normal temperature, the fire resistance R30 to R180 can be met, in addition to the concrete protective layer c of the steel section according to Table 1.2.

Table 1.2. A minimum protective layer of concrete for the steel section, when concrete plays the role of fire protection material.

	Standard fire resistance				
	R30	R60	R90	R120	R180
Concrete protective layer c (mm)	0	25	30	40	50

Where the concrete casing has a special insulating function, mesh reinforcement is required, in addition to R30.

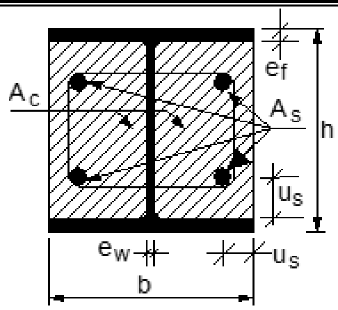
The reinforcement must contain at least 4 bars with a diameter of 12 mm. In all cases, the minimum percentage of longitudinal reinforcement must meet the requirements of EN 2.

The maximum percentage of longitudinal reinforcement as well as beams must also meet the requirements of EN 2.

1.2.1 Composite columns with partially concreted steel sections

Partially concreted steel section composite columns are classified according to load level, depth b or h, minimum axial distance of reinforcing bars and the ratio between rib thickness eW and flange thickness ef s iç is given in Table 1.3.

Table 1.3 Minimum cross-sectional dimensions, minimum axial spacing and minimum percentage of reinforcement for composite columns of partially concreted steel sections

		**Standard fire resistance**			
		R30	**R60**	**R90**	**R120**
	Minimum rib to flange thickness ratio e_w/e_f	0,5	0,5	0,5	0,5
1	Minimum cross-sectional dimensions for loading level $\eta_{fi,t} \le 0,28$				
1.1	minimum dimensions h and b [mm]	160	200	300	400
1.2	minimum axial distance of reinforcing bars us [mm]	-	50	50	70
1.3	the minimum percentage of armament $A_s/(A_c+A_s)$ во %	-	4	3	4
2	Minimum cross-sectional dimensions for loading level $\eta_{fi,t} \le 0,47$				
2.1	minimum dimensions h and b [mm]	160	300	400	-
2.2	minimum axial distance of reinforcing bars us [mm]	-	50	70	-
2.3	the minimum percentage of armament $A_s/(A_c+A_s)$ во %	-	4	4	-
3	Minimum cross-sectional dimensions for loading level $\eta_{fi,t} \le 0,66$				
3.1	minimum dimensions h and b [mm]	160	400	-	-
3.2	minimum axial distance of reinforcing bars us [mm]	40	70	-	-
3.3	the minimum percentage of armament $A_s/(A_c+A_s)$ во %	1	4	-	-

When determining Rd and Rfi,d,t = ηfi,t Rd, respectively with Table 1.3, the percentage of reinforcement As / (Ac + As) higher than 6% or lower than 1% is not taken into account. Table 1.3. can be applied to steel grades S 235, S 275, and S 355.

With these composite columns, the concrete between the flanges of the steel section is fixed to the ribs with either stirrups or bolts (cerebral steel elements) (Fig. 1.2.1a). The stirrups must be attached to the ribs or penetrate the ribs through holes. If bolts are used, they should be attached to the ribs.

The distance between bolts or staffs along the axis of the pole should not be greater than 500 mm. In the parts where the load is applied, this distance must be reduced and taken according to Eurocode 2.

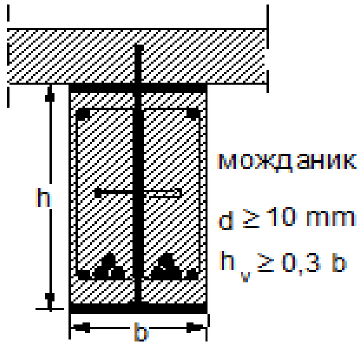
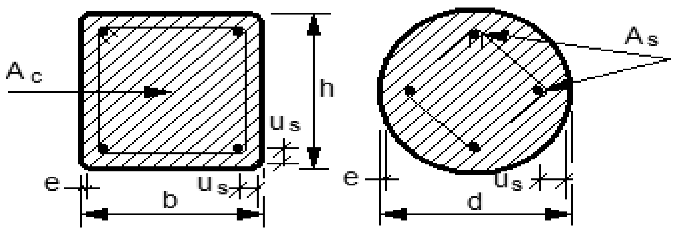


Fig. 1.2.1a

1.2.2 Composite pillars made of steel profiles in cylindrical tube shapes, filled with concrete

Composite columns made of steel profiles in cylindrical tube shapes, filled with concrete are classified according to the load level ηfi,t, the size of the cross-section b, h, or d, the percentage of reinforcement As / (Ac + As) and the minimum distance axes of reinforcing bars according to table 1.4.

Table 1.4 Minimum cross-sectional dimensions, minimum distance between axes of reinforcing bars, and minimum reinforcement coefficients for composite columns consisting of hollow sections filled with concrete.

	 Steel section: (b / e) ≥ 25 or (d / e) ≥□ 25	**Standard fire resistance**				
		R30	R60	R90	R120	R180
1	Minimum cross-sectional dimensions for loading level $\eta_{fi,t} \leq 0,28$					
1.1	minimum dimensions h and b [mm]	160	200	220	260	400
1.2	minimum axial distance of reinforcing bars us [mm]	0	1,5	3,0	6,0	6,0
1.3	the minimum percentage of armament $A_s/(A_c+A_s)$ во %	-	30	40	50	60
2	Minimum cross-sectional dimensions for loading level $\eta_{fi,t} \leq 0,47$					
2.1	minimum dimensions h and b [mm]	260	260	400	450	500
2.2	minimum axial distance of reinforcing bars us [mm]	0	3,0	6,0	6,0	6,0
2.3	minimum arming percentage $A_s/(A_c+A_s)$ во %	-	30	40	50	60
3	Minimum cross-sectional dimensions for loading level $\eta_{fi,t} \leq 0,66$					
3.1	minimum dimensions h and b [mm]	260	450	550	-	-
3.2	minimum axial distance of reinforcing bars us [mm]	3,0	6,0	6,0	-	-
3.3	minimum arming percentage $A_s/(A_c+A_s)$ во %	25	30	40	-	-

When determining Rd and R fi,d,t = η fi,t Rd, according to Table 1.1, the following rules apply:

-Regardless of the type of steel for pipe-cylindrical cuts, the nominal point e is considered

yield strength of 235 N/mm²;

- the wall thickness of the tubular cross-section is taken at most 1/25 of b or d;

- As / (Ac + As) reinforcement percentages greater than 3% are not considered;

- Concrete resistance is taken during design for normal temperature.

The values given in Table 1.4 apply to steel type S 500 used for as reinforcement.

The criteria that these pillars must meet according to Eurocode 2 are:

- Between the beam-column connections, there should not be an additional bending shear connection along the column; - additional armor to be reinforced with stirrups and spacers;

- The distance between the stirrups along the pole axis must not be greater than 15 times the smallest diameter of the longitudinal armature;

- The tubular-cylindrical section of steel must contain holes with a diameter of not less than 20 mm, located at least one at the top and one at the bottom of the column for each floor.

- the distance between such holes should never be greater than 5 m.

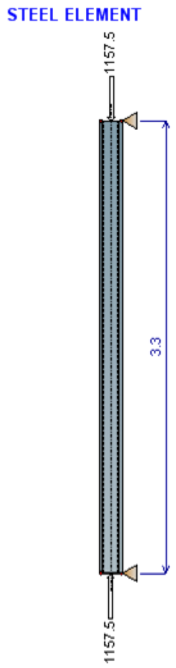
Tables 1.2, 1.3 and 1.4 apply to rigid frames. The load level in 1.3 and table 1.4 is determined by the load level of the fire model where for the calculation of Rd it is assumed that the columns are supported at the joints and in case of fire both ends of the column are equipped with hindered rotation-rotation. This is common in practice when it is assumed that only the sole under consideration is exposed to fire. When using tables 1.3 and 1.4, when calculating Rd, twice the length of the fire used during design-prediction should be taken.

Tables 1.2 to 1.4 apply to both central and eccentric axial loads acting on columns. When calculating Rd, the design load at normal temperature, the eccentricity of the load must be taken into account. The tabular data given in Tables 1.2 to 1.4 are valid for columns with a maximum length 30 times greater than the minimum external dimensions of the section selection criteria.

2 COMPOSITE COLUMNS

2.1 Unprotected steel pole-POS 370

FIRE DEPARTMENT-1ST FLOOR



I - PARAMETERS

I.1 - General parameters

Element type :	Steel element
Span length of the member :	L = 3.3 m
Buckling length :	$L_{cr,y}$ = 3.3 m
Orientation :	Vertical
Supports :	Simply supported
Bending axis :	y-y

I.2 - Position of the member

The considered member is an intermediate member.

Spacing of the considered member :
- To the left member : D_1 = 6 m
- To the right member : D_2 = 6 m

I.3 - Cross-section

Profile :	HE 200 B
Grade :	S355 - JR/J0/J2/K2
Standard :	EC3

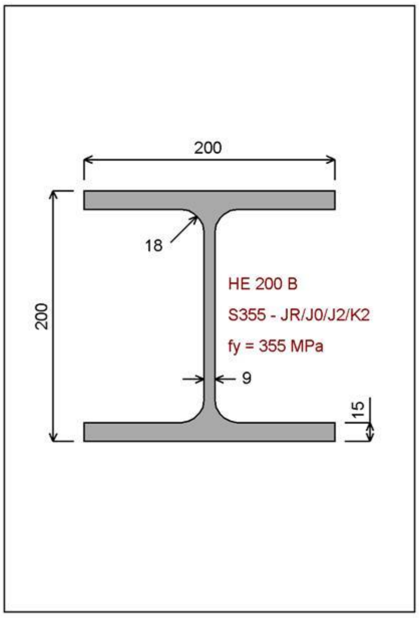


Figure 1 : Cross-section

- Geometrical properties :

A	= 149.08 cm^2	m	= 117.026 kg/m
$A_{v,y}$	= 114 cm^2	$A_{v,z}$	= 47.43 cm^2
I_y	= 25165.69 cm^4	I_z	= 8562.83 cm^4
$W_{el,y}$	= 1677.71 cm^3	$W_{el,z}$	= 570.86 cm^3
$W_{pl,y}$	= 1868.67 cm^3	$W_{pl,z}$	= 870.14 cm^3
I_t	= 185.77 cm^4		
I_w	= 1687791.86 cm^6		

- Steel properties :

Grade :	S355 - JR/J0/J2/K2
Standard :	EN 10025-2 : 2004
Flange :	f_y = 355 MPa
Web :	f_y = 355 MPa
Cross-section :	f_y = 355 MPa
	ε = 0.81

I.4 - Lateral restraints

Lateral restraints of the member :

Restraint No.	x(*) (m)	Restrained top flange	Restrained bottom flange
1	0	X	X
2	3.3	X	X

(*) : Location from the left end

I.5 - Load cases

- Load case 1 : Permanent loads (G)

Member weight : 1.148 kN/m

Axial force : N_{Ed} = 1157.5 kN

I.6 - Calculation parameters

Partial factors on loads :
$\gamma_{G,sup}$ = 1.35
$\gamma_{G,inf}$ = 1.00
γ_Q = 1.50

Partial factors on resistances :
γ_{M0} = 1.00
γ_{M1} = 1.00

Partial factors for fire calculation :
$\gamma_{M,fi,concrete}$ = 1.00
$\gamma_{M,fi,steel}$ = 1.00
$\gamma_{M,fi,reinforcing\ bars}$ = 1.00

National Annexes: No

Design plastic resistance under combined actions (M-N or M-N-V): Exact calculation

The buckling resistance of members in bending is calculated using Clause 6.3.2.2 (EN 1993-1-1)

V.2 - Synthesis of verifications

- Resistance of cross-sections

Resistance of the cross-section to axial force (Comb. ULS01 (Auto)) $\Gamma_{N,max}$ = 0.296 < 1 => *Satisfied*

Resistance of the cross-section to shear force
at x = 0 m, Comb. ULS01 (Auto): $\Gamma_{Vz,max}$ = 0 < 1 => *Satisfied*

Resistance of the cross-section to bending moment
at x = 0 m, Comb. ULS01 (Auto): $\Gamma_{My,max}$ = 0 < 1 => *Satisfied*

Resistance of the cross-section to combined actions M-N
at x = 0 m, Comb. ULS01 (Auto): $\Gamma_{MyN,max}$ = 0 < 1 => *Satisfied*

Resistance of the cross-section to combined actions M-V
at x = 0 m, Comb. ULS01 (Auto): $\Gamma_{MyV,max}$ = 0 < 1 => *Satisfied*

Resistance of the cross-section to combined actions M-N-V
at x = 0 m, Comb. ULS01 (Auto): $\Gamma_{MyNV,max}$ = 0 < 1 => *Satisfied*

- Web resistance

No verification required ($V_{z,Ed}$ = 0)

- Member resistance

Flexural buckling about the major axis (Comb. ULS01 (Auto)):
Γ_{by} = 0.311 < 1 => *Satisfied*

Flexural buckling about the minor axis (Comb. ULS01 (Auto)):
$\Gamma_{bz,max}$ = 0.369 < 1 => *Satisfied*

Lateral torsional buckling: No verification required

M-N buckling interaction: No verification required

VI - SERVICEABILITY LIMIT STATES (SLS)

VI.1 - Deflections

Maximum deflection :

$w_{z,max}$ = 0.0 mm at x = 0 m (Combination SLS01 (Auto))
$w_{z,3,max}$ = 0.0 mm at x = 0 m (Combination SLS01 (Auto))

Verification :

$w_{z,max}$ = 0.0 mm < L/200 = 16.5 mm => *Satisfied*
$w_{z,3,max}$ = 0.0 mm < L/250 = 13.2 mm => *Satisfied*

- Member resistance at 28 min - Flexural buckling

Buckling length about the major axis	$l_{fi,y}$	= 1.65 m
Buckling length about the minor axis	$l_{fi,z}$	= 1.65 m
Non-dimensional slenderness about the major axis	$\lambda_{y,\theta}$	= 0.215
	$\phi_{y,\theta}$	= 0.58
Non-dimensional slenderness about the minor axis	$\lambda_{z,\theta}$	= 0.369
	$\phi_{z,\theta}$	= 0.666
Reduction coefficient for buckling	$\chi_{fi,min}$	= 0.82
Member resistance	$N_{b,fi,t,Rd}$	= 1192.5 kN
Criterion	$\Gamma_{fi,b}$	= 0.974 < 1 =>*Satisfied*

- Member resistance at 28 min - Lateral torsional buckling

No verification required because $M_{fi,Ed} = 0$

- Member resistance at 28 min - M-N buckling interaction

No verification required because $M_{fi,Ed} = 0$

VII.4 - Synthesis of verifications

- Resistance of cross-sections

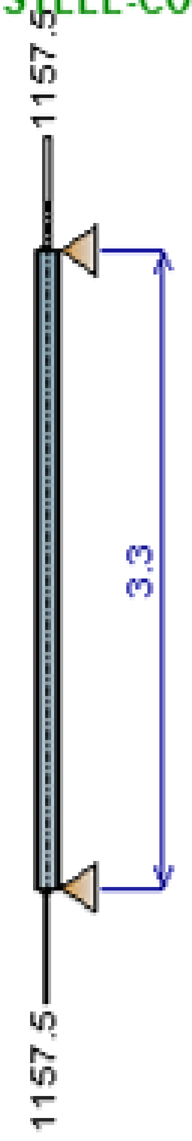
Standard fire resistance	**R15**	
Temperature of the cross-section of the member	θ_a	= 681 °C
Resistance to axial force (Comb. FIRE01 (Auto))	$\Gamma_{fi,N,max}$	= 0.798 < 1 =>*Satisfied*
Resistance to shear force (at x = 0 m, Comb. FIRE01 (Auto))	$\Gamma_{fi,V,z,max}$	= 0 < 1 =>*Satisfied*
Resistance to bending moment (at x = 0 m, Comb. FIRE01 (Auto))	$\Gamma_{fi,M,y,max}$	= 0 < 1 =>*Satisfied*
Resistance to M-N interaction (at x = 0 m, Comb. FIRE01 (Auto))	$\Gamma_{fi,MyN,max}$	= 0.798 < 1 =>*Satisfied*
Resistance to M-V interaction (at x = 0 m, Comb. FIRE01 (Auto))	$\Gamma_{fi,MzVy,max}$	= 0 < 1 =>*Satisfied*

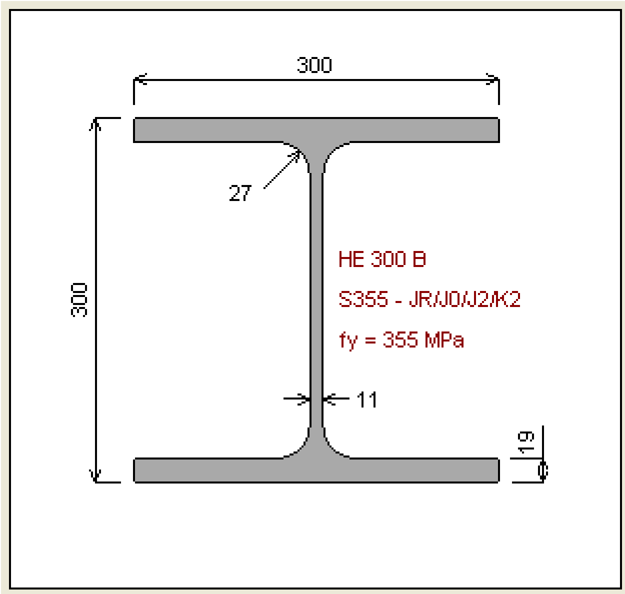
- Member resistance

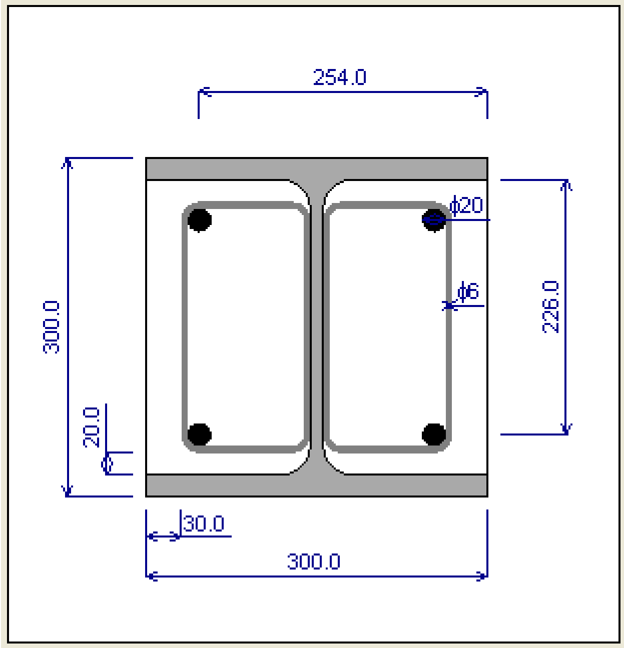
Flexural buckling (Comb. FIRE01 (Auto))	$\Gamma_{fi,b}$	= 0.974 < 1 =>*Satisfied*
Lateral torsional buckling	No verification required	
M-N buckling interaction	No verification required	

2.2 POS 370 concrete partial composite column

PARTIALLY ENCASED COMPOSITE STEEL-CONCRETE COLUMN




300
300
27
HE 300 B
S355 - JR/J0/J2/K2
fy = 355 MPa
11
19


254.0
ϕ20
ϕ6
300.0
226.0
20.0
30.0
300.0

I - PARAMETERS

I.1 - General parameters

Element type : Partially encased composite steel-concrete column

Span length of the member : L = 3.3 m
Supports : Simply supported

I.2 - Position of the member

The considered member is an intermediate member.

Spacing of the considered member :
- To the left member : D_1 = 6 m
- To the right member : D_2 = 6 m

I.3 - Cross-section

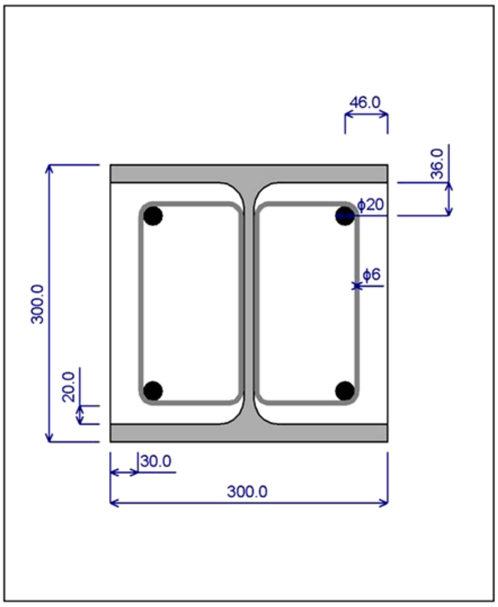


Figure 1 : Cross-section

Height of the section : h = 300 mm
Width of the section : b = 300 mm

Profile : HE 300 B

- Geometrical properties :

A	= 149.08 cm^2	m	= 117.026 kg/m
$A_{v,y}$	= 114 cm^2	$A_{v,z}$	= 47.43 cm^2
I_y	= 25165.69 cm^4	I_z	= 8562.83 cm^4

V - FIRE ULTIMATE LIMIT STATES (FIRE)

V.1 - Calculation options

No bending: Simplified calculation method
Standard fire resistance
The axial buckling load will be determined

Fire resistance class	**R120**

V.2 - Minimum and/or maximum dimensions of the cross-section

Maximum buckling length	l_θ	<= 4.050 m
Minimum cross-section depth	h	>= 230.0 mm
Maximum cross-section depth	h	<= 1100.0 mm
Minimum cross-section width	b	>= 300.0 mm
Maximum cross-section width	b	<= 500.0 mm
Minimum ratio of reinforcement	$A_s/(A_s+A_c)$	>= 1.0 %
Maximum ratio of reinforcement	$A_s/(A_s+A_c)$	<= 6.0 %

V.3 - Member resistance

- Parameters for the flanges of the profile

Temperature	$\theta_{o,t}$	= 900 °C
Coefficient	k_t	= 4.65
Section factor	A_m/V	= 13.33 m^{-1}
Average temperature	$\theta_{f,t}$	= 962 °C
Reduction factor for the yield strength of structural steel	$k_{y,\theta}$	= 0.048
Maximum stress level	$f_{ay,f,t}$	= 16.898 MPa
Reduction factor for the Young's modulus of structural steel	$k_{E,\theta}$	= 0.054
Modulus of elasticity	$E_{a,f,t}$	= 11245.500 MPa
Plastic resistance to axial compression	$N_{fi,pl,Rd,f}$	= 192.637 kN
Effective stiffness	$EI_{fi,f,z}$	= 961.490 kN m^2
Reduction coefficient for bending stiffness	$\phi_{f,\theta}$	= 1.0

- Parameters for the web of the profile

Distance	H_t	= 1250 mm
Height reduction of the web	$h_{w,fi}$	= 55.4 mm
Maximum stress level	$f_{ay,w,t}$	= 204.959 MPa
Modulus of elasticity	$E_{a,w}$	= 210000.000 MPa
Plastic resistance to axial compression	$N_{fi,pl,Rd,w}$	= 341.037 kN
Effective stiffness	$EI_{fi,w,z}$	= 3.523 kN m^2
Reduction coefficient for bending stiffness	$\phi_{w,\theta}$	= 1.0

- Parameters for the concrete

Section factor	A_m/V	= 13.33 m^{-1}
Thickness reduction	$b_{c,fi}$	= 40.7 mm
Average temperature	$\theta_{c,t}$	= 462 °C
Reduction factor	$k_{c,\theta}$	= 0.657
Maximum stress level	$f_{c,\theta}$	= 32.857 MPa
Corresponding strain	$\varepsilon_{cu,\theta}$	= 1.31 %
Secant modulus	$E_{c,sec,\theta}$	= 2509.091 MPa
Plastic resistance to axial compression	$N_{fi,pl,Rd,c}$	= 1024.655 kN

Effective stiffness	$EI_{fi,c,z}$	= 360.735 kN m²
Reduction coefficient for bending stiffness	$\phi_{c,\theta}$	= 0.8

- Parameters for the reinforcing bars

Geometrical average of the axis distances	u	= 40.7 mm
Reduction factor	$k_{y,t}$	= 0.177
Maximum stress level	$f_{s,\theta}$	= 88.678 MPa
Reduction factor	$k_{E,t}$	= 0.112
Modulus of elasticity	$E_{s,t}$	= 23624.649 MPa
Plastic resistance to axial compression	$N_{fi,pl,Rd,s}$	= 111.436 kN
Second moment of area	$I_{s,z}$	= 1362.32 cm⁴
Effective stiffness	$EI_{fi,s,z}$	= 321.843 kN m²
Reduction coefficient for bending stiffness	$\phi_{s,\theta}$	= 1.0

- Balanced summation

Plastic resistance to axial compression	$N_{fi,pl,Rd}$	= 1669.765 kN
Effective stiffness	$EI_{fi,eff,z}$	= 1575.445 kN m²
Buckling length	$l_{\theta,z}$	= 1.65 m
Euler buckling load	$N_{fi,cr,z}$	= 5711.303 kN
Non-dimensional slenderness	$\lambda_{\theta,z}$	= 0.541
Reduction coefficient for buckling	χ_z	= 0.82
Design axial buckling load	$N_{fi,z,Rd}$	= 1369.126 kN

V.4 - FIRE Verifications

- COMBINATION: FIRE01 (Auto) = G

Resistance to axial force	$\Gamma_{fi,N}$	= 0.699 < 1 =>*Satisfied*
Buckling about the minor axis	$\Gamma_{fi,b,z}$	= 0.853 < 1 =>*Satisfied*

The required fire resistance class is verified

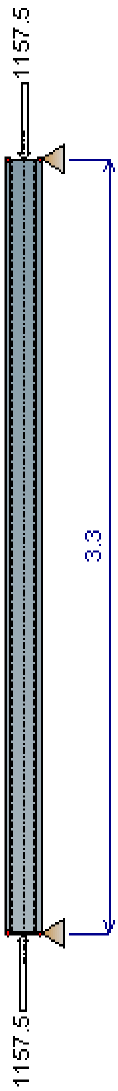
V.5 - Synthesis of verifications

Resistance to axial force (Comb. FIRE01 (Auto))	$\Gamma_{fi,N}$	= 0.699 < 1 =>*Satisfied*
Buckling about the minor axis (Comb. FIRE01 (Auto))	$\Gamma_{fi,b,z}$	= 0.853 < 1 =>*Satisfied*

1.3 Fully concreted composite column -POS 370

FIRE DEPARTMENT - 1ST FLOOR

FULLY ENCASED COMPOSITE STEEL-CONCRETE COLUMN



I - PARAMETERS

I.1 - General parameters

Element type : Fully encased composite steel-concrete column

Span length of the member : L = 3.3 m
Supports : Simply supported

I.2 - Position of the member

The considered member is an intermediate member.

Spacing of the considered member :
- To the left member : D_1 = 6 m
- To the right member : D_2 = 6 m

I.3 - Cross-section

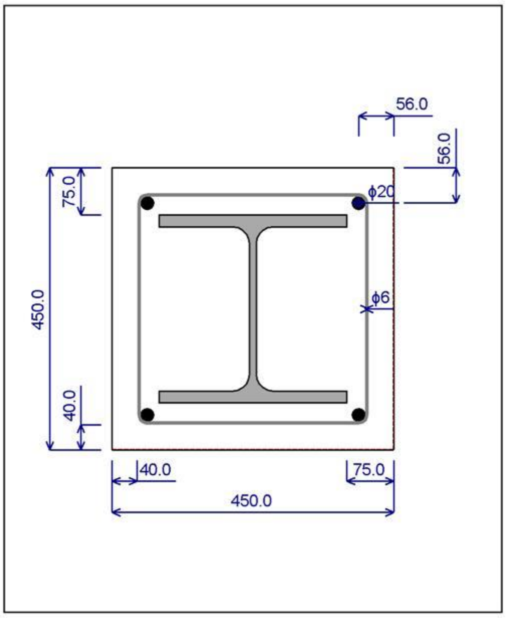


Figure 1 : Cross-section

Height of the section : h = 450 mm
Width of the section : b = 450 mm

Profile : HE 300 B

- Geometrical properties :

A	= 149.08 cm^2	m	= 117.026 kg/m
$A_{V,y}$	= 114 cm^2	$A_{V,z}$	= 47.43 cm^2
I_y	= 25165.69 cm^4	I_z	= 8562.83 cm^4

V - FIRE ULTIMATE LIMIT STATES (FIRE)

V.1 - Calculation options

Dimensions of the composite cross-section
The duration of fire resistance will be estimated

V.2 - FIRE Verifications

- COMBINATION: FIRE01 (Auto) = G

Load level	$\eta_{fi,t}$	= 0.183 < 1.0 => *Satisfied*

V.3 - Synthesis of verifications

The fire resistance class **R240** is verified
Options 1 and 2 are identical

Minimum depth and width	h_c and b_c	>= 400 mm
Cross-section depth	h_c	= 450.0 mm > 400.0 mm => *Satisfied*
Cross-section width	b_c	= 450.0 mm > 400.0 mm => *Satisfied*
Minimum concrete cover of steel section	c	>= 75 mm
Concrete cover of the steel section along major axis	c_y	= 75.0 mm = 75.0 mm => *Satisfied*
Concrete cover of the steel section along minor axis	c_z	= 75.0 mm = 75.0 mm => *Satisfied*
Minimum axis distance of reinforcing bars	u_s	>= 50 mm
Major axis distance of the reinforcing bars	u_{sy}	= 56.0 mm > 50.0 mm => *Satisfied*
Minor axis distance of the reinforcing bars	u_{sz}	= 56.0 mm > 50.0 mm => *Satisfied*

CONCLUSIONS AND RECOMMENDATIONS

In the last ten years, with the burning of a large number of high-rise buildings, the question arises of which materials are best to make them. The choice of these materials should be such that they do not rely on each other in case of fire, but help each other during the fire. One of the main problems with high-rise buildings is the survival of load-bearing structural elements under fire conditions. The structural elements are the ones that determine, to the greatest extent, the fire behavior of the construction. Protective measures with thermal insulation materials significantly improve the fire resistance of the structural elements of the building.

conclusions

Based on fire resistance calculations of unprotected and protected joint elements of a multi-story building, the following conclusions can be drawn.

1. Combined elements of steel and concrete, in high-rise buildings, have a much higher fire resistance than those of pure steel.

2. The column of the unprotected steel profile (HE 300 B) in the fire sector-1, when using the standard curve, reaches plasticization at a critical temperature of θcr=684° C, during A_t=28 min, which is much more below that described. bearing fire resistance (R15). Proper protection is necessary for such cases.

3. The partially concreted column in fire sector-1 achieves the prescribed fire resistance of R120, exposed to fire from the standard curve.

4. Fully concreted column in fire sector 1 exceeds the prescribed fire resistance of R120 for one class and is R180 for fire exposure according to the standard curve.

5. The moment load capacity for fully concreted and partially concreted fire columns is greater by 12% along the strongest axis and by 23% along the weakest axis, which also contributes to greater safety from fire.

6. The unprotected column of the steel profile (HE 200 B) in the fire sector-2, when using the standard curve, the same as in the first fire sector, is of the lower class R15. For such cuts, we must immediately take appropriate protection.

7. The fully concreted column in fire sector 2 reaches the fire resistance class R120, for fire exposure according to the standard curve. Jointed columns in the upper floors, which generally have a smaller cross-section than those in the lower floors, have a lower fire resistance than those in the lower floors and must be adequately protected.

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