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Design of HC floors for fire resistance
Conclusions from fire tests

1. Introduction

Since the appearance of hollow core floors on the market at the end of the 60-ties, certainly more than hundred fire tests have been carried out at fire laboratories over the world. The tests were done on a wide range of cross-sections together with variations in reinforcement quantities and position, etc. They are usually performed on simply supported elements, with an imposed loading corresponding to the frequent load combination. Most tests were focussing the bending moment capacity of the hollow core slabs and only a small number performed with an important shear loading.

2. Flexural resistance

2.1 Simply supported floors

The bending moment capacity of a hollow core element exposed to fire functions according to the same mechanical model as at ambient temperature: a force couple between the compression zone at the top of the member and the tensile strength of the strands. When the component is exposed to fire at its underside, the strands will heat, but the compressive zone will remain cold. Therefore the fire resistance in bending of a simply supported HC component is governed by the decrease of the strength of the prestressing reinforcement in function of the temperature.

Figure 1 gives temperature profiles within the lower part of the cross-section of hollow core units. Full-scale tests have shown that the temperature evolution within the vicinity of the prestressing reinforcement is practically independent of the slab thickness and similar to the temperature distribution in solid concrete slabs.

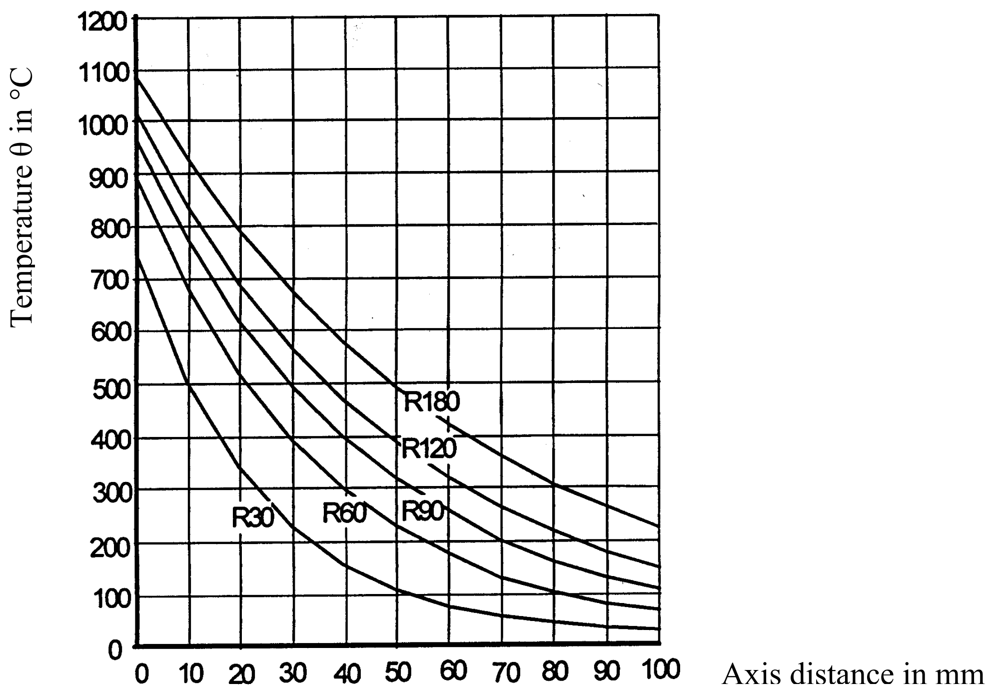
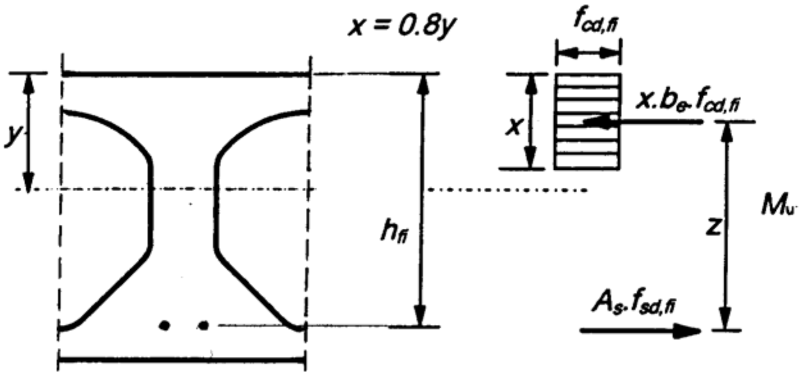


Fig.1 Temperature distribution in hollow core elements during standard ISO fire for siliceous aggregate concrete (Source EN 1192-1-2: Eurocode 2 Part 1-2: General rules – Structural fire design" [1])

The verification by calculation of the flexural capacity of hollow core floors at fire, is based on the Ultimate Limit State method. The load bearing capacity is calculated with reduced material characteristics corresponding to their temperature at a given fire exposure time. The method is extensively described in Eurocode 2 Part 1-2 [1] and in CEB Bulletin 208 [4].

The procedure for calculating the resistance of a HC cross-section in the fire situation may be carried out as follows:

(a) Determine the temperature of the prestressing strands or wires in the tension zone. The temperature of the individual strands or wires can be evaluated from the temperature profiles given in Figure 3.2.2.1, and is taken as the temperature in the centre of the bar.
(b) Determine the reduced strength of the reinforcement due to temperature according to Table 3.2.3.1.
(c) Use conventional calculation methods for the determination of the ultimate load bearing capacity. (Figure 3.2.5.2.1 shows the calculation model for the bending moment capacity).



h_{fi}	height of effective cross-section
z	lever arm between tension reinforcement and the compressive zone in the concrete
x	see conventional calculation method for ULS (usually 0.8 y)

Fig. 2. Stress distribution at ULS for a rectangular concrete cross-section

Calculation example

Slab type: HC 265
Self weight inclusive joint filling: 3.86 kN/m²
Slab length 10.60m
Imposed loading: 5.5 kN/m² (office building)
Prestressing: 2 ø ⅜ ' + 6 ø ½' strands, axis distance 50 mm (f_{pk} = 1860 N/mm²)
f_{ck} = 45 kN/mm²
$f_{cd,fi}$ = 1.00 x 45 kN/mm² (EN1992-1-2 advises to take γ_m = 1.00)

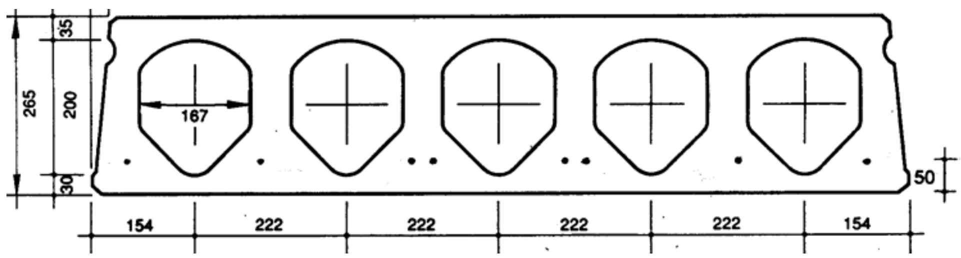


Fig. 2 Hollow core slab of 265 mm thickness

a. Determine the temperature of the individual prestressing strands

From the temperature profiles in Figure 3.2.2.1 and the material properties at elevated temperatures from Figure 3.2.3.1, the following data are obtained:

Axis distance 50 mm	**R60**	**R90**	**R120**	**R180**
θ_i	230	320	385	490
$f_{py}/0.9\ f_{pk}$	0.82	0.66	0.53	0.32
f_{py}	1372	1105	887	535

Table. 3. Residual strength prestressing strands in function of the fire duration

b. Use conventional calculation methods for the determination of the ultimate load bearing capacity with strength of the prestressing tendons as obtained in the foregoing table.

The value of z (Figure 3.2.5.2.1) is determined on the assumption that at the ultimate limit state for bending, the total compression force due to flexure on a HC unit of 1200 mm wide is equal to the total tensile force produced by the prestressing reinforcement.

$$x.b_e.f_{cd,fi} = A_s.f_{sd,fi}$$
$$z = h_{fi} - \frac{x}{2}$$

c. Calculation of $M_{Rd,\ fi}$

- At 60 minutes exposure

$$N_a = (2x51.6 + 6x93)x1372 = 907.1kN$$
$$x = \frac{907100}{1200x45} = 16.8mm$$
$$h_{fi} = 265 - 50 = 215mm$$
$$M_{Rd,fi} = 907.1x(215 - \frac{16.8}{2}) = 187.4kNm$$

- At 90 minutes exposure

$$N_a = (2x51.6 + 6x93)x1105 = 730.6kN$$
$$x = \frac{730600}{1200x45} = 13.5mm$$
$$M_{Rd,fi} = 730.6x(215 - \frac{13.5}{2}) = 152.1kNm$$

- At 120 minutes exposure

$$N_a = (2x51.6 + 6x93)x887 = 586.5kN$$
$$x = \frac{586500}{1200x45} = 10.8mm$$
$$M_{Rd,fi} = 586.5x(215 - \frac{10.8}{2}) = 122.9kNm$$

- At 180 minutes exposure

$$N_a = (2x51.6 + 6x93)x535 = 359.7kN$$
$$x = \frac{359700}{1200x45} = 6.5mm$$
$$M_{Rd,fi} = 359.7x(215 - \frac{6.5}{2}) = 76.1kNm$$

The results are represented on Figure 3.2.5.2.4

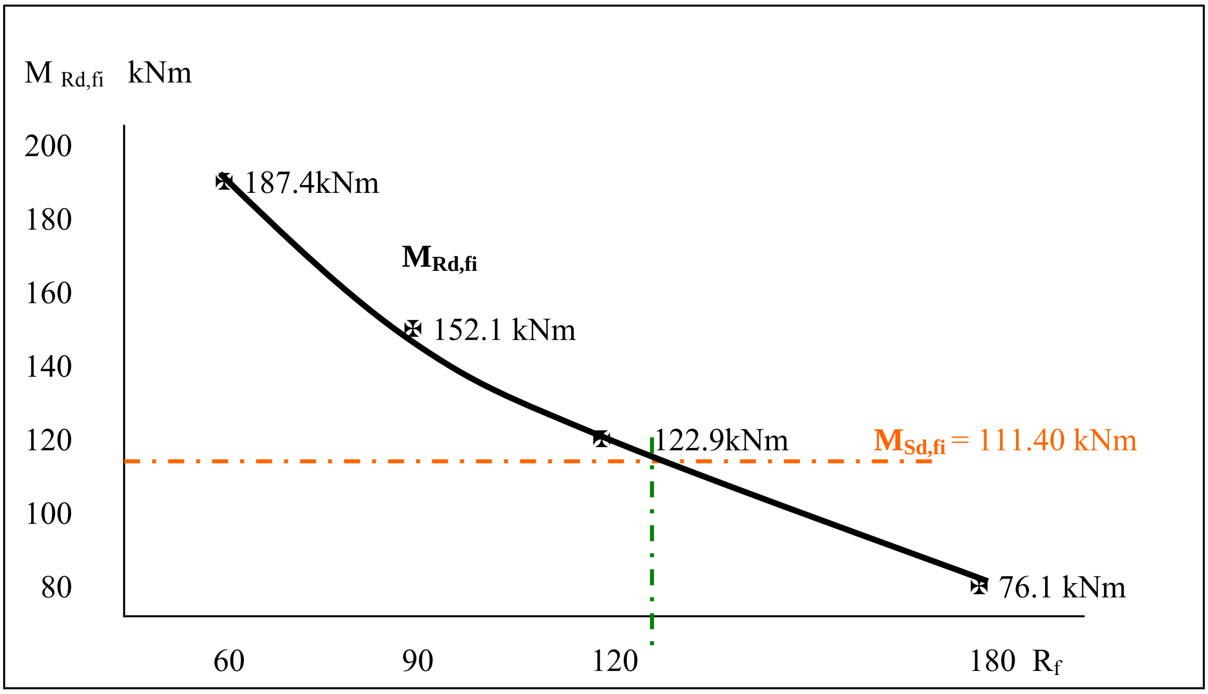


Fig. 4 Bending capacity at fire of a HC floor of 265 mm thickness

d. Calculation of $M_{Sd,fi}$

- for q_k = 5.5 kN/m² and max. span 10.60 m

$$M_{Sd,fi} = \frac{1}{8}x10.60^2(1.0x3.86 + 1.0x0.5x5.5)x1.20 = 111.40kNm$$

Conclusion: HC 265 with 50 mm axis distance and the given design load at ambient temperature, can resist a fire exposure of 120 minutes for the given span/load conditions.

2.2 Restrained support

During a fire the simply supported HC floors will undergo an additional deflection due to the imposed thermal deformation. However, in a continuous floor structure, the deflection will be hindered by the continuity reinforcement on top of the slab, and the support moment will increase according to the moment capacity of the top reinforcement. The support moment will create compressive stresses at the bottom of the floor.

Fig. 5 Thermal deformation of two simply supported hollow core floors

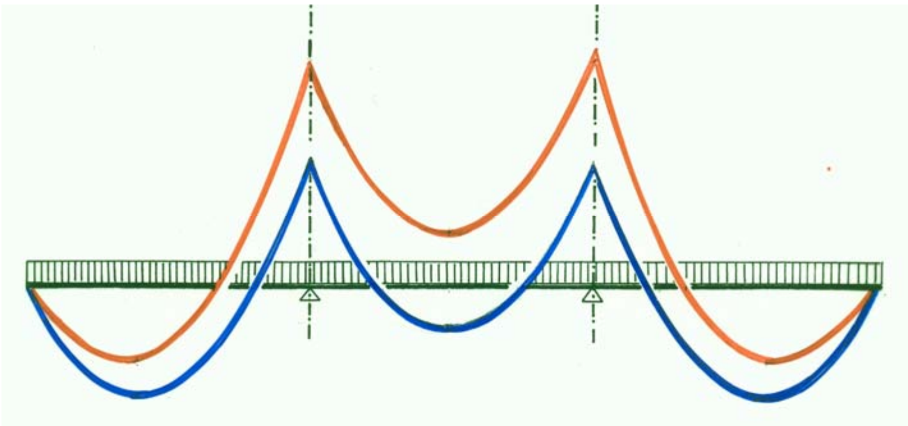


Fig. 6 Bending moment line in a continuous floor structure:
blue line at ambient temperature, red line at fire

During a fire test in Denmark on a heavily loaded cantilevering HC floor structure, premature failure occurred in the bottom flange of the HC slab at the support. The failure was due to heavy compressive forces at the bottom plate of the slabs, originated by different actions:

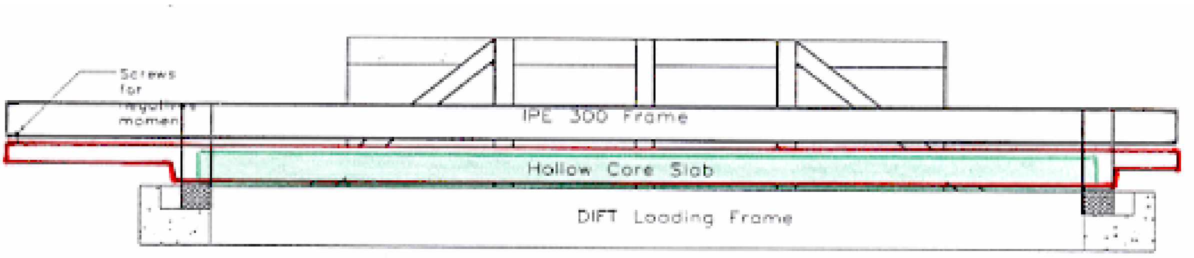


Fig. 7: Test set-up HC floor with restrained support

- The prestressing force
- Hindered longitudinal expansion
- Force couple to take up the support moment under the heavy imposed loading
- Induced thermal stresses

Design recommendations

Restrained or cantilevering hollow core floors shall be checked against compression failure of the fire exposed lower concrete flange, due to a combination of different factors:

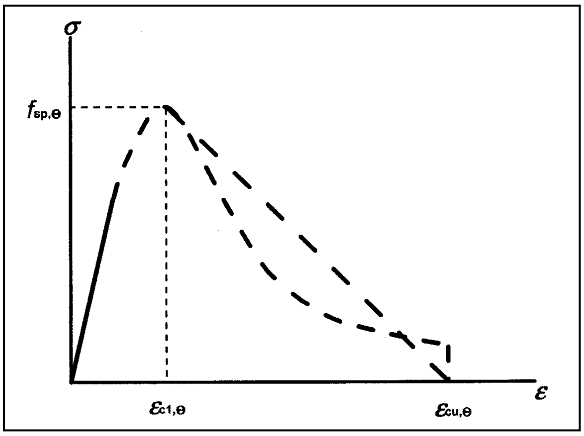
- Stresses at the underflange due to support moment $M_{neg,fi}$
- Prestressing force σ_{cp}
- Compression stresses due to temperature gradient $\sigma_{c,grad}$
- Compression stresses due to possible blocking of the thermal expansion by the surrounding structure $\sigma_{dil,fi}$

$$\sigma_{Ed,fi} = \sigma_{c,Mneg,fi} + \sigma_{cp} + \sigma_{c,grad} + \sigma_{dil,fi}$$

$$\sigma_{Ed,fi} \leq f_{cd,fi}$$

and $\varepsilon < \varepsilon_{c1,\theta} \cong 0{,}025$

The stress-strain diagram of concrete at elevated temperature θ is shown in Figure 7, with a descending branch when $\varepsilon \geq \varepsilon_{c1,\theta}$ (ref. EN1992-1-2 § 3.2.2.1)



for $\varepsilon \leq \varepsilon_{c1,\theta}$

$$\sigma_\theta = \frac{3\varepsilon . f_{c,\theta}}{\varepsilon_{c1,\theta}\left[2+\left(\frac{\varepsilon}{\varepsilon_{c1,\theta}}\right)^3\right]}$$

Fig. 8 Mathematical model for stress-strain relationships
of concrete under compression at elevated temperature

A simplified procedure for the verification of a hollow-core floor slab with restraint at support is given hereafter (Figure 8)

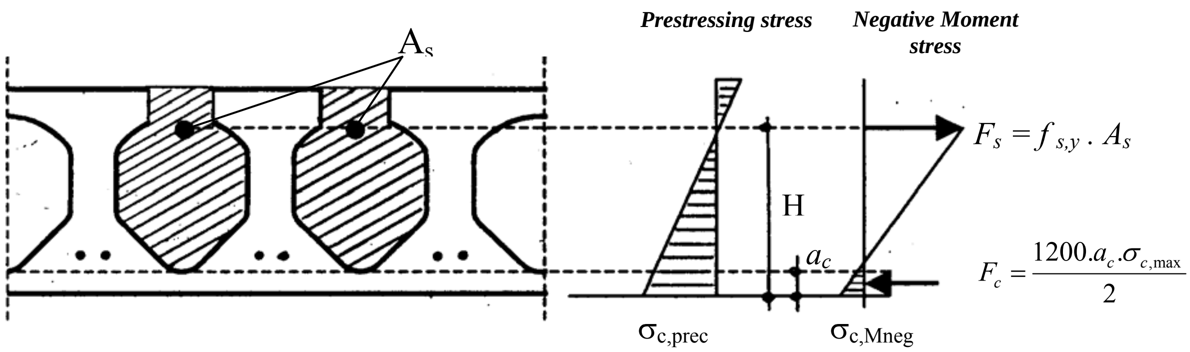


Fig. 9 Compressive stress of hollow core lower flange due to M_{neg} and prestressing

The equilibrium condition between the tensile force due to the support moment and de compression force at the bottom flange is expressed by:

$$F_s = F_c$$

$$F_c = \frac{a_c.1200.\sigma_{c,M_{neg}}}{2}$$

$$f_{sy}.A_s = \frac{1200.a_c.\sigma_{c,M_{neg}}}{2}$$

where As: area of negative moment reinforcement
a_c: average concrete thickness lower flange

The average value of compressive stress under negative moment is then:

$$\sigma_{c,M_{neg}} = \frac{\sigma_{c,\max,M_{neg}}}{2} = \frac{\mathrm{f_{sy}} \cdot \mathrm{A_s}}{1200 \cdot \mathrm{a}}$$

The prestressing stresses in the underflange are calculated in the classical way and the resulting stresses after 90 minutes fire exposure are calculated as follows:

$\sigma_{cp} = x$ N/mm² (x symbolizes the stress in the concrete of the underflange at ambient temperature)
strand temperature at 90 minutes fire for 50 mm axis distance: 320 °C
$f_{p,y} / 0.9\ f_{pk} = 0.66$ (Table 3.3 in EN 1992-1-2)
$\sigma_{cp,fi} = 0.66\ x$

In addition to the loss of prestress due to the decrease of the material strength, there will be a loss due to strand slippage and additional thermal relaxation. To simplify the calculations, we assume that the loss of prestress is compensated by the compression stresses due to possible blocking of the thermal expansion by the surrounding structure $\sigma_{dil,fi}$

$\sigma_{prest,\theta} + \sigma_{dil,\theta} \cong \sigma_{prest}$ at ambient temperature

The calculation of the compression stresses due to temperature gradient $\sigma_{c,grad}$ is very complex and should be done by Finite Element Method. The calculations which were carried out in the University of Liège (Belgium) show a value in the order of 10 N/mm². The actual value at fire will certainly be lower because of the load induced thermal strain at the bottom of the slab. We propose to take a value of 8 N/mm².

The max compressive stress in the bottom flange under fire condition can be expressed as follows:

$\sigma_{d,\theta} = \sigma_{c,Mneg} + \sigma_{prest,\theta} + \sigma_{c,grad}$

$$= \frac{\mathrm{f_{sy}} \cdot \mathrm{A_s}}{1200 \cdot \mathrm{a}} + \sigma_{prest,max} + 8 \text{ N/mm}^2$$

$\sigma_{d,\theta} \leq f_{cd,\theta}$ (average strength in the underflange after θ minutes fire exposure)

The calculation of the concrete strain following the equation in Figure 7 can be done with some simplifications:

$(\varepsilon/\varepsilon_{c1,\theta})^3 << 2$ and may be deleted

$\varepsilon_{c1,\theta} = 0{,}025$ corresponds with a temperature θ of the lower flange higher than 560 ÷ 600 °C (ref. EN1992-1-2, Table 3.1)

$$\varepsilon_c = \frac{\sigma_{c,M_{neg}} x 2 x 0.025}{3 f_{c,\theta}}$$

(to be << 0,025)

and the verification of max steel strain

$$\varepsilon_S = \frac{\varepsilon_c\,(H - a_c)}{a_c}$$

(to be << 0,02)

Hollow core floors, subject to a usual loading of 4.0 to 5.0 kN/m^2 and designed with full restraint or continuity at support, normally do not present critical compressive stresses at fire, when the depth is such that $L/H \leq 35$ and when the axis distance of the prestressing steel is in accordance with the usual values corresponding to the required fire resistance.

When the floor loading is larger or more slender slabs are used, it is advisable to design the floor according to the simple bearing scheme or with partial restrainment at the support.

3. Shear resistance

Tests and calculations have shown that the thermal gradients over the cross-section of structural concrete elements exposed to fire may induce differential internal stresses in the cross-section due to the incompatibility between the linear deformation of the cross-section and the non-linear thermal expansion.

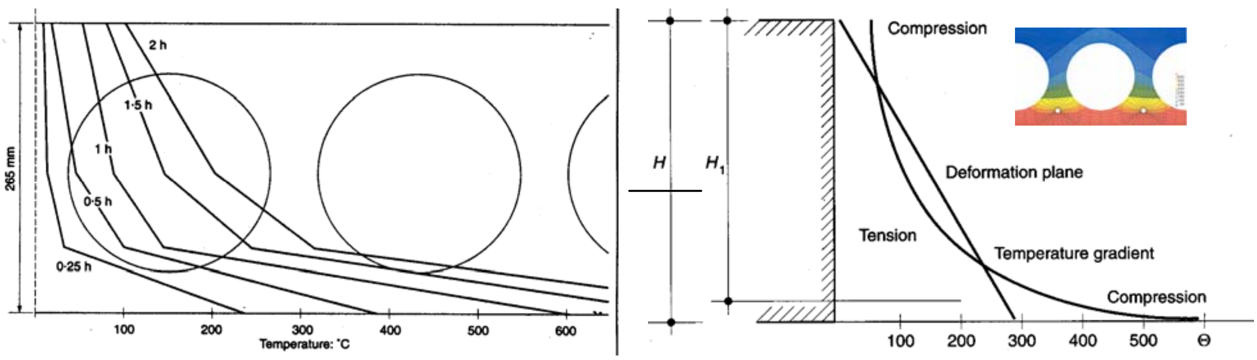


Fig. 10: Temperature profiles in a hollow core slab and induced stresses in the cross-section.

Measurements during ISO fire tests on hollow core elements show that the temperature gradient has a strong curvature. In Figure 10(a) the registered temperatures over the cross-section of a 265 mm thick slab are given respectively after 30,60,90 and 120 minutes of exposure. It is stated that the curvature of the temperature profile is more pronounced at 30 minutes and decreases gradually for larger fire exposure times.

Due to the thermal expansion at the intrados, the slab will bend downwards. The resulting deformation of the cross-section is more or less linear. However, due to the fact that the temperature distribution over the cross-section is not linear, (see Figure 10(a), compressive stresses will manifest at the top and the bottom part of the concrete section and tensile stresses in the middle. During the study at the University of Liege, calculations carried out on the shear resistance of hollow core slabs during fire confirm this.

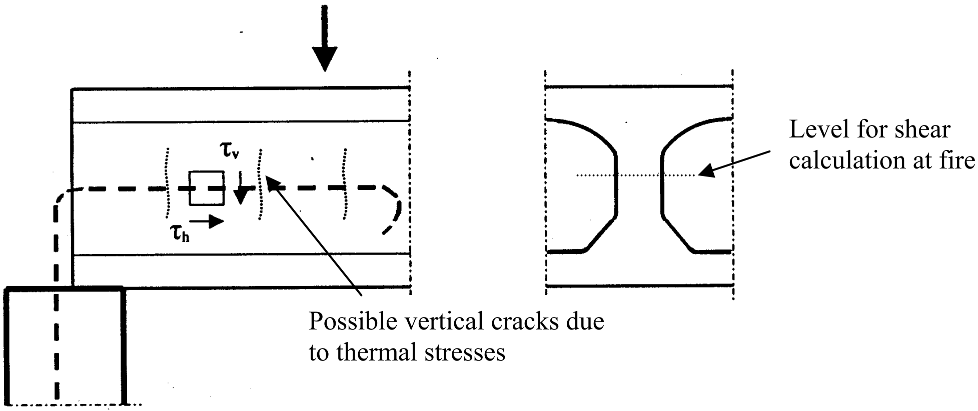


Fig. 11 Longitudinal section of HC under shear loading

4. Connections

Experience during fire tests in laboratories has shown that the structural integrity and diaphragm capacity of hollow core floors through correctly designed connections, which as a matter of fact constitutes the basis for the stability of the floor at ambient temperature, are also essential in the fire situation.

The thermal stresses induced in the cross-section, as explained above may lead to vertical cracks in the webs of the hollow core units, and decease the shear capacity. To counteract this effect, these cracks should remain closed. In principle, cracked concrete sections can take up shear as good as non-cracked sections on condition that the cracks are not opening. The crack borders are rough and shear forces can be transmitted y shear friction and aggregate interlocking (Fig.12). The figure illustrates the generation of transverse forces due to the wedging effect. In hollow core floors, this transversal! force is taken up y the transversal! tie reinforcement at the support.

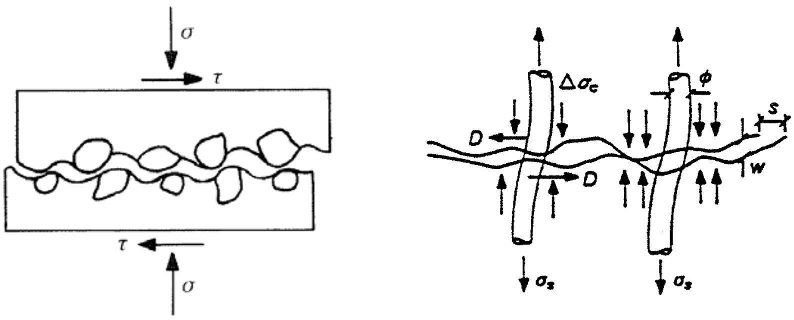


Fig. 12: Transfer of shear forces through aggregate interlocking

The decrease of the concrete strength at higher temperatures is hardly playing a role in the shear capacity of hollow core floors. Such temperatures appear only at the bottom part of the concrete section, and much less in the centre. From the foregoing, it appears that at the design stage, provisions are to be taken to realize the necessary connections between the units in order to obtain an effective force transfer through cracked concrete sections. The fact that shear failures have not been observed in real fires shows that there exist enough possibilities to realize the connections between the units. (This has also been proven in numerous fire tests in different laboratories).

The possible design provisions are explained hereafter.

- *Reinforcing bars in cast open cores*

The reinforcing bars are first designed to connect the floor units at the support. The reinforcing bars are placed centrally in the section, where the thermal stresses appear, in order to keep the cracks closed. The effectiveness of such reinforcement in the preservation of the shear capacity of the units at fire has been proven in many full-scale tests

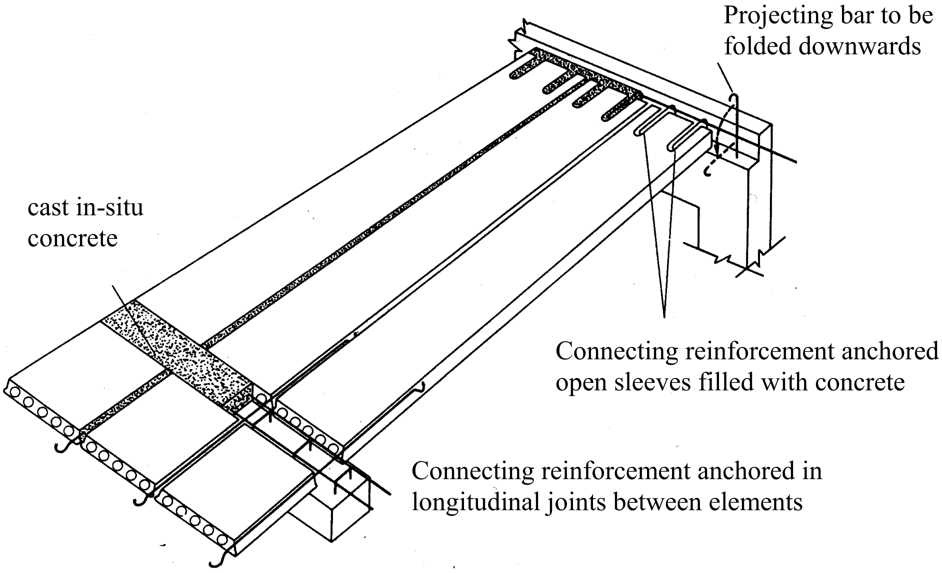


Fig. 13: Connections between hollow core floor and supporting structure

- *Reinforcing bars in the longitudinal joints*

This is a variant solution of the above. Good anchorage of the bars in the longitudinal joint between units is required. This presupposes that the joints remain closed, which can be achieved by the tying effect of peripheral tie reinforcement. The real function of the latter is to ensure the diaphragm action of the floor and the lateral distribution of concentrated loading, even through cracked joints by shear friction. The anchorage capacity of steel bars in cracked longitudinal joints between hollow core units has been extensively studied [Engstrom (1992)]. It is recommended to limit the diameter of the bars to 12 mm maximum and to provide a larger anchorage length than normally needed, e.g. 1500 mm for a bar of 12 mm,. When the above conditions are met, the reinforcing bars in the joints ensure the interlocking effect of the possible cracked concrete section, and hence the shear capacity of the units at fire. Also this case has been proven repeatedly in many 150 fire tests.

- *Peripheral ties*

The peripheral ties contribute to preserving the shear capacity of the units when exposed to fire by obstructing, directly and indirectly, the expansion of the floor through the rigidity of the tie beam itself, and through the coherence between neighbouring floor units.

When fire occurs in the central part of a large floor, the thermal expansion of the units will be practically completely blocked by the rigidity of the surrounding floor. The blocking will mobilize important compressive forces in the fire exposed floor units. (This has been observed in real fires, where large spalling sometimes occurs under the high compressive forces.) In such cases, the central part of the cross-section will not be cracked because of the differential thermal stresses, but the whole section will be subjected to compression. The shear capacity will therefore be unaffected.

Connections with steel beams

In case of partially encased steel profiles, for example in slim floor structures, the temperature rise in the steel profile will be slower than in non-encased unprotected profiles, due to the effect of the thermal conductivity of the surrounding concrete. However, it is recommended to protect the exposed steel flange by a fire insulating material (Fig. 14). The reason lies in the fact that in case of hollow core elements, the floor support should always be under the floor slab, since an indirect support via links or stirrups will introduce tensile stresses in the webs of the hollow core slab, which should be avoided as much as possible.

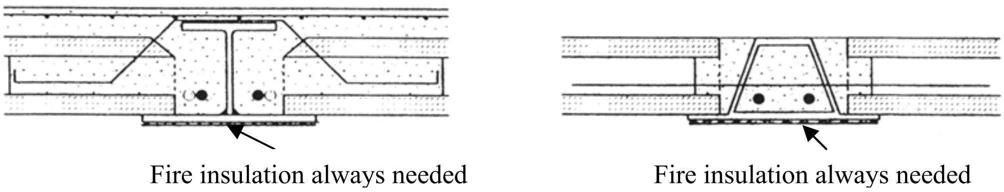


Fig. 14: Examples of slim floor structures