

PUBLICATION N° 32

Calculation of the Fire Resistance of
Composite Concrete Slabs with
Profiled Steel Sheet
Exposed to the Standard Fire.



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The calculation method is based on the present state of knowledge. Because this knowledge is still incomplete, conservative solutions have been adopted.

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0. FOREWORD

This Technical Note is to be considered as a supplement to the European Recommendations for the Fire Safety of Steel Structures also prepared by Technical Committee 3 of the European Convention for Constructional Steelwork. The ECCS-Recommendations do not cover composite structures.

The use of composite steel concrete building components is becoming increasingly important today. By structurally combining steel and concrete in one component, the advantages of each material can fully be utilized. Compared with more traditional structural elements (steel or concrete) the composite steel concrete system offers the following benefits:

- reduced overall dimensions
- easy prefabrication and erection
- good resistance to instability
- high thermal inertia.

These benefits are certainly achieved for the composite concrete slab with profiled steel sheet, which is the type of composite system most frequently found in buildings.

The fire resistance of composite slabs is significantly higher than that of unprotected steelwork even if no additional fire safety precautions are taken. If necessary, the fire resistance can be increased to practically any desired level by simple and reliable means.

Until recently the structural fire engineering design of composite slabs could only be based on fire resistance tests. Such a procedure is time consuming and expensive, and sometimes gives rise to anomalies due to variations in test results.

For these reasons it is felt within the ECCS-Technical Committee 3 that there is a strong need for a calculation method by which the fire resistance of composite slabs can be determined, and this should lead to more uniform levels of safety. Furthermore it leads to a more simple and systematic design procedure, thus stimulating the use of composite slabs.

Editing and other preparatory work has been made mainly at the Institute TNO for Building materials and Building Structures, Delft, The Netherlands. Final editing was undertaken by Fire Research Station, United Kingdom.

Delft, December 1983
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Chairman of the ECCS-
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1. SCOPE

These calculation rules apply to the practical design of concrete floor slabs acting compositely with profiled steel sheet when exposed to the standard fire, and are derived for normal weight concrete. The performance criteria are based on a simplification of the more general fire resistance criteria as specified in ISO 834.

For composite slabs without additional means of fire protection, the Technical Note enables the designer to avoid undertaking an explicit analysis of the behaviour under fire conditions, when the required fire resistance is not over 30 minutes. Analytical methods are presented, designed to meet given fire resistance requirements, when additional reinforcement is applied.

This Technical Note reflects the present state of knowledge. Where appropriate, (conservative) assumptions have been made, rather than try to give a detailed description of the phenomena involved.

2. CRITERIA FOR FIRE RESISTANCE

2.1 Introduction

Fire resistance is determined under standard fire conditions, characterized by the so-called standard gas-temperature-time curve. This curve is shown in Fig. 1.

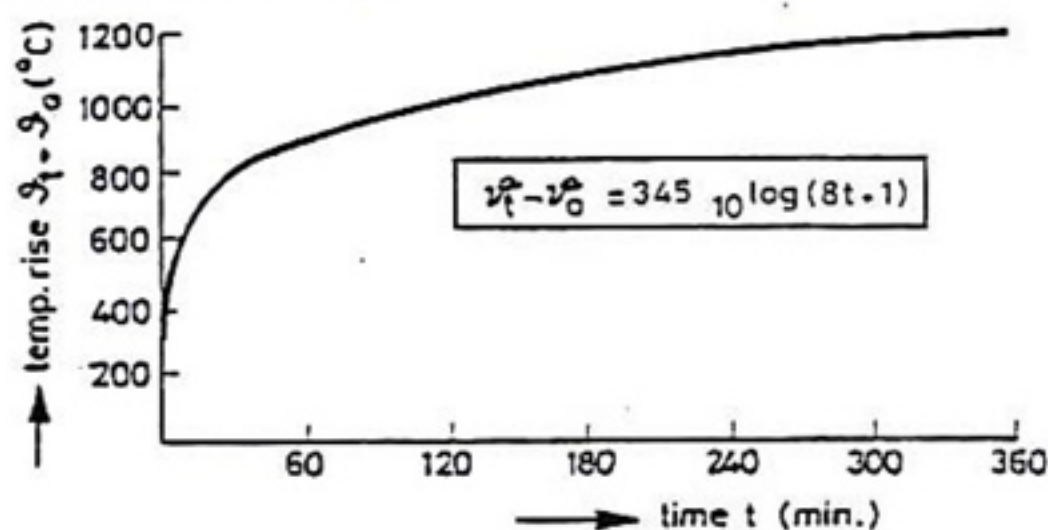


Fig. 1. The standard fire curve.

Composite steel-concrete slabs have both a load bearing and a separating function, and the following criteria for fire resistance shall therefore be taken into account: [1].

- | | |
|---------------------------|--|
| (1) LOAD BEARING CAPACITY | : Resistance to collapse or excessive deflection under structural loading |
| (2) INSULATION | : Limitation of the temperature increase on the unexposed side of the slab |
| (3) INTEGRITY | : Ability of the slab to resist penetration of flames or hot gases through the formation of cracks and openings. |

The time taken to fail any of these 3 criteria is taken as the fire rating of the slab, even though failure under other criteria may not occur until much later.

It is common practice to determine the fire resistance by means of standard fire resistance tests. During such tests, the test specimen is exposed to the standard fire on the underside while loaded with a load calculated to produce the normal maximum working stresses in the floor construction.

Similar assumptions are adopted when using the analytical approach. In accordance with generally accepted practice, fire exposure on the upper side of the slab is excluded, being less critical than fire exposure from below.

2.2 Load bearing capacity

The criterion of load bearing capacity requires that the slab shall not cease to perform the load bearing function for which it was constructed, viz.:

$$q_{u\psi} > q \quad \dots(1)$$

where: $q_{u\psi}$ = load bearing capacity of the slab after the required period of standard fire exposure
 q = loading on the slab during a fire.

During tests, collapse of the slab is avoided so as to prevent damage to the furnace and other apparatus. This is achieved by limiting an excessive deflection. A commonly used criterion is that a fire exposed slab is assumed to fail if one or both of the following conditions is met [2]:

$$\begin{aligned} \frac{d\delta}{dt} &> \frac{L^2}{9000 H} \\ \delta &> \frac{L^2}{800 H} \quad *) \end{aligned} \quad \dots(2)$$

*) For many practical applications this criterion is simplified to:

$$\frac{\delta}{L} > \frac{1}{30}$$

where

t - time of fire exposure	[min]
δ - deflection of the slab	[mm]
L - span of the slab	[mm]
H - height of the slab	[mm]

From a functional point of view, the failure condition as presented by equation (1) is preferred. Since this Technical Note deals with an analytical approach rather than with experiments, this load bearing criterion is used here.

2.3 Insulation

In order to fulfill the insulation criterion, the temperature rise of the unexposed side should not exceed 180°C at any point and the average should not exceed 140°C . This criterion, given in [1], is applied in most national standards.

Because of the profiled shape of the slab, care must be taken when checking that the insulation criterion is satisfied. See Fig. 2.

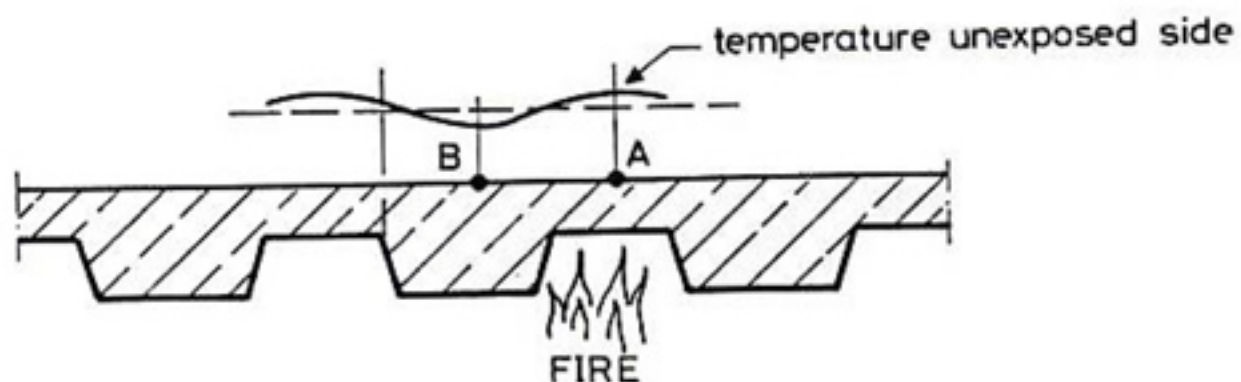


Fig. 2. Theoretical temperature distribution over the unexposed side of a concrete slab with profiled steel sheet.

Theoretically the temperature at point A will be higher than at point B. Tests show however that in practical cases the temperature differences are small (Fig. 3).

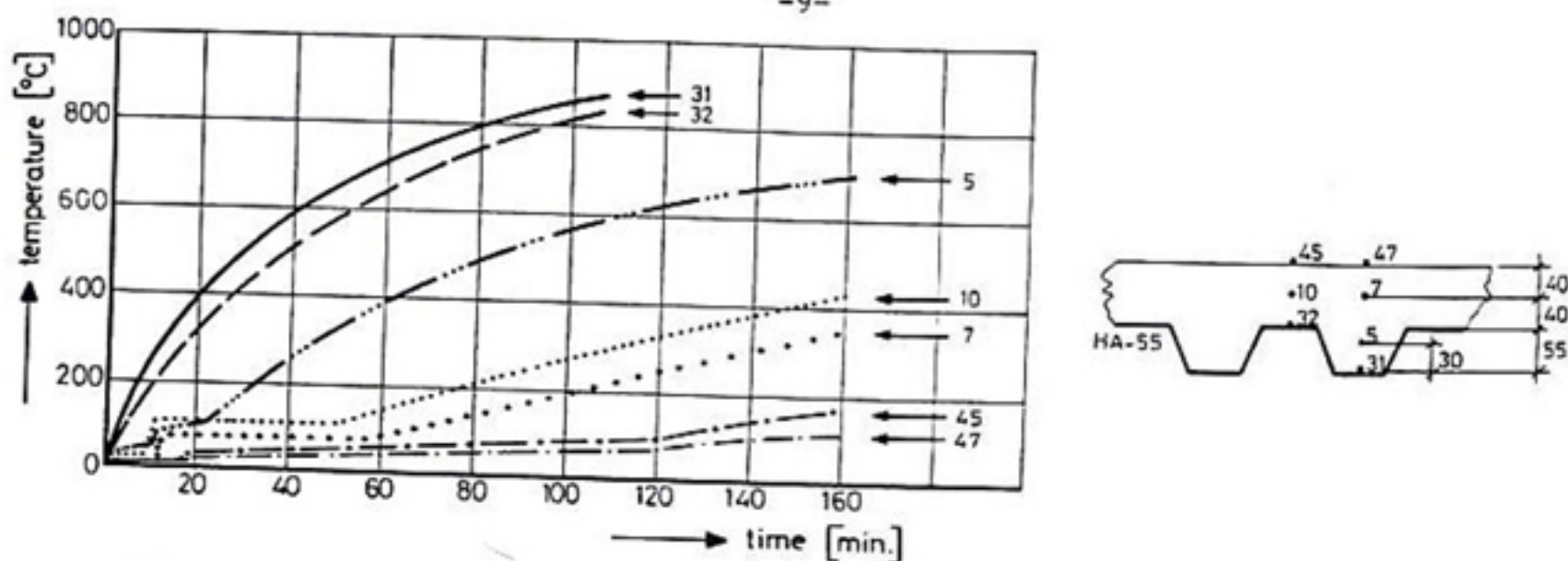


Fig. 3. Measured temperature distribution in a concrete slab with profiled steel sheet under standard fire exposure.

In theory it is also possible that passage of heat through joints may result in a non uniform temperature distribution at the unexposed side. However, composite steel concrete slabs are normally manufactured in situ and this complication does not arise.

In this Technical Note therefore a uniform temperature distribution at the unexposed side is assumed. A temperature increase of 140°C at this side is taken as the limiting insulation criterion.

2.4 Integrity

Integrity is a measure of the ability of the construction to resist the passage of flames and hot gases through cracks. It is determined by the ignition of cotton wool pads held close to the unexposed side of the slab. The various national standards are based on this principle although minor differences do exist.

For composite steel concrete floors the integrity criterion is not difficult to fulfill. The main reason is that, as mentioned before, the floor slab is cast in situ. This means that joints are adequately sealed. Any cracks which may occur in the concrete during fire exposure are unimportant because the steel sheet will prevent penetration by the flames and hot gases.

Therefore it is assumed in this Technical Note that if the insulation criterion is fulfilled, then the integrity criterion is also fulfilled.

3. BEHAVIOUR OF FIRE EXPOSED COMPOSITE CONCRETE SLABS WITH PROFILED STEEL SHEET

The steel sheet of a composite slab is normally designed to transmit the tensile stresses due to positive bending moments when at ambient temperature.

When exposed to fire, the temperature of the steel sheet will increase, and consequently the mechanical properties such as yield stress and elastic modulus will decrease. At a certain temperature, which is dependent on the load level and the statical system, the steel sheet is no longer able to transmit the applied tensile force and as a result the slab fails the criterion for load bearing capacity. Such a failure may, dependent on the thickness of the slab, be preceded by failure under the insulation criterion.

Table 1 gives results of fire resistance tests on composite concrete slabs with profiled steel sheet, conducted in various European fire test laboratories. No additional means of fire protection was present. The tests cover a practical range of application. It is seen that in all cases the fire resistance is governed by the criterion for load bearing capacity and is over 30 minutes. The considerable scatter is caused by differences in design assumptions at room temperature and in the support conditions.

Some designs assume that working stresses in the concrete do not exceed the tensile strength. In such cases a fire resistance of about 60 minutes can be obtained. Reference is made to the results of test 2 and 3 of Table 1. See also [3].

With increasing midspan deflection, horizontal displacements of the slab will tend to occur. If these displacements are prevented, for example when the slab is attached effectively to the building framework or when the slab is part of a continuous floor in which only one span is exposed to fire, axial restraint (so called "membrane action") is introduced. This may lead to a significant increase in fire resistance, provided the building framework can adequately resist the horizontal membrane forces.

ECCS Publication - Fire Resistance of Composite Slabs

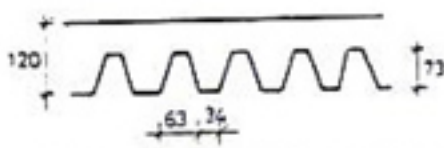
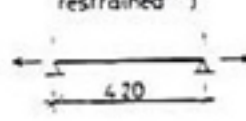
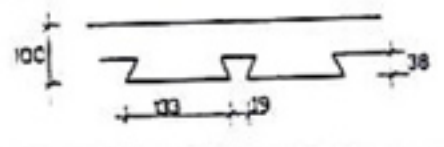
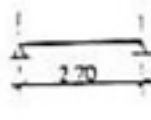
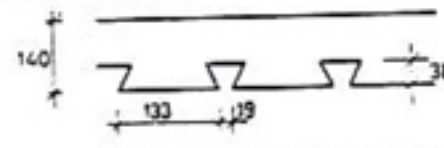
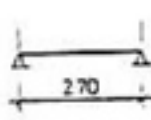
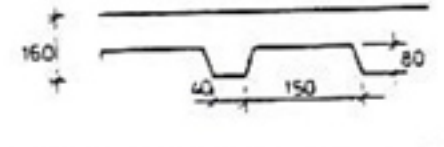
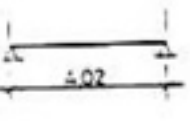
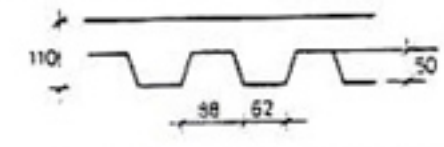
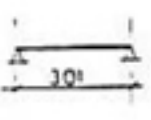
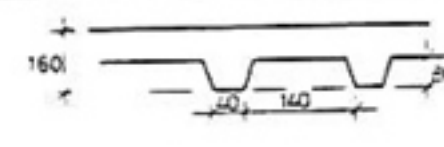
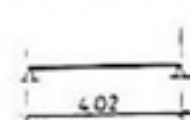
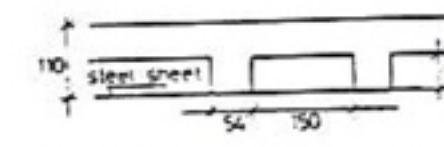
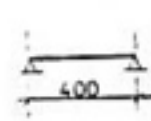
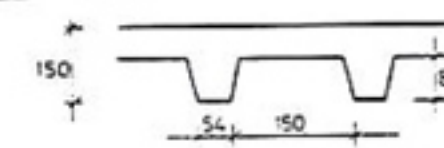
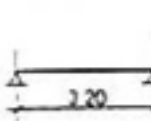
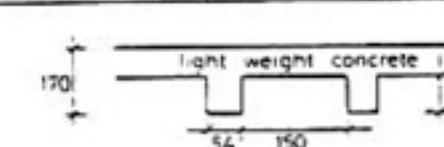
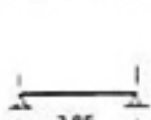
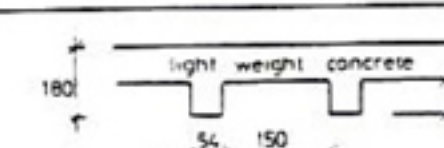
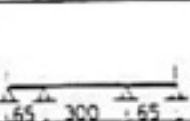
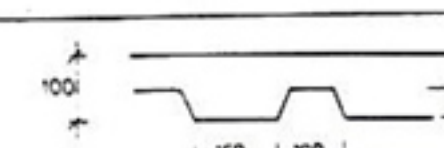
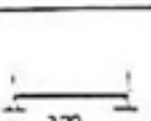
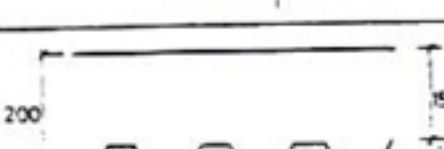
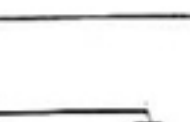
Laboratory Test + report nr. (country)	Cross section 1) (dimensions in mm)	Statistical system (span in m)	Live load [N/m ²]	Fire resistance criterion regarding load bearing capacity	Fire resistance time [min]
1 TNO BV-73-74 (The Netherlands)			2700	$\frac{\delta}{L} > \frac{L^*}{900 H}$	40
2 EMPA 66356/1 (Switzerland)			4800	$\frac{\delta}{L} > \frac{1}{20}$	55
3 EMPA 66356/2 (Switzerland)			10100	$\frac{\delta}{L} > \frac{1}{20}$	69
4 CTICM 74V58/T41 (France)			2500	$\frac{\delta}{L} > \frac{1}{20}$	33
5 CTICM 74V64/T47 (France)			2500	$\frac{\delta}{L} > \frac{1}{30}$	35
6 CTICM 74V59/T42 (France)			2500	$\frac{\delta}{L} > \frac{1}{30}$	37
7 CSTB 66.2385D (France)			3830	$\frac{\delta}{L} > \frac{1}{30}$	38
8 CSTB 66.2478A (France)			3800	$\frac{\delta}{L} > \frac{1}{30}$	44
9 CSTB 70.4018 (France)			6880	$\frac{\delta}{L} > \frac{1}{30}$	38
10 CSTB 69.3595 (France)			10100	$\frac{\delta}{L} > \frac{1}{30}$	40
11 EMPA 11009 (Switzerland)			3300	$\frac{\delta}{L} > \frac{1}{30}$	40
12 VTI/PAL A8378 (Finland)			5500	$\frac{\delta}{L} > \frac{1}{30}$	43

Table 1 Fire resistance of composite concrete slabs with
profiled steel sheet without additional means of fire
protection.

*) 1) dense concrete unless otherwise stated

2) steel sheet at the supports fixed with rivets to a steel frame

The possible effect of axial restraint on the fire resistance can only be quantified when enough information is available on the rigidity of the boundary structural components. Such information is generally not available. Therefore the beneficial effect of membrane action on the fire resistance of composite concrete steel slabs is ignored in this Technical Note.

Accordingly, in only one of the tests (Test 1) reviewed in Table 1, axial restraint was introduced intentionally. All tests, except for Test 1 and 10, were conducted on simply supported slabs. Such a static system obviously constitutes relatively unfavourable conditions, since no beneficial moment redistribution or catenary force can occur as it often does in practice.

Table 1 therefore suggests the following general practical design rule:

" The fire resistance of composite concrete slabs with profiled steel sheet without additional means of fire protection is at least 30 minutes when assessed under the criterion for load bearing capacity".

The application of this rule should obviously be restricted to those cases in which the design at room temperature is based on an approved method. The European Recommendations for the Design of Composite Floors with Profiled Steel Sheet [4], could be taken as a reference. It is also necessary to check that the insulation criterion is fulfilled. The simple verification rules presented in chapter 5 can be used for this purpose.

Requirements for fire resistance, if any, are normally expressed as multiples of 30 minutes (viz. 30, 60, 90, ... minutes). As a direct consequence of the above design rule, an explicit analysis of the load bearing capacity of fire exposed composite concrete steel slabs is only necessary for requirements over 30 minutes. Additional means of fire protection may then be necessary.

4. ADDITIONAL MEANS OF FIRE PROTECTION

The following means of additional fire protection can be distinguished:

- additional reinforcement
- insulating coatings
- suspended ceilings

Additional reinforcement

In a composite slab some steel reinforcement (say: $100 \text{ mm}^2/\text{m}$ width of slab) is normally included to control shrinkage and creep of the concrete. This reinforcement may be placed directly upon the steel sheet. When no coating or suspended ceiling is used, the steel sheet is directly exposed to fire. As a result, the temperature increase in the steel sheet and in the reinforcement can be expected to be approximately the same. Consequently, the beneficial effect of the shrinkage reinforcement on the fire resistance may only be marginal. However, additional reinforcement placed in the centre of the ribs may significantly contribute to the fire resistance. The same holds for the top reinforcement over intermediate supports of continuous slabs. Fig. 4 illustrates the various possibilities.

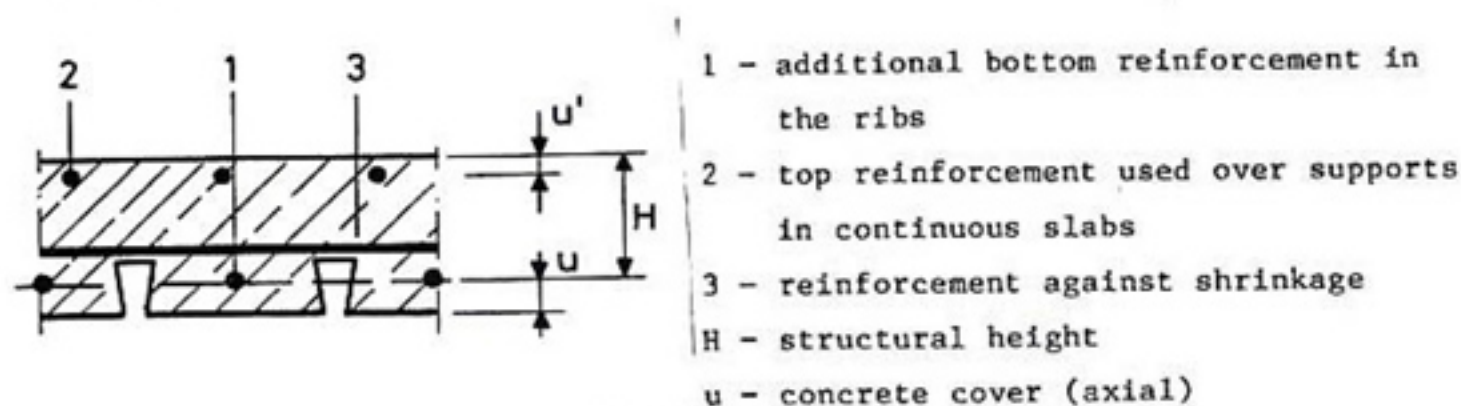


Fig. 4. Reinforcement in a composite concrete slab with profiled steel sheet.

With the additional reinforcement, the fire exposed slab will behave as a reinforced concrete element and its fire resistance will depend upon the amount of reinforcement and the position of the reinforcement bars in the slab, i.e. upon structural height and concrete cover. The minimum concrete cover shall ensure that the temperature of the reinforcement does not reach the critical value, say 500°C , too early.

In a similar way, analytical design rules can be formulated assuming the slab behaves as a reinforced concrete floor. The additional reinforcement only affects the load bearing capacity. The insulation performance depends only on the average height of the slab and must be checked separately, as shown in chapter 5.

Coatings

Insulating coatings may be necessary when extremely high fire resistance ratings are required and/or when deflections have to be severely limited under fire exposure. Sprayed coatings (e.g. based on mineral fibres or vermiculite) are directly applied to the surface of the steel sheet. In order to achieve good adhesion, the steel surface should be properly cleaned to remove dirt and grease. Fire protecting boards can also be used (e.g. based on vermiculite, gypsum, fibre). These are then directly adhered or mechanically fixed to the ribs of the steel sheet. As with sprayed coatings, a thorough cleaning is necessary to ensure good adhesion. Special attention should be paid to the type of adhesive used and to adequate connection of boards under fire conditions. These aspects must be verified by experimental evidence. Only a relatively small thickness of insulation is necessary to achieve a considerable fire resistance and this is illustrated by the tentative values presented in Table 2 [5]. Nevertheless, the application of fire protecting coating will involve considerable extra cost.

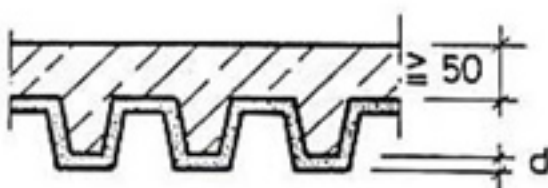
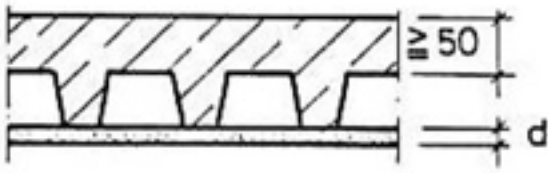
Type of coating	Fire resistance (min.)		
	90	120	
- sprayed mineral fibre	12	14	
- vermiculite boards			

Table 2 Fire resistance of composite concrete slabs with profiled steel sheet as a function of the thickness of the applied coating.

Suspended ceilings

A suspended ceiling functions as a heat shield for the structural components above and thus can contribute to the fire resistance of the floor assembly. In the cavity over the fire protecting ceiling a time-temperature curve which is less severe than the standard fire curve, can be taken into account, subject to the condition that there is only a limited amount of combustible material in the (unventilated) cavity. The extent to which the standard fire curve is thus reduced, will depend on the quality of the ceiling and on the floor above it.

The ceiling will not only protect the floor slab, but also the beams which support the slab.

Another advantage of applying suspended ceilings as additional means of fire protection is, that its function can be combined with a decorative and/or a physical function such as thermal and acoustic insulation.

Moreover the cavity above the ceiling may be used for installations of services.

The behaviour of a suspended ceiling during fire, however, is critical since it depends very much on good detailing, workmanship and maintenance. The connection of the ceiling to the floor above, for example, should function effectively during a fire. Another factor to consider is the effect of thermal expansion. The ceiling systems envisaged here will often consist of boards supported in a metal grid. If expansion of this frame is prevented, parts of it may buckle, with the result that the ceiling boards are displaced and the protective function may be lost. It is possible to detail the grid in such a way that free expansion can take place (i.e. with sliding joints or expansion cut-outs).

Both aspects have to be verified experimentally and fire resistance tests are therefore essential.

Summarizing, it can be concluded that additional steel reinforcement is a simple, reliable and economic device to increase the fire resistance of composite concrete steel slabs. Moreover, the assessment of the effect of additional reinforcement is open to a theoretical analysis.

The reliability of the two other means of fire protection, i.e. coatings and suspended ceilings, is much more critical, being highly dependent on factors such as detailing and workmanship. Experimental verification is deemed to be necessary in these cases.

Additional means of fire protection may be necessary when a fire resistance of more than 30 minutes is needed. It follows that additional reinforcement is then preferred. Such a solution requires a check on both the insulation and load bearing capacity criteria. Rules for analytical verification of these criteria are presented in chapter 5 and 6 respectively.

5. CALCULATION RULES FOR MINIMUM SLAB THICKNESS

The insulation criterion of fire resistance is fulfilled if the average temperature increase at the unexposed side of the slab exceeds 140 °C. See 2.3. This requires a sufficient effective thickness of slab, which will depend on the period of fire resistance required. Based on experimental data, the following rules for effective slab thickness can be given [6].

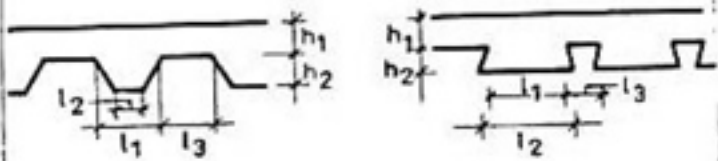
Required fire resistance Min.	Minium effective thickness h_e mm	Equation for effective thickness h_e	Restrictions
30 60 90 120 180 240	60 70 80 100 130 150	$h_e = h_1 + \frac{h_2}{2} \cdot \frac{l_1 + l_2}{l_1 + l_3}$ 	- for $h_2/h_1 > 1.5$ $h_e = h_1$ - $h_1 > 50$ mm

Table 3 Effective thickness of a composite concrete slab with profiled steel sheet as function of the fire resistance time.

As is seen from the equation for h_e , the effective thickness corresponds to an arithmetical average of the thickness which takes account of the profiled shape of the slab.

The formula may only be used if the minimum thickness of the slab, h_1 , exceeds 50 mm. For $h_2/h_1 > 1.5$, the effective thickness should be taken equal to h_1 .

Fig. 5 provides a comparison between the above presented rules and test results. It is seen that all test results are on the safe side.

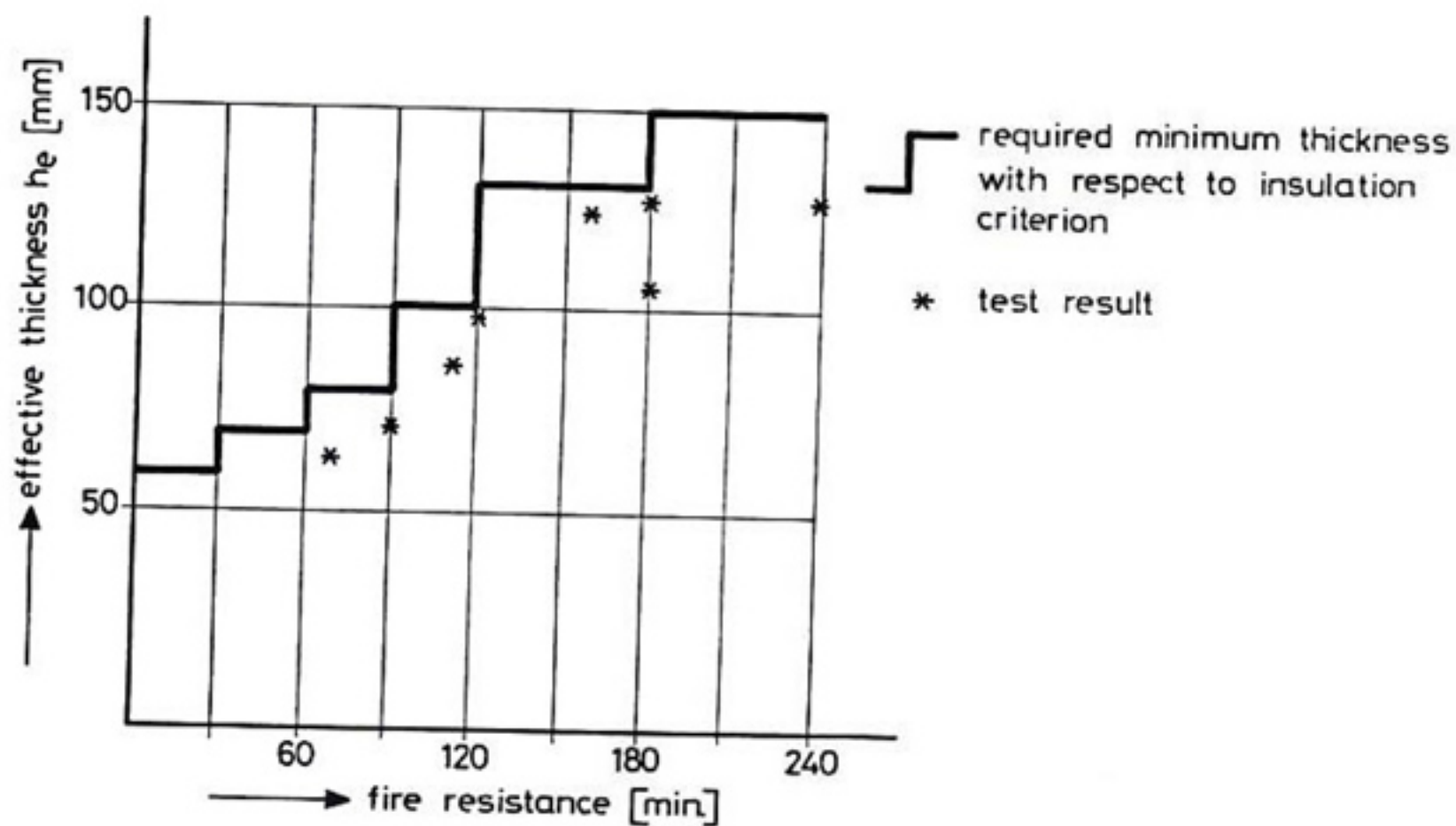


Fig. 5. Experimental verification of the rules for minimum slab thickness.

6. CALCULATION RULES FOR ADDITIONAL REINFORCEMENT6.1 Failure conditions

The load bearing capacity may be analysed on the basis of elementary plastic theory (limit state design). For various statical systems the failure conditions can then easily be formulated (Table 4).

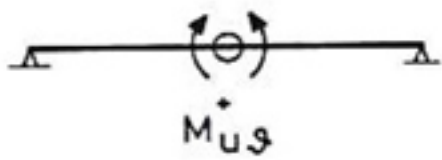
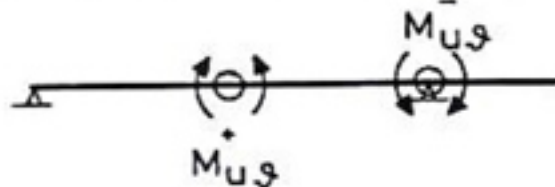
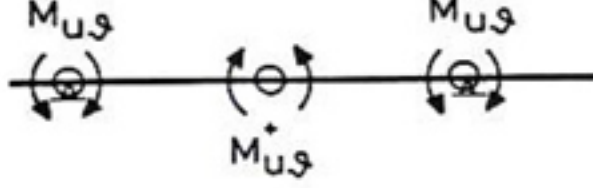
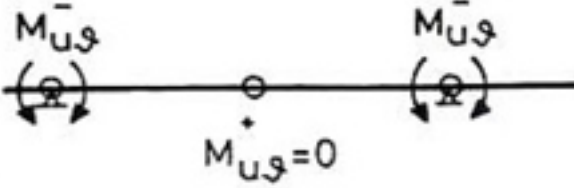
Statical system		Failure condition
no negative reinforcement		$M_{u\vartheta}^+ \leq q \cdot L^2/8$ or $q \geq 8 \cdot M_{u\vartheta}^+/L^2$
negative and positive reinforcement		$M_{u\vartheta}^+ + 0.5 \cdot M_{u\vartheta}^- \leq q \cdot L^2/8$ or $q \geq (8M_{u\vartheta}^+ + 4M_{u\vartheta}^-)/L^2$
		$M_{u\vartheta}^+ + M_{u\vartheta}^- \leq q \cdot L^2/8$ or $q \geq 8 \cdot (M_{u\vartheta}^+ + M_{u\vartheta}^-)/L^2$
no positive reinforcement		$M_{u\vartheta}^- \leq q \cdot L^2/8$ or $q \geq 8 \cdot M_{u\vartheta}^-/L^2$

Table 4 Failure conditions for slabs.

In Table 4 the following notation is used:

- $M_{u\psi}^+$, $M_{u\psi}^-$ - absolute value of the positive and negative plastic bending moment respectively at the end of the required period of standard fire exposure
- q - load on the slab to be accounted for during fire
- L - span of the slab.

To evaluate the failure conditions it is necessary to quantify the plastic moments $M_{u\psi}^+$ and $M_{u\psi}^-$. Typical stress distributions over the cross section are represented in Fig. 6 and 7 respectively.

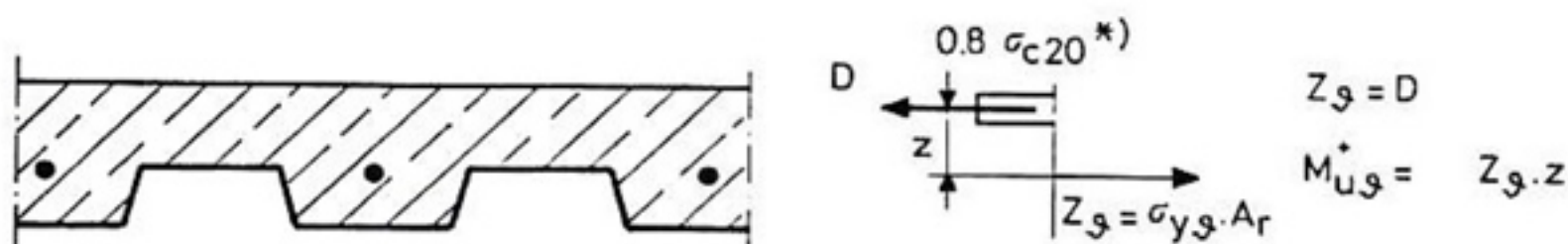


Fig. 6. Positive plastic moment $M_{u\psi}^+$

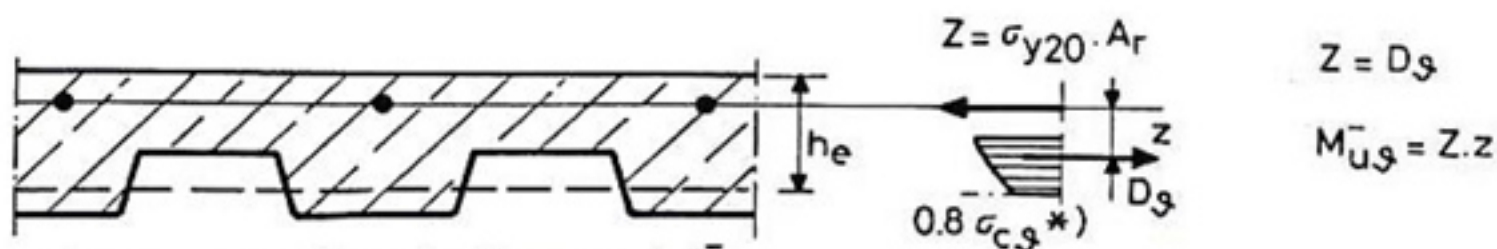


Fig. 7. Negative plastic moment $M_{u\psi}^-$

The following simplifying assumptions are made:

General:

- the tensile strength of concrete does not contribute to the load bearing capacity at elevated temperatures and thus may be ignored,
- the steel sheet does not contribute to the load bearing capacity at elevated temperatures and thus may be ignored.

*) The factor 0.8 is introduced to correct for the assumed full plastic stress distribution in the concrete compression zone. In the ultimate state, a non uniform stress distribution will occur, due to the limited capacity of the concrete to accept deformation.

For the positive plastic moment:

- the ultimate strength of concrete in the compression zone is not influenced by temperature and the room temperature values may be taken.
- the effective yield stress of the additional reinforcement is affected by temperature. This temperature is a function of the position of the reinforcement bars and may be calculated according to 6.4. The relationship between the yield stress and the temperature is specified under 6.5.

For the negative plastic moment:

- in calculations, the profiled concrete slab may be replaced by a slab with a uniform thickness equal to the effective thickness h_e in accordance with Table 3.
- the ultimate strength of concrete in the compression zone (exposed side) is affected by the temperature. Approximate temperature distributions are given under 6.4 and the relationship between the compressive strength and the temperature of concrete is given in 6.5.
- the effective yield stress of the reinforcement (unexposed side) is not influenced by temperature and room temperature values may be used.

On basis of the above mentioned assumptions and using the information as presented under 6.4 and 6.5, the evaluation of the failure conditions can proceed in a similiar way as for conventional reinforced concrete slabs under ambient temperature conditions.

First however, the structural load to be accounted for, as well as some additional design consideration should be discussed.

6.2 Structural load to be accounted for

In the experimental approach according to ISO standard 834 [1], the design load (dead load + characteristic live load) is considered to act on the structure throughout the fire.

The analytical rules given in 6.1 are based on the assumption of a uniformly distributed temperature over the width and length of the slab. Moreover, nominal values of the mechanical properties of both concrete and reinforcement steel are adopted, while - in case of fire - the contribution of the steel sheet to the load bearing capacity is neglected.

Compared with the amount of fire resistance measured in a fire resistance test, the calculations will consequently give a conservative result.

The fire resistance testing procedure is accepted world wide, and can not easily be changed. In order to bridge the above mentioned discrepancy, the structural load may be reduced for use in calculations as follows:

$$q = \kappa \cdot q^{\star} \quad \dots(3)$$

with

q = load to be accounted for

$q^{\star}$ = load considered to act on the slab (chosen in accordance with ISO 834)

κ = load multiplier (=0.85)

For similar reasons a load multiplier is introduced in [7], when dealing with the analytical determination of the fire resistance of structural steel elements. These κ -values vary with the load level and the statical system involved. See also [8]. Since in the case of fire exposed steel-concrete slabs only limited information is available, a single value of $\kappa = 0.85$ will be adopted in this Technical Note. This value is based on comparative calculations.

6.3 Design considerations

In statically indeterminate slabs a redistribution of moments will occur during the period of fire exposure. This phenomenon will be discussed for the continuous slab presented in Fig. 8^a. The moment distribution at room temperature under working load conditions is shown in Fig. 8^b

(a) Statical system :

(b) Moment distribution at room temperature :

(c) Moment distribution in an initial stage of fire exposure :

(d) Moment distribution at failure :

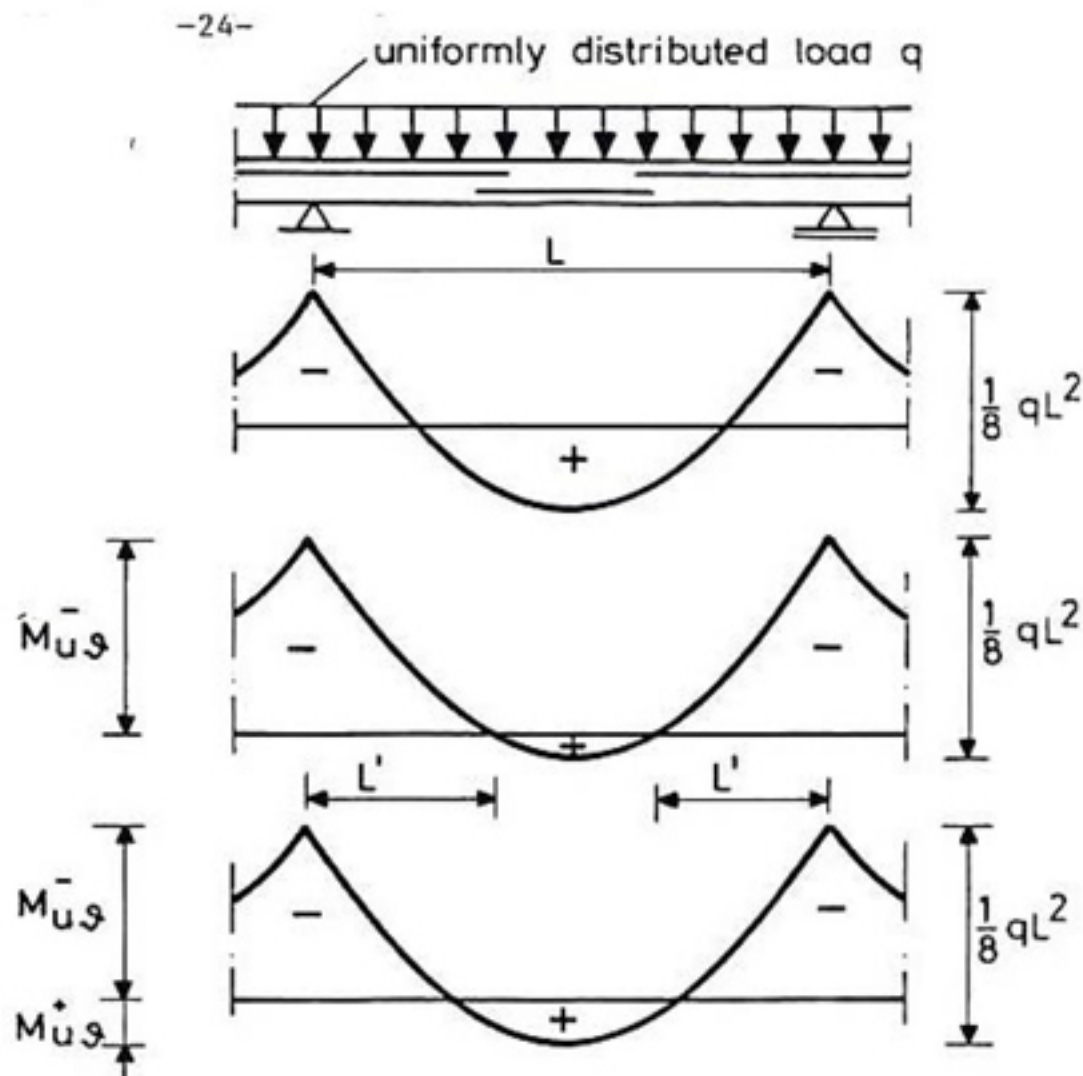


Fig. 8. Moment redistribution in a continuous slab during fire exposure.

Directly after commencement of the fire exposure, a steep temperature gradient will be attained over the height of the slab due to the relatively low thermal conductivity of the concrete. Consequently, additional negative bending moments will develop, which relieve the positive moment in the mid span region but increase the negative moment at the supports. Since, in the first stage of fire exposure, the value of the full plastic moment at the supports will not be affected significantly by temperature, a moment distribution as presented in Fig. 8^c will tend to occur (conservative assumption). The additional negative reinforcement should then be designed to cope with such a moment distribution. This means that the additional reinforcement at the supports shall be extended at least over a distance L' (cf Fig. 8^c). The anchorage length should be determined in accordance with room temperature design. For other statical systems, other minimum lengths will apply.

When heating continues, both the (positive) plastic moment capacity at midspan and the (negative) plastic moment capacity at the supports will decrease, finally leading to failure. The moment distribution at failure is presented in Fig. 8^d and corresponds to the relevant condition given in

Table 4. To arrive at such a moment distribution, sufficient rotation capacity is necessary, especially at the supports. The amount of the negative reinforcement and its ductility are then of crucial importance. The present state of knowledge does not allow the formulation of criteria, specifically derived for fire circumstances. In the room temperature design, however, a limit is normally set for both the maximum and the minimum amount of negative reinforcement in order to guarantee adequate rotation capacity at the supports. When these rules are obeyed, it will be assumed in this Technical Note that under fire conditions the necessary moment redistribution is also possible. Also the ductility of the reinforcement steel should meet room temperature specifications.

6.4 Temperature distribution

Concrete

The temperature distribution in the concrete slab is assumed to be independent of the effective thickness h_e and can be derived, for various times of the standard fire exposure, from Table 5 [9].

Depth X mm	Temperature in °C after a fire duration [min] of:				
	60	90	120	180	
5	705				
10	642	738			
15	581	681	754		
20	525	627	697		
25	469	571	642	738	
30	421	519	591	689	
35	374	473	542	635	
40	327	428	493	590	
45	289	387	454	549	
50	250	345	415	508	
55	200	294	369	469	
60		271	342	438	

Table 5 Temperature distribution in the concrete.

Additional reinforcement

The temperature of the additional reinforcement depends on the position of the reinforcement bars and the shape of the steel sheet profile. Both factors can be represented by the coefficient γ which is given by following equation (Fig. 9): [6]

$$\frac{1}{\gamma} = \frac{1}{\sqrt{u_1}} + \frac{1}{\sqrt{u_2}} + \frac{1}{\sqrt{u_3}}$$

The distances u_1 , u_2 and u_3 shall be taken in mm.



Fig. 9. Calculation of the coefficient γ .

The temperature of the reinforcement bars can be calculated using the following equations:

- fire duration 30 min: $\vartheta_s = 900 - 350 \cdot \gamma \leq 500 \text{ } ^\circ\text{C}$ ($\gamma \leq 2,5$)
- fire duration 60 min: $\vartheta_s = 1175 - 350 \cdot \gamma \leq 810 \text{ } ^\circ\text{C}$ ($\gamma \leq 3,3$)
- fire duration 90 min: $\vartheta_s = 1285 - 350 \cdot \gamma \leq 880 \text{ } ^\circ\text{C}$ ($\gamma \leq 3,6$)
- fire duration 120 min: $\vartheta_s = 1370 - 350 \cdot \gamma \leq 930 \text{ } ^\circ\text{C}$ ($\gamma \leq 3,8$)

6.5 Mechanical properties at elevated temperature

Concrete

The following, approximate relation between the ultimate compressive strength $\sigma_{c\vartheta}$ and the temperature ϑ_c of concrete may be used [10].

$$\vartheta_c < 200 \text{ } ^\circ\text{C}; \quad \sigma_{c\vartheta} = \sigma_{cu}$$

$$200 \leq \vartheta_c \leq 700 \text{ } ^\circ\text{C}; \quad \sigma_{c\vartheta} = \sigma_{c20} \cdot \left(1 - 0,8 \frac{\vartheta_c - 200}{500}\right) = \alpha \cdot \sigma_{c20}$$

$$\vartheta_c > 700 \text{ } ^\circ\text{C}; \quad \sigma_{c\vartheta} = 0$$

Where σ_{c20} = compression strength of concrete at ambient temperature.

The strength of the compressive zone of a slab (Fig. 10) with a width of 1000 mm can be calculated as follows:

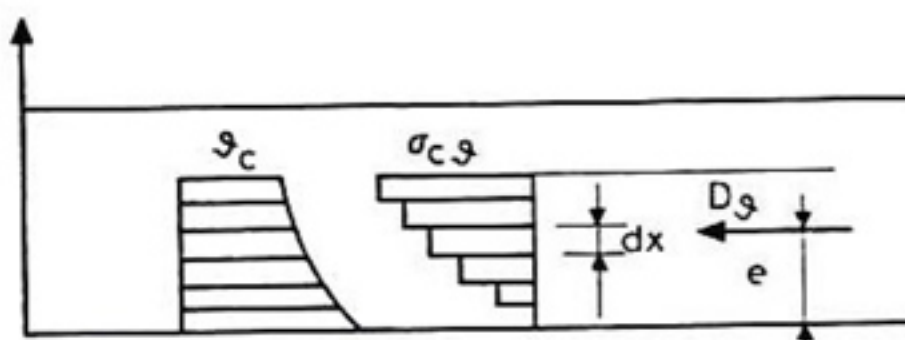


Fig. 10. Resultant compressive force in the concrete compressive zone.

$$D_{\vartheta} = 0,8 \sum (\sigma_{c\vartheta} \cdot dx \cdot 1000) = 0,8 \sigma_{c20} \cdot \sum (\alpha \cdot dx \cdot 1000) = 0,8 \sigma_{c20} \cdot A_{cr} \text{ with}$$

$$e = (\sum (\sigma_{c\vartheta} \cdot dx \cdot 1000) \cdot x) / D_{\vartheta}$$

The reduced compressive area A_{cr} in mm^2 and the position of the compressive force e in mm are given in Table 6.

Total depth of compressive zone	Values of A_{cr} and e after a fire duration in min.							
	60		90		120		180	
	A_{cr} $\times 10^{-2}$	e	A_{cr} $\times 10^{-2}$	e	A_{cr} $\times 10^{-2}$	e	A_{cr} $\times 10^{-2}$	e
mm	mm ²	mm	mm ²	mm	mm ²	mm	mm ²	mm
10	12.1	7.5						
15	29.1	10.4						
20	50.9	13.5	13.7	17.5				
25	77.1	16.5	31.8	20.3	12.4	22.5		
30	107.5	19.6	54.2	23.3	29.0	25.4		
35	141.7	22.7	80.5	26.3	49.7	28.3	13.0	32.5
40	179.6	25.9	110.4	29.3	74.2	31.4	30.0	35.3
45	221.0	29.0	143.8	32.4	102.3	34.4	50.4	38.2
50	265.4	32.1	180.5	35.5	133.5	37.5	74.1	41.2
55	313.4	35.2	220.9	38.6	168.2	40.6	101.0	44.2
60	363.4	38.3	264.2	41.7	205.7	43.7	130.7	47.2
65			309.6	44.7	245.4	46.7	162.8	50.2
70			357.0	47.8	287.2	49.7	197.0	53.2
75			405.9	50.7	331.4	52.8	233.4	56.2
80			455.9	53.7	376.7	55.7	271.4	59.2
85							311.1	62.2
90							352.3	65.1

Table 6 Strength of the concrete compressive zone at elevated temperature.

Steel

The following approximate relation between the effective yield stress $\sigma_{y\theta}$ and the temperature θ_s of the additional reinforcement may be used: [10]

$$\theta \leq 250 \text{ }^{\circ}\text{C} \quad \sigma_{y\theta} = 1.0 \sigma_{y20}$$

$$250 \text{ }^{\circ}\text{C} \leq \theta \leq 650 \text{ }^{\circ}\text{C} \quad \sigma_{y\theta} = \sigma_{y20} \cdot \left(1 - 0.6 \cdot \frac{\theta - 250}{400}\right) \quad \text{for hot rolled bars}$$

$$\sigma_{y\theta} = \sigma_{y20} \cdot \left(1 - 0.8 \cdot \frac{\theta - 250}{400}\right) \quad \text{for cold drawn bars}$$

where: σ_{y20} = yield stress at room temperature.

7. CALCULATION EXAMPLES

7.1 Simply supported slab

Assumptions:

Consider a simply supported slab, with a cross section as indicated in Fig. 11. Span $L = 3$ m. Characteristic load $q^* = 4500 \text{ N/m}^2$. Additional reinforcement $\phi 8$ in every rib with $\sigma_{y20} = 420 \text{ N/mm}^2$. Concrete with $\sigma_{c20} = 18 \text{ N/mm}^2$.

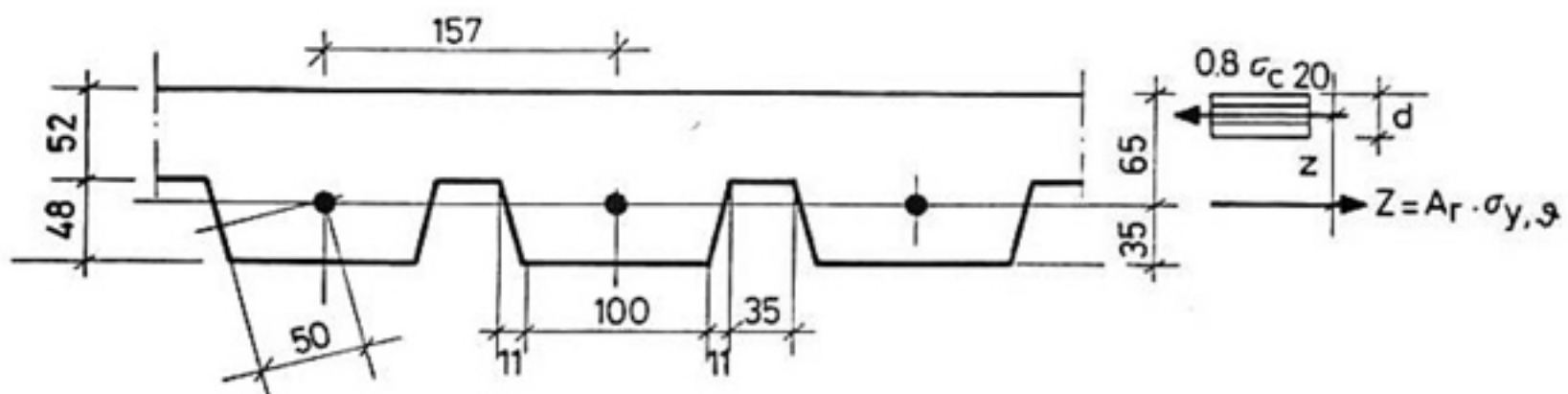


Fig. 11. Calculation example 7.1

Required:

A fire resistance of 90 minutes.

Check 1: Insulation

Required effective thickness: $h_e \geq 80 \text{ mm}$ (from Table 3).

Effective thickness: $h_1 = 52 \text{ mm} (\geq 50 \text{ mm})$; $h_2 = 48 \text{ mm}$

$l_1 = 122 \text{ mm}$; $l_2 = 100 \text{ mm}$; $l_3 = 35 \text{ mm}$

$$h_e = 52 + \frac{48(122+100)}{2(122+35)} = 86 \text{ mm} > 80 \text{ mm}.$$

Check 2: Load bearing capacity

Required:

$$q_{u,y} > \kappa q^*, \text{ resp. } M_{u,y}^+ > \kappa M^*$$

Maximum bending moment in slab:

$$M = \frac{1}{8} \kappa q^* L^2 = \frac{1}{8} \times 0.85 \times 4.5 \times 3^2 = 4.30 \text{ kNm}$$

Temperature of additional reinforcement after 90 minutes of standard fire exposure:

$$T_s = 1285 - 350 \gamma \leq 880 \text{ } ^\circ\text{C}; \quad (\gamma \leq 3.6)$$

$$u_1 = u_2 = 50 \text{ mm}; u_3 = 35 \text{ mm}$$

$$\frac{1}{\gamma} = 1/50 + 1/50 + 1/35 = 0.45$$

$$\gamma = 2.2$$

$$T_s = 1285 - 350 \times 2.2 = 515 \text{ } ^\circ\text{C}.$$

Effective yield stress at this temperature:

$$\sigma_{y,T} = 420 \times \left(1 - \frac{0.6 \times (515 - 250)}{400}\right) = 253 \text{ N/mm}^2$$

Area of the additional reinforcement:

$$\phi 8 \text{ every } 0.157 \text{ m: } A_s = 320 \text{ mm}^2/\text{m}$$

It then follows (cf Fig. 11)

$$Z_{y,T} = A_r \times \sigma_{y,T} = 320 \times 253 \times 10^{-3} = 81 \text{ kN}$$

$$d = \frac{Z_{y,T}}{0.8 \sigma_{c20} \times 10^3} = \frac{81.0 \times 10^3}{0.8 \times 18 \times 10^3} = 5.6 \text{ mm}$$

Plastic moment after 90 minutes of standard fire exposure:

$$M_{uy}^+ = Z(H - 0.5 d) = 81.0 (65 - 0.5 \times 5.6) \times 10^{-3} \\ = 5.03 \text{ kNm} > 4.30 \text{ kNm}$$

Hence $M_{u\bar{u}}^+ > M$

Conclusion:

The fire resistance of the slab is more than 90 minutes.

7.2 Continuous slab

Assumptions:

Consider a continuous slab with a cross section as indicated in Fig. 12^a and 12^b. Span $L = 5 \times 2.80$ m. Characteristic load: $q^* = 7500 \text{ N/m}^2$.

Reinforcement for negative moments: ϕ 8 over every rib, with $\sigma_{y20} = 420 \text{ N/mm}^2$. Concrete with $\sigma_{c20} = 20 \text{ N/mm}^2$

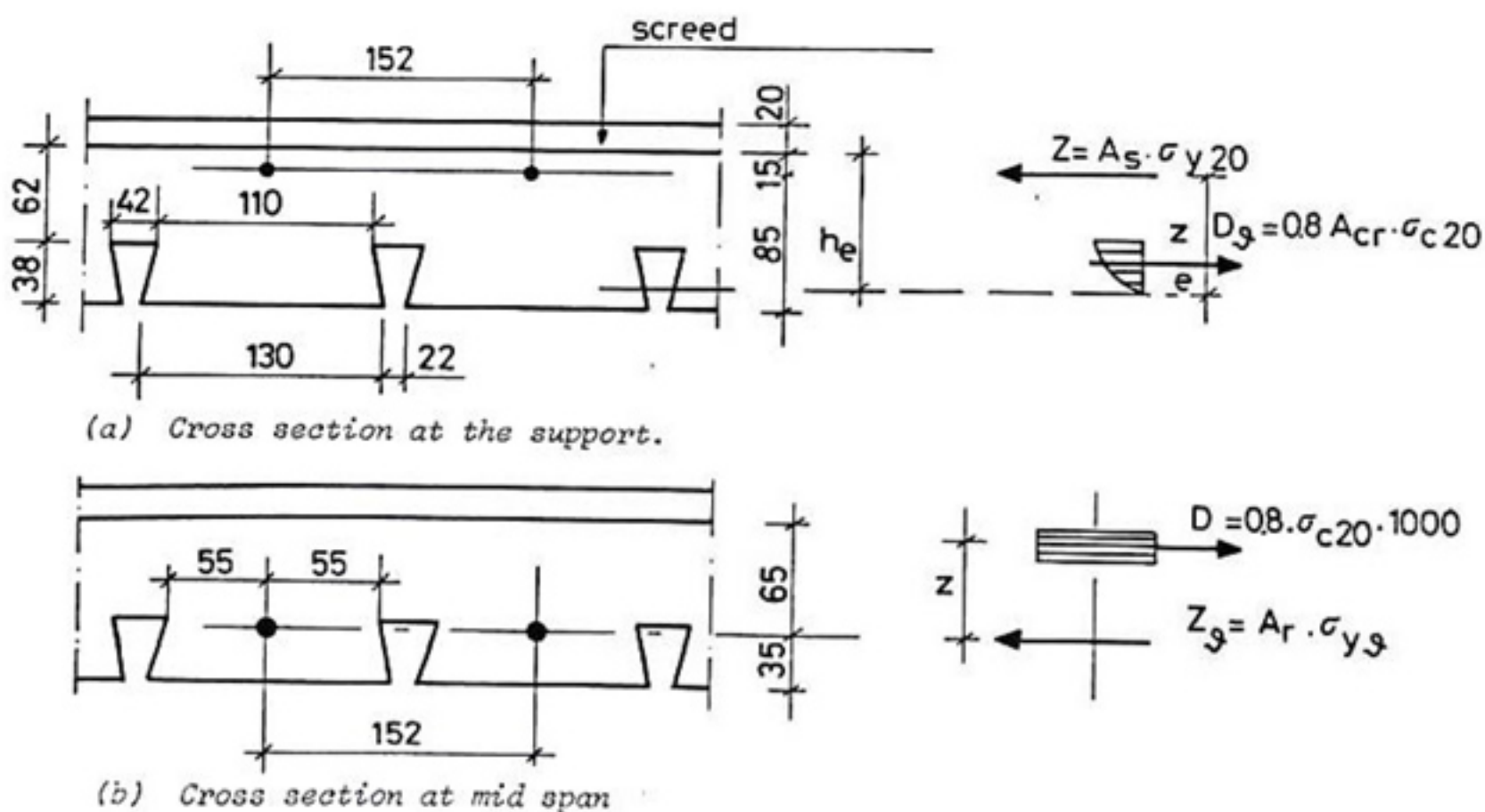


Fig. 12. Calculation example 7.2

Required:

A fire resistance of 90 minutes.

Check 1: Insulation

Required effective thickness $h_e \geq 80$ mm.

Effective thickness: $h_1 = 62$ mm (≥ 50 mm); $h_2 = 38$ mm

$l_1 = 110$ mm; $l_2 = 130$ mm; $l_3 = 42$ mm

$$h_e = 62 + \frac{38(110+130)}{2(110+42)} = 92 \text{ mm} \geq 80 \text{ mm}.$$

Note that, when checking the insulation criterion, the concrete screed may be added to h_e . The effective slab thickness then becomes $92 + 20 = 112$ mm which is more than sufficient.

Check 2^a: Load bearing capacity for end span

Required:

$$M_{u\psi}^+ + 0.5 M_{u\psi}^- > \frac{1}{8} \kappa q^* L^2$$

$$\text{with } \frac{1}{8} \kappa q^* L^2 = \frac{1}{8} \times 0.85 \times 7.5 \times (2.8)^2 = 6.25 \text{ kNm}$$

Plastic moment at support ($M_{u,\psi}^-$):

$$\phi 8 \text{ every } 0.152 \text{ m: } A_r = 330 \text{ mm}^2/\text{m}$$

It then follows: (cf Fig. 12^a)

$$Z = A_r \sigma_{y20} = 330 \times 420 \times 10^{-3} = 138.6 \text{ kN}$$

Compression force in concrete ($\sigma_{c20} = 20 \text{ N/mm}^2$)

From Table 6 for a period of fire exposure of 90 minutes:

- compression depth: 35 mm (= d)

$$D = 0.8 A_{cr} \sigma_{c20} = 0.8 \times 8050 \times 20 \times 10^{-3} = 129 \text{ kN}$$

$$e = 26.3 \text{ mm} = 0.75 d$$

- compression depth: 40 mm (= d)

$$D = 0.8 \times 1140 \times 20 \times 10^{-3} = 177 \text{ kN}$$

$$e = 29.3 \text{ mm} = 0.73 d$$

For $D = Z = 138.6 \text{ kN}$ it follows by linear interpolation:

$$d = \frac{138.6 - 129}{177 - 129} \times 5 + 35 = 36 \text{ mm}$$

and $e = 26.9 \text{ mm}$

Hence $z = h_e - e - u' = 92 - 26.9 - 15 = 50.1 \text{ mm}$

Thus $M_{u\psi}^- = 138.6 \times 0.0501 = 6.94 \text{ kNm}$

The necessary plastic moment ($M_{u\psi}^+$) follows from:

$$M_{u\psi}^+ = 6.25 - 0.5 \times 6.94 = 2.78 \text{ kNm}$$

The temperature of the additional reinforcement after 90 minutes of standard fire exposure: (cf Fig. 12^b).

$$\psi_s = 1285 - 350 \gamma \leq 880; (\gamma \leq 3.6)$$

$$u_1 = u_2 = 55 \text{ mm}; u_3 = 35 \text{ mm}$$

$$\frac{1}{\gamma} = 1/55 + 1/55 + 1/35 = 0.439$$

$$\gamma = 2.28; (< 3.6)$$

$$\psi_s = 1285 - 350 \times 2.28 = 487 \text{ }^\circ\text{C}.$$

Effective yield stress at this temperature:

$$\sigma_{y\psi} = 420 \left(1 - \frac{0.6(487-250)}{400}\right) = 271 \text{ N/mm}^2$$

With $z = 0.9 H = 0.9 \times 65 = 58.5 \text{ mm},$

The necessary reinforcement is

$$A_r = \frac{M_{uy}^+}{z \sigma_{yk}} = \frac{2.56 \times 10^6}{58.5 \times 271} = 161 \text{ mm}^2/\text{m}$$

Chosen: ϕ 8 m every rib: $A_r = 330 \text{ mm}^2/\text{m}$.

Then: (cf Fig. 12^b)

$$Z_y = 330 \times 271 \times 10^{-3} = 89.4 \text{ kN}$$

$$d = \frac{89.4 \times 10^3}{0.8 \times 20 \times 10^3} = 5.59 \text{ mm}$$

$$\begin{aligned} M_{uy}^+ &= 89.4 (0.065 - 0.5 \times 0.00559) = \\ &= 5.56 \text{ kNm} > 2.56 \text{ kNm}. \end{aligned}$$

Check 2^b: Load bearing capacity intermediate span

Required:

$$M_{uy}^+ + M_{uy}^- > \frac{1}{8} \kappa q^* L^2 \quad \text{or without reinforcement: } M_{uy}^- > \frac{1}{8} \kappa q^* L^2$$

Since $M_{uy}^- = 7.39 \text{ kNm}$ is greater than $\frac{1}{8} \kappa q^* L^2 = 6.25 \text{ kNm}$, no additional positive reinforcement is necessary.

Note that in the initial phase of fire exposure the positive moment will have been completely relieved due to thermal restraint. Consequently the negative reinforcement should be extended over the full length of the span. Ref.: chapter 6.3. Of course the amount of reinforcement may be adapted to the moment distribution.

Conclusion:

The proposed floor, with additional positive reinforcement only in the end spans, has a fire resistance of over 90 minutes.

APPENDIX: VALIDATION

Introduction

Two fire tests on composite steel concrete slabs, adequately documented to allow for a theoretical analysis, have been selected from literature. The experimental results will be compared with the analytical calculations obtained using the calculation rules presented in this Technical Note.

Test no. 1 (TH Darmstadt, [11])

Test arrangement and basic data

The test applies to a simply supported slab with a cross section as presented in Fig. A.

Span : 4.84 m
 Loading : 6438 N/m^2 (dead + live load)
 Concrete : $\sigma_{c20} = 30 \text{ N/mm}^2$ (design value)
 Reinforcement: $4 \phi 10$, hence $A_r = 393 \text{ mm}^2/\text{m}$
 $\sigma_{y20} = 460 \text{ N/mm}^2$ (design value)

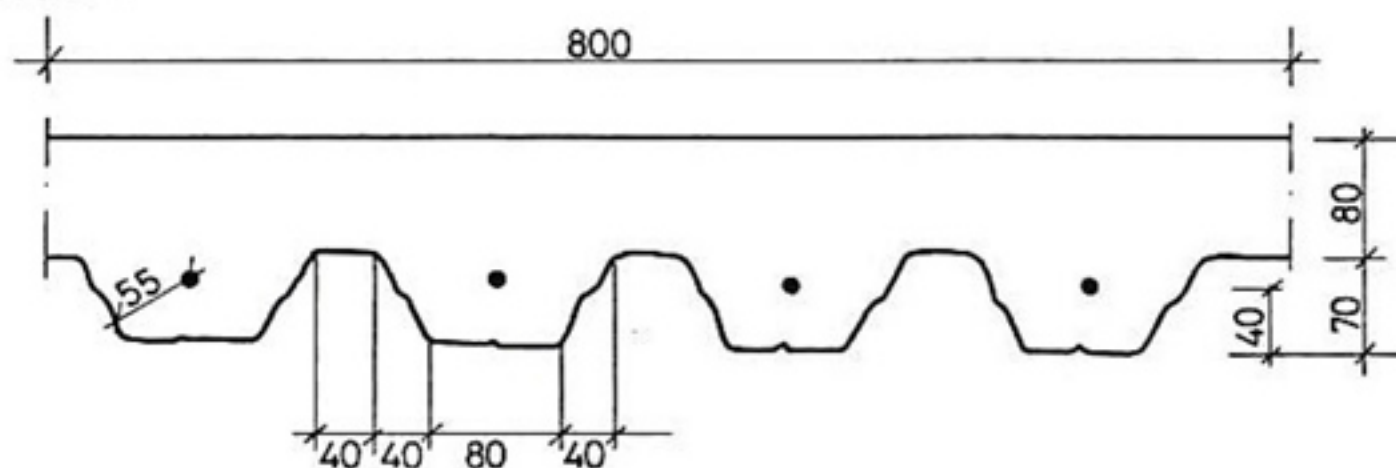


Fig. A. Cross section of the slab of test no. 1.

Test result

Achieved fire resistance : 93 minutes
 Relevant fire resistance test criterion: the test was stopped due to excessive deformation.

Analysis according the Technical Note

(i) Insulation (cf chapter 5)

$$h_1 = 80 \text{ mm}; h_2 = 70 \text{ mm}$$

$$l_1 = 160 \text{ mm}; l_2 = 80 \text{ mm}; l_3 = 40 \text{ mm}$$

hence (cf Table 3)

$$h_e = 80 + \frac{70}{2} \times \frac{160 + 80}{160 + 40} = 122 \text{ mm.}$$

It follows from Table 3 that the fire resistance with respect to insulation is well over 120 minutes.

(ii) Load bearing capacity (cf chapter 6)

Temperature of additional reinforcement (cf section 6.4)

$$u_1 = u_2 = 55 \text{ mm}; u_3 = 40 \text{ mm}$$

$$\frac{1}{\gamma} = \frac{1}{\sqrt{55}} + \frac{1}{\sqrt{55}} + \frac{1}{\sqrt{40}} = 0.43$$

$$\gamma = 2.34$$

It follows (cf section 6.4)

$$\text{after: 60 minutes: } \sigma_s = 1175 - 350 \times 2.34 = 356 \text{ }^\circ\text{C (measured 280 }^\circ\text{C)}$$

$$90 \text{ minutes: } \sigma_s = 1285 - 350 \times 2.34 = 466 \text{ }^\circ\text{C (" " 450 }^\circ\text{C)}$$

$$120 \text{ minutes: } \sigma_s = 1370 - 350 \times 2.34 = 551 \text{ }^\circ\text{C}$$

Effective yield stress for additional reinforcement (cf section 6.5)

$$\text{after: 60 minutes: } \sigma_{sy} = 460(1 - 0.6 \times \frac{356-250}{400}) = 387 \text{ N/mm}^2$$

$$90 \text{ minutes: } \sigma_{sy} = 460(1 - 0.6 \times \frac{466-250}{400}) = 311 \text{ N/mm}^2$$

$$120 \text{ minutes: } \sigma_{sy} = 460(1 - 0.6 \times \frac{551-250}{400}) = 252 \text{ N/mm}^2$$

Plastic moment (cf. Fig. 6)

$$M_{uy}^+ = \sigma_{y\gamma} A_r z$$

with

$$z = H - 0.5d \quad \text{and} \quad d = \frac{\sigma_{y\gamma} A_r}{0.8 \cdot 1000 \cdot \sigma_{c20}}$$

$$\text{after 60 minutes: } d = \frac{387 \times 393}{0.8 \times 1000 \times 30} = 6.3 \text{ mm}$$

$$z = 110 - 0.5 \times 6.3 = 106.8 \text{ mm}$$

$$M_{uy}^+ = 387 \times 393 \times 106.8 \times 10^{-6} = 16.2 \text{ kNm}$$

$$\text{after 90 minutes: } d = \frac{311 \times 393}{0.8 \times 1000 \times 30} = 5.1 \text{ mm}$$

$$z = 110 - 0.5 \times 5.1 = 107.5 \text{ mm}$$

$$M_{uy}^+ = 311 \times 393 \times 107.5 \times 10^{-6} = 13.1 \text{ kNm}$$

$$\text{after 120 minutes: } d = \frac{252 \times 393}{0.8 \times 1000 \times 30} = 4.1 \text{ mm}$$

$$z = 110 - 0.5 \times 4.1 = 107.9 \text{ mm}$$

$$M_{uy}^+ = 252 \times 393 \times 107.9 \times 10^{-6} = 10.7 \text{ kNm}$$

Check on load bearing capacity (cf. section 6.2)

$$M_{uy}^+ > 1/8 \times 0.85 \times 6.438 \times 4.84^2 = 16.0 \text{ kNm}$$

It follows that:

$$M_{uy}^+(90) < 16.0 < M_{uy}^+(60)$$

By linear interpolation, the calculation method leads to a fire resistance of 62 minutes. Compared with the experimental result (93 minutes) this is a conservative result.

Test no 2: (TH Darmstadt, [11])

Test arrangement and basic data

Cross section and

static system : simply supported slab as in test no 1 (cf Fig. A)
 Span : 4.84 m
 Loading : 6940 N/m²
 Concrete : $\sigma_{c20} = 30 \text{ N/mm}^2$ (design value)
 Reinforcement : 4 ϕ 12, hence $A_r = 565 \text{ mm}^2/\text{m}$
 $\sigma_{y20} = 460 \text{ N/mm}^2$ (design value)

Test results:

Achieved fire resistance : 120 minutes
 Relevant fire resistance test criterion : the test was stopped due to excessive deformation

(i) Insulation

See test no 1

Conclusion: fire resistance with respect to insulation is over 120 minutes.

(ii) Load bearing capacity

Temperature of additional reinforcement.

As in test no 1:

after 60 minutes: 356 °C (measured: 250 °C)
 after 90 minutes: 466 °C (" : 390 °C)
 after 120 minutes: 551 °C (" : 500 °C)

Effective yield stress of additional reinforcement.

As in test no 1:

after 60 minutes: $\sigma_{y\psi} = 387 \text{ N/mm}^2$
 90 minutes: $\sigma_{y\psi} = 311 \text{ N/mm}^2$
 120 minutes: $\sigma_{y\psi} = 252 \text{ N/mm}^2$

Plastic moment:

after 60 minutes: $d = 9.1 \text{ mm}$; $z = 105.4 \text{ mm}$; $M_{u\psi}^+ = 23.0 \text{ kNm}$
 90 minutes: $d = 7.3 \text{ mm}$; $z = 106.3 \text{ mm}$; $M_{u\psi}^+ = 18.7 \text{ kNm}$
 120 minutes: $d = 5.9 \text{ mm}$; $z = 107.0 \text{ mm}$; $M_{u\psi}^+ = 15.2 \text{ kNm}$.

Check on load bearing capacity

$$M_{u\psi}^+ > 1/8 \times 0.85 \times 6.940 \times 4.84^2 = 17.3 \text{ kNm}.$$

It follows that:

$$M_{u\psi}^+(90) < 17.3 < M_{u\psi}^+(120)$$

By linear interpolation, the calculation method leads to a fire resistance of 102 minutes. Compared with the experimental result (120 minutes) this is conservative.

Test no. 3: (EMPA, ref. report: 66356/3)

Test arrangement and basic data

The test applies to a simply supported slab with a cross section as presented in Fig. B.

Span : 2.70 m
 Loading : 7000 N/m² (dead + live load)
 Concrete : $\sigma_{c20} = 25 \text{ N/mm}^2$ (design value)
 $\sigma_{c20} = 53.6 \text{ N/mm}^2$ (measured)

Reinforcement : ϕ 8 at distance of 152 mm, hence $A_r = 328 \text{ mm}^2/\text{m}$
 $\sigma_{y20} = 460 \text{ N/mm}^2$

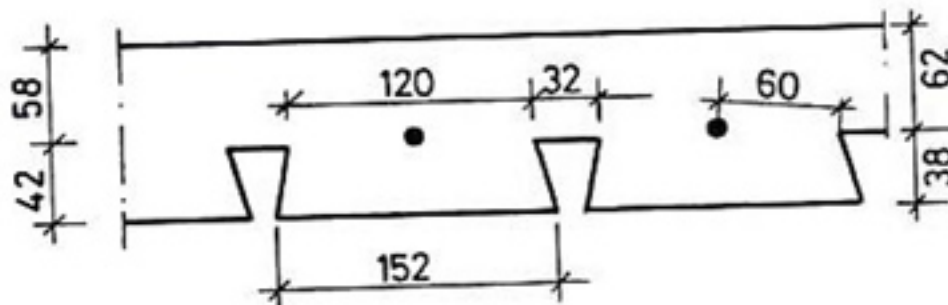


Fig. B. Cross section of the slab of test no. 3.

Test result

Achieved fire resistance : 90 minutes
 Relevant fire resistance test criterion : the test was stopped due to excessive deformation.

Analysis according to the Technical Note

(1) Insulation (cf chapter 5)

$$h_1 = 62 \text{ mm}; h_2 = 38 \text{ mm}$$

$$l_1 = 120 \text{ mm}; l_2 = 140 \text{ mm}; l_3 = 32 \text{ mm}$$

$$\text{hence: } h_e = 62 + \frac{38}{2} \cdot \frac{120+140}{120+32} = 94.5 \text{ mm.}$$

It follows from Table 3 that the fire resistance with respect to insulation is well over 90 minutes.

(11) Load bearing capacity (cf chapter 6)

Temperature of additional reinforcement

$$u_1 = u_2 = 60 \text{ mm}; u_3 = 42 \text{ mm.}$$

$$\frac{1}{\gamma} = 2 \times \frac{1}{\sqrt{60}} + \frac{1}{\sqrt{42}} = 0.41$$

$$\gamma = 2.42$$

It follows:

$$\begin{aligned} \text{after 60 minutes: } \gamma_s &= 1174 - 350 \times 2.42 = 328^\circ\text{C (measured } 200^\circ\text{C)} \\ 90 \text{ minutes: } \gamma_s &= 1285 - 350 \times 2.42 = 438^\circ\text{C (" " } 234^\circ\text{C)} \\ 120 \text{ minutes: } \gamma_s &= 1370 - 350 \times 2.42 = 523^\circ\text{C (- . -)} \end{aligned}$$

Effective yield stress for additional reinforcement

$$\begin{aligned} \text{after 60 minutes } \sigma_{yy} &= 460 \left(1 - 0.6 \times \frac{238-230}{400}\right) = 406 \text{ N/mm}^2 \\ 90 \text{ minutes } \sigma_{yy} &= 460 \left(1 - 0.6 \times \frac{438-250}{400}\right) = 330 \text{ N/mm}^2 \\ 120 \text{ minutes } \sigma_{yy} &= 460 \left(1 - 0.6 \times \frac{523-250}{400}\right) = 272 \text{ N/mm}^2 \end{aligned}$$

Plastic moment (cf Fig. 6)

$$\begin{aligned} \text{after 60 minutes: } d &= \frac{406 \times 328}{0.8 \times 1000 \times 53.6} = 3.1 \text{ mm} \\ z &= 58 - 0.5 \times 3.1 = 56.5 \text{ mm} \\ M_{uy}^+ &= 406 \times 328 \times 56.5 \times 10^{-6} = 7.52 \text{ kNm} \\ 90 \text{ minutes: } d &= 2.5 \text{ mm; } z = 56.8 \text{ mm; } M_{uy}^+ = 6.15 \text{ kNm} \\ 120 \text{ minutes: } d &= 2.1 \text{ mm; } z = 57.0 \text{ mm; } M_{uy}^+ = 5.08 \text{ kNm} \end{aligned}$$

Check on the load bearing capacity (cf Section 6.1).

Since in the above calculations the actual room temperature values for the mechanical properties of concrete and reinforcement are used, for the load multiplier κ of value of 1.0. rather than 0.85 should be chosen. It then follows:

$$M_{uy}^+ > 1/8 \times 1.0 \times 7 \times (2.7)^2 = 6.38 \text{ kNm}$$

Hence

$$M_{u\psi}^+(60) < 6.38 < M_{u\psi}^+(90) \cdot$$

By linear interpolation, the calculation methods leads to a fire resistance of 85 minutes. Compared with the experimental results (90 minutes) this is, again, conservative.

NOTATIONS

A_T	: area of reinforcement steel	$[\text{mm}^2/\text{m}]$
A_{cr}	: reduced area of the exposed concrete compression zone	$[\text{mm}^2/\text{m}]$
D	: compression force concrete (non exposed side)	$[\text{N}/\text{m}]$
D_y	: compression force concrete (exposed side)	$[\text{N}/\text{m}]$
H	: structural height of the slab	$[\text{mm}]$
L	: span of the slab	$[\text{m}]$
L'	: minimum distance over which the negative reinforcement should be extended (anchorage length excluded)	$[\text{mm}]$
M	: bending moment	$[\text{Nm}/\text{m}]$
M_{uy}^+	: positive plastic moment after a specified period of standard fire exposure	$[\text{Nm}/\text{m}]$
M_{uy}^-	: negative plastic moment after a specified period of standard fire exposure	$[\text{Nm}/\text{m}]$
Z	: tensile force in reinforcement (non exposed side)	$[\text{Nm}]$
Z_y	: tensile force in reinforcement (exposed sided)	$[\text{Nm}]$
d	: depth of concrete compression zone (non exposed side)	$[\text{mm}]$
e	: position of concrete compression force (exposed side)	$[\text{mm}]$
h_e	: effective thickness of the slab	$[\text{mm}]$
$h_{1,2}$	: minimum and maximum thickness of the slab respectively	$[\text{mm}]$
$l_{1,2,3}$	: profile dimensions of the steel sheet	
q	: loading on the slab, to be used in an analytical fire safety design	$[\text{N}/\text{m}^2]$
q^*	: loading on the slab to be accounted for in fire test according to ISO 834	$[\text{N}/\text{m}^2]$
q_{uy}	: load bearing capacity of the slab after a specified period of standard fire exposure	$[\text{Nm}^2]$
t	: time	$[\text{min}]$
u	: cover to positive reinforcement (axial)	$[\text{mm}]$
u'	: cover to negative reinforcement (axial)	$[\text{mm}]$
$u_{1,2,3}$	: position of reinforcement bar (exposed side)	$[\text{mm}]$
z	: arm of internal bending moment	$[\text{mm}]$

α	: ratio between compressive strength of concrete at elevated temperature and at room temperature	[-]
γ	: coefficient used to calculate the temperature of the reinforcement bars (exposed side).	[$m^{-\frac{1}{2}}$]
δ	: deflection	[mm]
ϑ_o	: initial ambient gas temperature	[°C]
ϑ_c	: temperature of concrete	[°C]
ϑ_s	: temperature of reinforcement steel	[°C]
ϑ_t	: ambient gas temperature at time t	
κ	: load multiplier (< 1)	[-]
σ_{c20}	: compressive strength of concrete at room temperature	[N/mm ²]
$\sigma_{c\vartheta}$	: compressive strength of concrete at elevated temperature	[N/mm ²]
σ_{y20}	: yield stress of reinforcement steel at room temperature	[N/mm ²]
$\sigma_{y\vartheta}$	: yield stress of reinforcement steel at elevated temperature	[N/mm ²]

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