Research Fund
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MACS+

Membrane Action of Composite Structures in Case of Fire

Design Guide

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FOREWORD

Membrane Action in Fire design of Composite Slab with solid and cellular steel beams - Valorisation (MACS+)

This project has been funded with support from the European Commission, Research Fund for Coal and Steel.

The publication has been produced as a result of different research projects:

- The RFCS Project FICEB+
- The RFCS Project COSSFIRE
- The project Leonardo Da Vinci 'Fire Resistance Assessment of Partially Protected Composite Floors' (FRACOF).
- A former project sponsored jointly by ArcelorMittal and CTICM and executed by a partnership of CTICM and SCI.

The simple design method was initially developed as the result of large scale fire testing conducted on a multi-storey steel framed building at the Building Research Establishment's Cardington test facility in the UK. Much of the theoretical basis of the design method has been in existence since the late 1950's, following studies of the structural behaviour of reinforcement concrete slabs at room temperature. The first version of the simple design method was available in the SCI Design Guide P288 'Fire Safe Design: A new approach to Multi-story Steel Framed Buildings', 2 Ed.

Although the application of the method to fire resistance design is relatively new the engineering basis of the method is well established.

The simple design method was implemented in a software format by SCI in 2000 and an updated version was released in 2006, following improvements to the simple design method.

Valuable contributions were received from:

- Mary Brettle The Steel Construction Institute
- Ian Sims The Steel Construction Institute
- Louis Guy Cajot ArcelorMittal
- Renata Obiala ArcelorMittal
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Contents

SUMMARY

Large-scale fire tests conducted in a number of countries and observations of actual building fires have shown that the fire performance of composite steel framed buildings is much better than is indicated by fire resistance tests on isolated elements. It is clear that there are large reserves of fire resistance in modern steel-framed buildings and that standard fire resistance tests on single unrestrained members do not provide a satisfactory indicator of the performance of such structures.

This publication presents guidance on the application of a simple design method, as implemented in MACS+ software. The recommendations are conservative and are limited to structures similar to that tested, i.e. non-sway steel-framed buildings with composite floors and composite floors with Cellular Beams. The guidance gives designers access to whole building behaviour and allows them to determine which members can remain unprotected while maintaining levels of safety equivalent to traditional methods.

In recognition that many fire safety engineers are now considering natural fires, a natural fire model is included alongside the use of the standard fire model, both expressed as temperature-time curves in Eurocode 1.

In addition to the design guidance provided by this publication, a separate Engineering Background document provides details of fire testing and finite element analysis conducted as part of the FRACOF, COSSFIRE and FICEB project and some details of the Cardington tests which were conducted on the eight-storey building at Cardington. The background document will assist the reader to understand the basis of the design recommendations in this publication.

1 INTRODUCTION

The design recommendations in this publication are based on the performance of composite floor plates, as interpreted from actual building fires and from full-scale fire tests[1,2,3]. These conservative recommendations for fire design may be considered as equivalent to advanced methods in the Eurocodes.

The elements of structure of multi-storey buildings are required by national building regulations to have fire resistance. The fire resistance may be established from performance in standard fire resistance tests or by calculations in accordance with recognised standards, notably EN 1991-1-2[4], EN 1993-1-2[5] and EN 1994-1-2[6]. In a standard fire test, single, isolated and unprotected I or H section steel beams can only be expected to achieve 15 to 20 minutes fire resistance. It has thus been normal practice to protect steel beams and columns by use of fire resisting boards, sprays or intumescent coatings, or, in slim floor or shelf angle floor construction, by encasing the structural elements within floors.

Large-scale natural fire tests[7] carried out in a number of countries have shown consistently that the inherent fire performance of composite floor plates with unprotected steel elements is much better than the results of standard tests with isolated elements would suggest. Evidence from real fires indicates that the amount of protection being applied to steel elements may be excessive in some cases. In particular, the Cardington fire tests presented an opportunity to examine the behaviour of a real structure in fire and to assess the fire resistance of unprotected composite structures under realistic conditions.

As the design recommendations given in this publication are related to generalised compartment fire, they can be easily applied under standard fire condition such as it is demonstrated through the real scale floor test within the scope of FRACOF and COSSFIRE project. Obviously, this possibility provides a huge advantage to engineers in their fire safety design of multi-storey buildings with steel structures. Large scale fire test realised in Ulster in the scope of the FICEB project highlight that the membrane action theory can also be applied with Cellular Beams.

Where national building regulations permit performance-based design of buildings in fire, the design method provided by this guide may be applied to demonstrate the fire resistance of the structure without applied fire protection. In some countries acceptance of such demonstration may require special permission from the national building control authority.

The recommendations presented in this publication can be seen as extending the fire engineering approach in the area of structural performance and developing the concept of fire safe design. It is intended that designs carried out in accordance with these recommendations will achieve at least the level of safety required by national regulations while allowing some economies in construction costs.

In addition to fire resistance for the standard temperature-time curve, recommendations are presented for buildings designed to withstand a natural fire. Natural fires can be defined in the MACS+ software using the parametric temperature-time curve given in EN 1991-1-2. This takes account of the size of the compartment, the size of any openings and the amount of combustibles. Alternatively, the MACS+ software permits temperature-time curves to be read from a text file, allowing output from other fire models to be used.

The recommendations apply to composite frames broadly similar to the eight-storey building tested at Cardington, as illustrated in Figure 1-1 and Figure 1-2.

The design recommendations are presented as guide to the application of the MACS+ software, which is available as a free download from *www.arcelormittal.com/sections*.

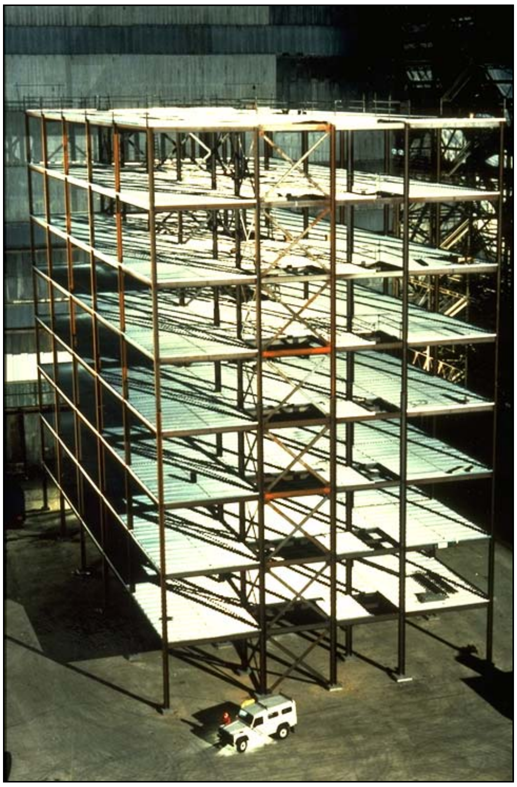


Figure 1-1 Cardington test building prior to the concreting of the floors

Figure 1-2 View of unprotected steel structure

2 BASIS OF DESIGN

This Section gives an overview of the design principles and assumptions underlying the development of the simple design method; more detailed information is given in the accompanying background document[7]. The type of structure that the design guidance is applicable to is also outlined.

The design guidance has been developed from research based on the results from fire tests, ambient temperature tests and finite element analyses.

2.1 Fire safety

The design recommendations given in the simple design method have been prepared such that the following fundamental fire safety requirements are fulfilled:

- There should be no increased risk to life safety of occupants, fire fighters and others in the vicinity of the building, relative to current practice.
- On the floor exposed to fire, excessive deformation should not cause failure of compartmentation, in other words, the fire will be contained within its compartment of origin and should not spread horizontally or vertically.

2.2 Type of structure

The design guidance given in the simple design method applies only to steel-framed buildings with composite floor beams and slabs of the following general form:

- braced frames not sensitive to buckling in a sway mode,
- frames with connections designed using simple joint models,
- composite floor slabs comprising steel decking, a single layer of reinforcing mesh and normal or lightweight concrete, designed in accordance with EN 1994-1-1[9],
- floor beams designed to act compositely with the floor slab and designed to EN 1994-1-1.
- beams with service openings.

The guidance does **not** apply to:

- floors constructed using pre fabricated concrete slabs,
- internal floor beams that have been designed to act non-compositely (beams at the edge of the floor slab may be non-composite).

2.2.1 Simple joint models

The joint models adopted during the development of the guidance given in this publication assume that bending moments are not transferred through the joint. The joints are known as ‘simple’.

Beam-to-column joints that may be considered as ‘simple’ include joints with the following components:

- flexible end plates (Figure 2-1)
- fin plates (Figure 2-2)
- □eb cleats (Figure 2-3).

Further information on the design of the components of ‘simple’ joints is given in Section 3.6.

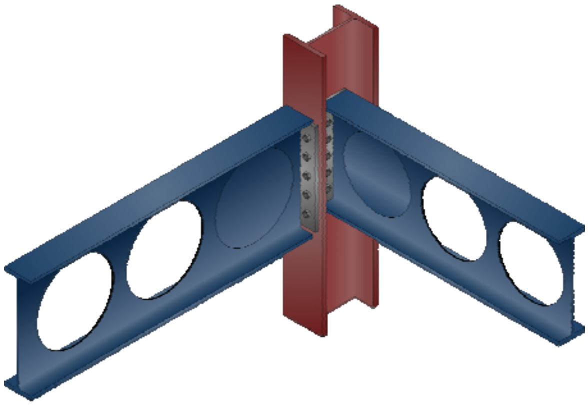


Figure 2-1 Example of a joint with flexible end plate connections

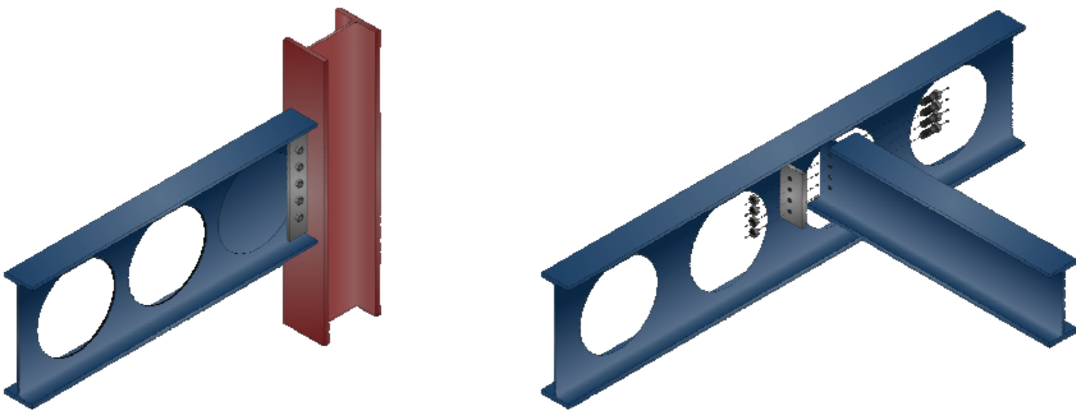


Figure 2-2 Examples of joints with fin plate connections

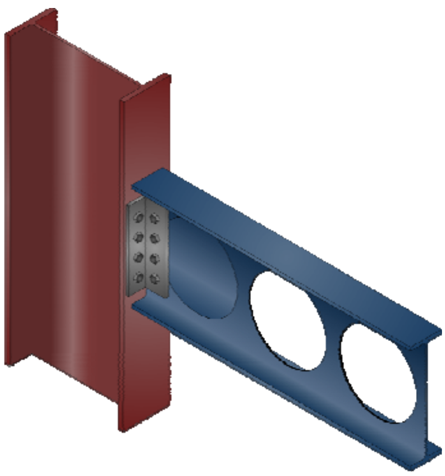


Figure 2-3 Example of a joint with a web cleat connection

2.2.2 Floor slabs and beams

The design recommendations given in this guide are applicable to profiled steel decking up to 80 mm deep with depths of concrete above the steel decking from 60 to 130 mm. The resistance of the steel decking is ignored in the fire design method but the presence of the steel decking prevents spalling of the concrete on the underside of the floor slab. This type of floor construction is illustrated in Figure 2-4.

The design method can be used with either isotropic or orthotropic reinforcing mesh, that is, meshes with either the same or different areas in orthogonal directions. The steel grade for the mesh reinforcement should be specified in accordance with EN 10080. The MACS+ software can only be used for welded mesh reinforcement and cannot consider more than one layer of reinforcement. Reinforcement bars in the ribs of the composite slab are **not** required**.**

The software includes A and B series standard fabric meshes as defined by UK national standards(11,12) (Table 2-1) and a range of mesh sizes defined by French national standards(13,14) (Table 2-2), and commonly used in the French construction market. User defined sizes of welded mesh are also permitted in the MACS+ software.

Table 2-1 Fabric mesh as defined by BS 4483(11)

Mesh Reference	Size of mesh (mm)	Weight (kg/m^2)	Longitudinal wires		Transverse wires	
			Size (mm)	Area (mm^2/m)	Size (mm)	Area (mm^2/m)
A142	200×200	2.22	6	142	6	142
A193	200×200	3.02	7	193	7	193
A252	200×200	3.95	8	252	8	252
A393	200×200	6.16	10	393	10	393
B196	100×200	3.05	5	196	7	193
B283	100×200	3.73	6	283	7	193
B385	100×200	4.53	7	385	7	193
B503	100×200	5.93	8	503	8	252

Table 2-2 Fabric mesh commonly used in French market

Mesh Reference	Size of mesh (mm)	Weight (kg/m^2)	Longitudinal wires		Transverse wires	
			Size (mm)	Area (mm^2/m)	Size (mm)	Area (mm^2/m)
ST 20	150×300	2.487	6	189	7	128
ST 25	150×300	3.020	7	257	7	128
ST 30	100×300	3.226	6	283	7	128
ST 35	100×300	6.16	7	385	7	128
ST 50	100×300	3.05	8	503	8	168
ST 60	100×300	3.73	9	636	9	254
ST 15 C	200×200	2.22	6	142	6	142
ST 25 C	150×150	4.03	7	257	7	257
ST 40 C	100×100	6.04	7	385	7	385
ST 50 C	100×100	7.90	8	503	8	503
ST 60 C	100×100	9.98	9	636	9	636

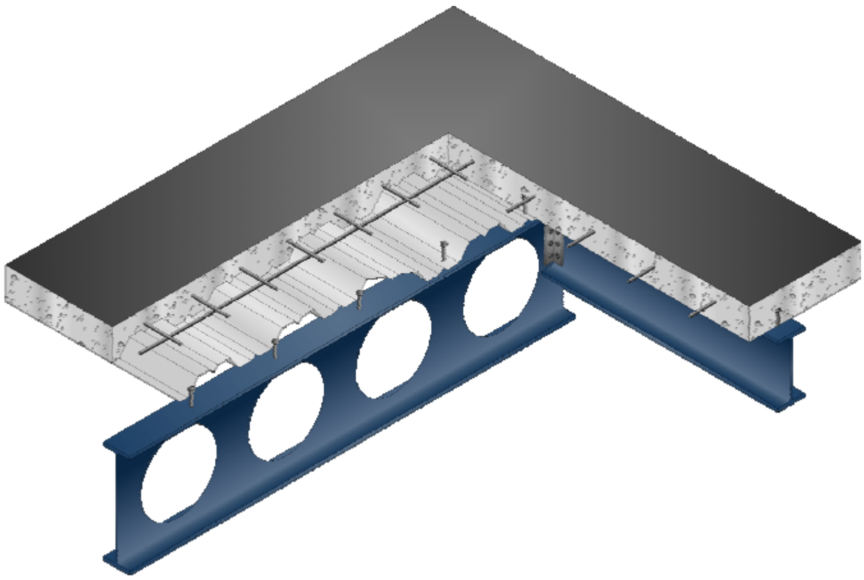


Figure 2-4 Cut away view of a typical composite floor construction

It is important to define the beam sizes used in the construction of the floor plate as this will influence the fire performance of the floor plate. The designer will need to have details of the serial size, steel grade and degree of shear connection available for each beam in the floor plate. The MACS+ software interface allows the user to choose from a predefined list of serial sizes covering common British, European and American I and H sections.

2.3 Floor design zones

The design method requires the designer to split the floor plate into a number of floor design zones as shown in Figure 2-5. The beams on the perimeter of these floor design zones must be designed to achieve the fire resistance required for the floor plate and will therefore normally be fire protected.

A floor design zone should meet the following criteria:

- Each zone should be rectangular.
- Each zone should be bounded on all sides by beams.
- The beams within a zone should only span in one direction.
- Columns should not be located within a floor design zone; they may be located on the perimeter of the floor design zone.
- For fire resistance periods in excess of 60 minutes, or when using the parametric temperature-time curve, all columns should be restrained by at least one fire protected beam in each orthogonal direction.

All internal beams within the zone may be left unprotected, provided that the fire resistance of the floor design zone is shown to be adequate using the MACS+ software. The size and spacing of these unprotected beams are not critical to the structural performance in fire conditions.

An example of a single floor design zone is given in Figure 2-5.

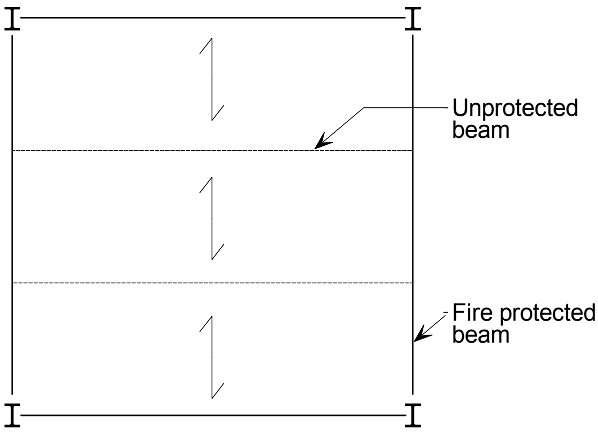


Figure 2-5 Example of a floor design zone

2.4 Combination of actions

The combination of actions for accidental design situations given in 6.4.3.3 and Table A1.3 of EN 1990 [15] should be used for fire limit state verifications. With only unfavourable permanent actions and no prestressing actions present, the combination of actions to consider is:

$$\sum G_{k,j,\text{sup}} + A_d + (\psi_{1,1} \text{ or } \psi_{2,1})Q_{k,1} + \sum \psi_{2,i} Q_{k,i}$$

with:

$G_{k,j,\text{sup}}$	unfavourable permanent action
A_d	leading accidental action
$Q_{k,1}$ and $Q_{k,i}$	accompanying variable actions, main and other respectively
$\psi_{1,1}$	factor for the frequent value of the leading variable action
$\psi_{2,i}$	factor for the quasi-permanent value of the i^{th} variable action

The use of either $\psi_{1,1}$ or $\psi_{2,1}$ with $Q_{k,1}$ should be specified in the relevant National Annex. The National Annex for the country where the building is to be constructed should be consulted to determine which factor to use.

The values used for the ψ factors relate to the category of the variable action they are applied to. The Eurocode recommended values for the ψ factors for buildings are given in Table A1.1 of EN 1990; those values are confirmed or modified by the relevant National Annex. The ψ factor values for buildings in the UK and France are summarised in Table 2-3. For floors that allow loads to be laterally distributed, the following uniformly distributed loads are given for moveable partitions in 6.3.1.2(8) of EN 1991-1-1[16]:

Movable partitions with a self-weight ≤ 1.0 kN/m wall length: $q_k = 0.5$ kN/m²

Movable partitions with a self-weight ≤ 2.0 kN/m wall length: $q_k = 0.8$ kN/m²

Movable partitions with a self-weight ≤ 3.0 kN/m wall length: $q_k = 1.2$ kN/m².

Movable partitions with self-weights greater than 3.0 kN/m length should be allowed for by considering their location.

The Eurocode recommended values for variable imposed loads on floors are given in Table 6.2 of EN 1991-1-1; those values may also be modified by the relevant National Annex. Table 2-4 presents the Eurocode recommended values and the values given in the UK and French National Annexes for the imposed load on an office floor.

Table 2-3 Values of ψ factors

Actions	Eurocode recommended values		UK National Annex values		French National Annex values	
	ψ_1	ψ_2	ψ_1	ψ_2	ψ_1	ψ_2
Domestic, office and traffic areas where: 30 kN < vehicle weight ≤ 160 kN	0.5	0.3	0.5	0.3	0.5	0.3
Storage areas	0.9	0.8	0.9	0.8	0.9	0.8
Other*	0.7	0.6	0.7	0.6	0.7	0.6

* Climatic actions are not included

Table 2-4 Imposed load on an office floor

Category of loaded area	Eurocode recommended values		UK National Annex values		French National Annex values	
	q_k (kN/m²)	Q_k (kN)	q_k (kN/m²)	Q_k (kN)	q_k (kN/m²)	Q_k (kN)
B – Office areas	3.0	4.5	2.5* or 3.0**	2.7	3.5 – 5.0	15.0

* Above ground floor level
**At or below ground floor level

2.5 Fire exposure

The recommendations given in the simple design method may be applied to buildings in which the structural elements are considered to be exposed to a standard temperature-time curve or parametric temperature-time curve, both as defined in EN 1991-1-2. Advanced model may also be used to define a temperature–time curve for a natural fire scenario. The resulting temperature-time time curve may be input to the MACS+ software in the form of a text file.

In all cases, the normal provisions of national regulations regarding means of escape should be followed.

2.5.1 Fire resistance

The recommended periods of fire resistance for elements of construction in various types of building in national regulations are given in Table 2-5 and Table 2-6.

The following recommendations are for buildings in which the elements of structure are required to have up to 180 minutes fire resistance. Provided that they are followed, composite steel framed buildings will maintain their stability for this period of fire resistance, when any compartment is subject to the standard temperature-time curve[(1)].

All composite steel framed buildings with composite floors may be considered to achieve 15 minutes fire resistance without fire protection, and so no specific recommendations are given in this case.

Table 2-5 Summary of fire resistance requirements from Approved Document B for England and Wales

	Fire resistance (mins) for height of top storey (m)			
	<5	**≤18**	**≤30**	**>30**
Residential (non-domestic)	30	60	90	120
Office	30	60	90	120*
Shops, commercial, assembly and recreation	30	60	90	120*
Closed car parks	30	60	90	120*
Open-sided car parks	15	15	15	60

Approved Document B allows the fire resistance periods to be reduced from 60 to 30 minutes or from 90 to 60 minutes, for most purpose groups.

* Sprinklers are required, but the fire resistance of the floor may be 90 minutes only.

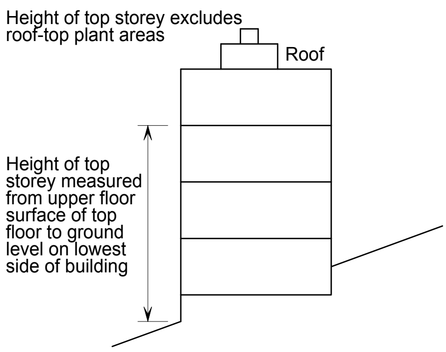


Table 2-6 Summary of fire resistance requirements from French Fire Regulations

<table>
<tr><td colspan="2" rowspan="2">Residential (non-domestic)</td><td>< 2 levels</td><td>2 levels < … ≤ 4 levels</td><td>4 levels < … ≤ 28 m</td><td>28 m < H < 50 m</td><td>> 50 m</td></tr>
<tr><td>R15</td><td>R30</td><td>R60</td><td>R90</td><td>R120</td></tr>
<tr><td colspan="2"></td><td colspan="2">Ground floor</td><td>Height of the top floor ≤ 8 m</td><td>Height of the top floor > 8 m</td><td>Height of the top floor > 28 m</td></tr>
<tr><td colspan="2">Office[1]</td><td colspan="3">0</td><td>R60</td><td>R 120</td></tr>
<tr><td rowspan="3">Shops, commercial, assembly and recreation</td><td>< 100 persons</td><td colspan="3">0</td><td>R60</td><td rowspan="3">R120</td></tr>
<tr><td>< 1500 persons</td><td colspan="3">R30</td><td>R60</td></tr>
<tr><td>> 1500 persons</td><td colspan="2">R30</td><td>R60</td><td>R90</td></tr>
<tr><td colspan="2"></td><td>Ground floor</td><td>> 2 levels</td><td colspan="3">Height of the top floor > 28 m</td></tr>
<tr><td colspan="2">Closed car parks</td><td rowspan="2">R30</td><td rowspan="2">R60</td><td colspan="3" rowspan="2">R90</td></tr>
<tr><td colspan="2">Open-sided car parks</td></tr>
</table>

Note: [1] Office which is not open to the public
H is the height of the top floor

2.5.2 Natural fire (parametric temperature-time curve)

The MACS+ software allows the effect of natural fire on the floor plate to be considered using the parametric temperature-time curve as defined in EN 1991-1-2 Annex A[(1)]. It should be noted that this is an Informative Annex and its use may not be permitted in some European countries, such as France. Before final design is undertaken the designer should consult the relevant National Annex.

Using this parametric fire curve, the software defines the compartment temperature taking account of:

- the compartment size:
  - compartment length
  - compartment width
  - compartment height
- the height and area of windows:
  - window height
  - window length
  - percentage open window
- the amount of combustibles and their distribution in the compartment:
  - fire load
  - combustion factor
  - the rate of burning
- the thermal properties of the compartment linings.

The temperature of a parametric fire will often rise more quickly than the standard fire in the early stages but, as the combustibles are consumed, the temperature will decrease rapidly. The standard fire steadily increases in temperature indefinitely.

The standard temperature-time curve and a typical parametric temperature-time curve are shown in Figure 2-6.

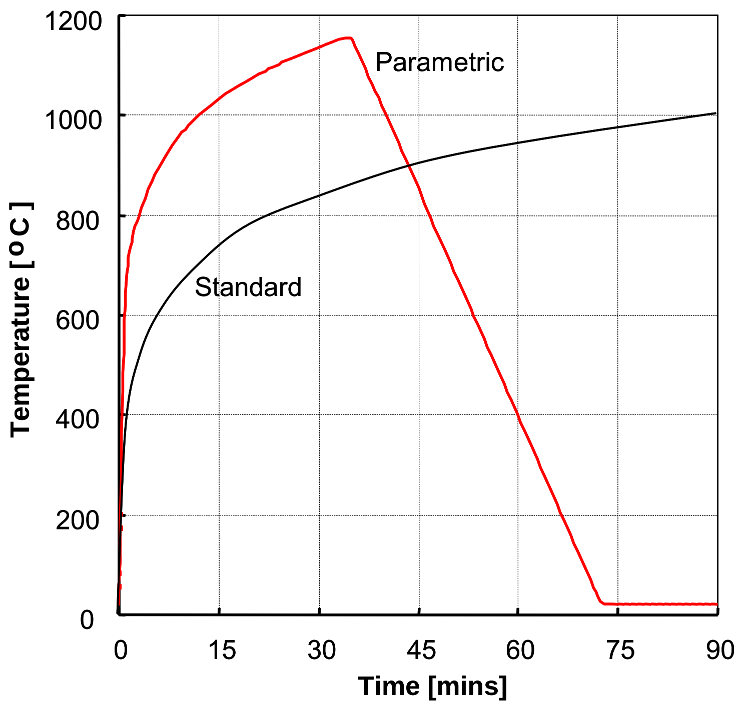


Figure 2-6 Comparison of typical parametric and standard temperature-time curve

3 RECOMMENDATIONS FOR STRUCTURAL ELEMENTS

3.1 Floor design zones

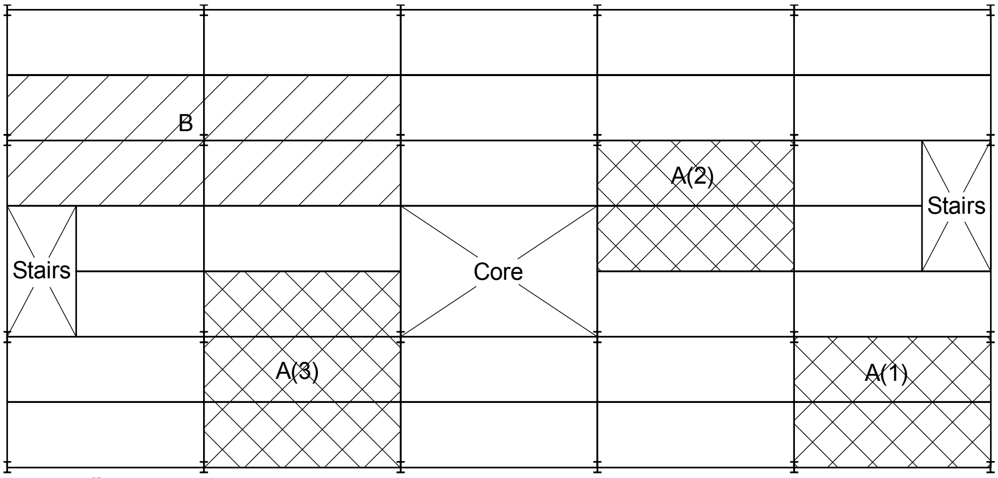
Each floor should be divided into design zones that meet the criteria given in Section 2.3.

The division of a floor into floor design zones is illustrated in Figure 3-1. Floor zones designated ‘A’ are within the scope of the MACS+ software and their load bearing performance in fire conditions may be determined using MACS+. The zone designated ‘B’ is outside the scope of the software because it contains a column and the beams within the zone do not all span in the same direction.

A single floor zone is illustrated in Figure 3-2 showing the beam span designations used in the MACS+ software. Normal design assumes that floor loads are supported by secondary beams which are themselves supported on primary beams.

The fire design method assumes that at the fire limit state, the resistance of the unprotected internal beams reduces significantly, leaving the composite slab as a two way spanning element simply supported around its perimeter. In order to ensure that the slab can develop membrane action, the MACS+ software computes the moment applied to each perimeter beam as a result of the actions on the floor design zone. To maintain the vertical support to the perimeter of the floor design zone in practice, the software calculates the degree of utilisation and hence the critical temperature of these perimeter beams. The fire protection for these beams should be designed on the basis of this critical temperature and the fire resistance period required for the floor plate in accordance with national regulations. The critical temperature and the degree of utilisation for each perimeter beam is reported for Side A to D of the floor design zone as shown by Figure 3-2.

As noted in Section 2.2.2, a restriction on the use of the MACS+ software is that for 60 minutes or more fire resistance, the zone boundaries should align with the column grid and the boundary beams should be fire protected. For 30 minutes fire resistance, this restriction does not apply and the zone boundaries do not have to align with the column grid. For example, in Table 3-3, zones A2 and A3 have columns at only two of their corners and could only be considered as design zones for a floor that requires no more than 30 minutes fire resistance.



Key to figure

A: These zones may be designed using MACS+

A(1) Any period of fire resistance

A(2) & A(3) only 30 minutes fire resistance

B: Outside the scope of MACS+

Figure 3-1 Possible floor design zones

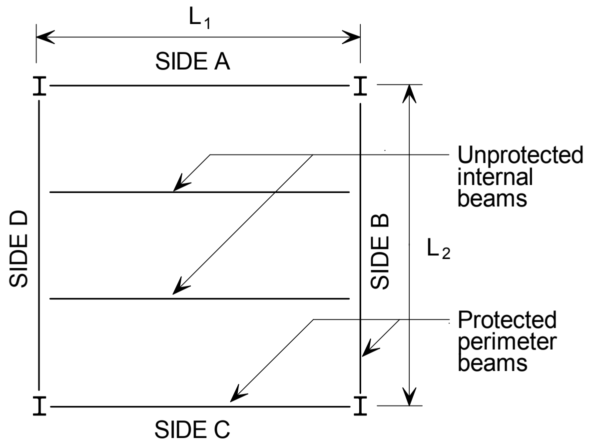


Figure 3-2 Definition of span 1 (L_1) and span 2 (L_2) and the beam layout for a floor design zone in a building requiring fire resistance of 60 minutes or more.

3.2 Floor slab and beams

The MACS+ software calculates the load bearing capacity of the floor slab and unprotected beams at the fire limit state. As the simple design method, implemented in the software, assumes that the slab will have adequate support on its perimeter the software also calculates the critical temperature for each perimeter beam based on the load bearing capacity of the floor design zone.

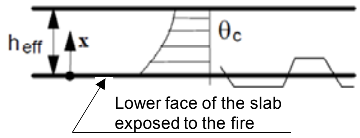
3.2.1 Temperature calculation of floor slab

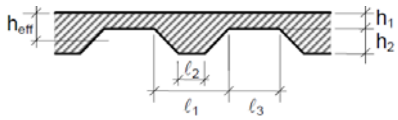
The temperature distribution in a composite slab can be determined using a calculation model by finite differences or finite elements taking into account the exact shape of the slab and respecting the principles and rules 4.4.2 of EN 1994-1-2 (6).

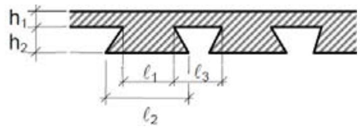
As an alternative, the temperature distribution in an unprotected composite slab subjected to standard fire can be determined from the values given in Table 3-1 established in accordance with EN 1992-1-2 (17) and its National Annex, depending on the effective thickness h_{eff} of the slab defined by D.4 of Annex D of EN1994-1-2 (6).

Table 3-1 Temperature distribution in a slab ($h_{eff, max}$ = 150mm) for standard fire exposure of 30 to 180 min

Distance x [mm]	Temperature in the concrete slab θ_c [°C]				
	30 min	60 min	90 min	120 min	180 min
2.5	675	831	912	967	1 042
10	513	684	777	842	932
20	363	531	629	698	797
30	260	418	514	583	685
40	187	331	423	491	591
50	135	263	349	415	514
60	101	209	290	352	448
70	76	166	241	300	392
80	59	133	200	256	344
90	46	108	166	218	303
100	37	89	138	186	267
110	31	73	117	159	236
120	27	61	100	137	209
130	24	51	86	119	186
140	23	44	74	105	166
150	22	38	65	94	149







$$\Phi = \frac{2}{\pi}\tan^{-1}\frac{2h_2}{\ell_1 + \ell_3 - \ell_2}$$

From the above temperature distribution, the three following parameters can be determined:

- θ_2 : temperature of the exposed face of the slab;
- θ_1 : temperature of the non-exposed face of the slab;
- θ_s : temperature of the slab at the level of the reinforcing mesh.

Under standard fire, the following values of x should be used to determine the temperatures θ_1, θ_2, and θ_s from Table 3-1:

- For θ_2, x = 2.5 mm;
- For θ_1, $x = h_{eff}$;

- For θ_s, $x = h_1 - d + 10\ \Phi$ (d: distance between the reinforcing mesh axis and the non-exposed face of the concrete, see Figure 3-3, and Φ: see Table 3-1).

3.2.2 Temperature calculation of unprotected composite beams

The temperatures of an unprotected steel beam under ISO fire can be determined in accordance with 4.3.4.2.2 of EN 1994-1-2. In order to facilitate the use of the calculation method, temperatures are given in Table 3-2 for unprotected steel cross-sections as a function of the resulting section factor (taken as the section factor multiplied by the correction factor for the shadow effect) and the fire exposure duration).

As an alternative, the temperature distribution in an unprotected composite slab subjected to standard fire can be determined from the values given in Table 3-1 established in accordance with EN 1992-1-2 (17) and its National Annex, depending on the effective thickness h_{eff} of the slab defined by D.4 of Annex D of EN1994-1-2 (6).

Table 3-2 Temperature of an unprotected steel cross-section under ISO fire

Resulting section factor $k_{sh}\left(\frac{A_i}{V_i}\right)$ [m^{-1}]	Temperature of the steel cross-section θ_a [°C]				
	30 min	60 min	90 min	120 min	180 min
20	432	736	942	1 030	1 101
30	555	835	987	1 039	1 104
40	637	901	995	1 042	1 106
50	691	923	997	1 043	1 106
60	722	931	999	1 044	1 107
70	734	934	1 000	1 045	1 107
80	742	936	1 001	1 046	1 108
90	754	937	1 001	1 046	1 108
100	768	938	1 002	1 046	1 108
110	782	939	1 002	1 047	1 108
120	793	939	1 003	1 047	1 108
130	802	940	1 003	1 047	1 109
140	810	940	1 003	1 047	1 109
150	815	941	1 003	1 047	1 109
200	829	942	1 004	1 048	1 109
500	838	944	1 005	1 048	1 109

3.2.3 Fire design of floor slab

Load bearing performance of the composite floor slab

When calculating the load bearing capacity of each floor design zone the resistance of the composite slab and the unprotected beams are calculated separately. The slab is assumed to have no continuity along the perimeter of the floor design zone. The load that can be supported by the flexural behaviour of the composite slab within the floor design zone is calculated based on a lower bound mechanism assuming a yield line pattern as shown in Figure 3-3.

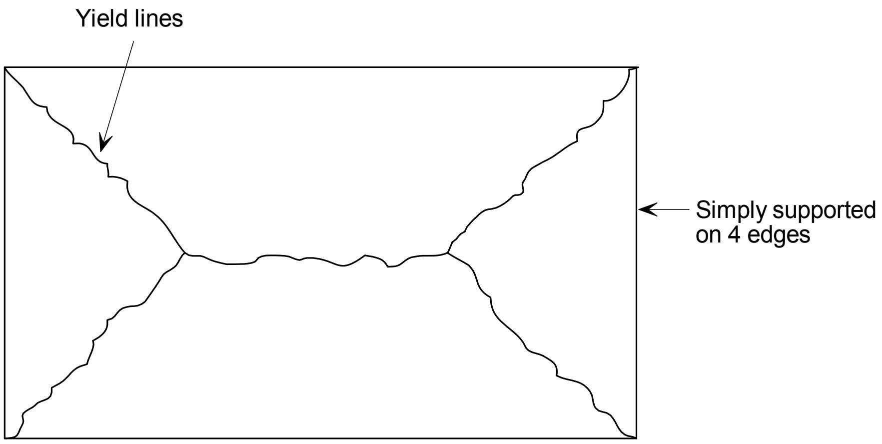


Figure 3-3 Assumed yield line pattern used to calculate slab resistance

The value of the resistance calculated using the lower bound mechanism is enhanced by considering the beneficial effect of tensile membrane action at large displacements. This enhancement increases with increasing vertical deflection of the slab until failure occurs due to fracture of the reinforcement across the short slab span or compressive failure of the concrete in the corners of the slab, as shown by Figure 3-4. As the design method cannot predict the point of failure, the value of deflection considered when calculating the enhancement is based on a conservative estimate of slab deflection that includes allowance for the thermal curvature of the slab and the strain in the reinforcement, as shown below.

$$w = \frac{\alpha(T_2 - T_1)l^2}{19.2h_{eff}} + \sqrt{\left(\frac{0.5f_y}{E_a}\right)\frac{3L^2}{8}}$$

The deflection allowed due to elongation of the reinforcement is also limited by the following expression.

$$w \le \frac{\alpha(T_2 - T_1)l^2}{19.2h_{eff}} + \frac{l}{30}$$

where:

$(T_2 - T_1)$ is the temperature difference between the top and bottom surface of the slab

L is the longer dimension of the floor design zone

l is the shorter dimension of the floor design zone

f_y is the yield strength of the mesh reinforcement

E is the modulus of elasticity of the steel

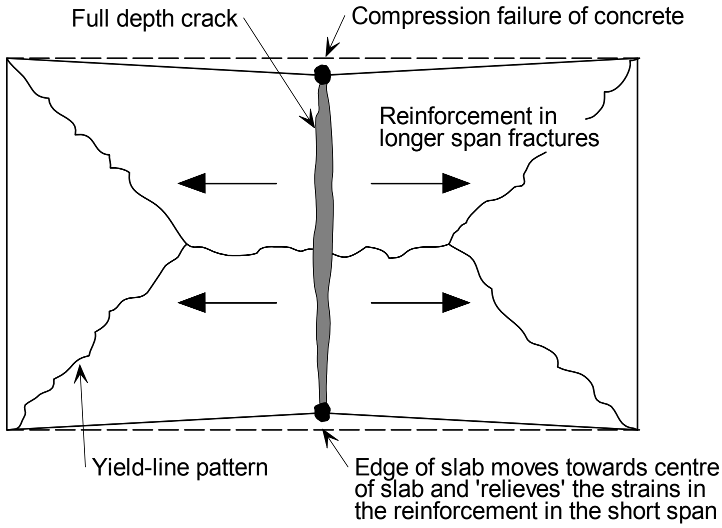
h_{eff} is the effective depth of the composite slab

α is the coefficient of thermal expansion of concrete.

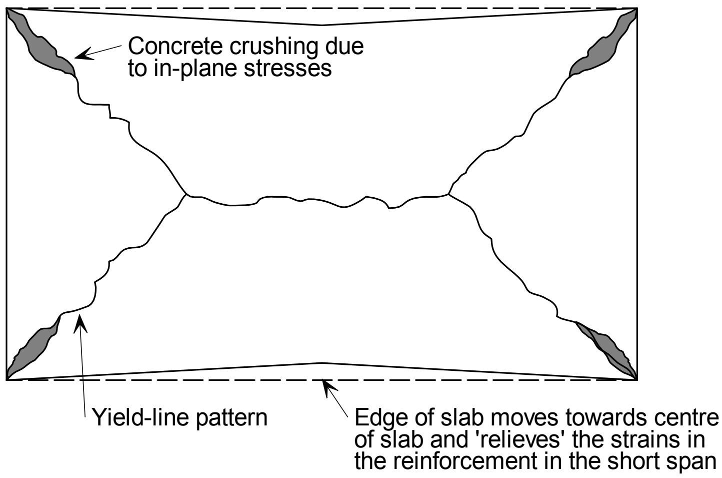
All of the available test evidence shows that this value of deflection will be exceeded before load bearing failure of the slab occurs. This implies that the resistance predicted using the design method will be conservative compared to its actual performance.

The overall deflection of the slab is also limited by the following expression:

$$w \leq \frac{L+l}{30}$$



(a) Tensile failure of the reinforcement



(b) Compressive failure of the concrete

Figure 3-4 Failure mode due to fracture of the reinforcement

The residual bending resistance of the unprotected composite beams is then added to the enhanced slab resistance to give the total resistance of the complete system.

Integrity and insulation performance of the composite slab

The MACS+ software does not explicitly check the insulation or integrity performance of the floor slab. The designer must therefore ensure that the slab thickness chosen is sufficient to provide the necessary insulation performance in accordance with the recommendations given in EN1994-1-2.

To ensure that the composite slab maintains its integrity during the fire and that membrane action can develop, care must be taken to ensure that the reinforcing mesh is properly lapped. This is especially important in the region of unprotected beams and around columns. Further information on required lap lengths and placement of the reinforcing mesh is given in Section 3.3.

3.2.4 Fire design of beams on the perimeter of the floor design zone

The beams along the perimeter of the floor design zone, labelled A to D in Figure 3-2, should achieve the fire resistance required for the floor plate, in order to provide the required vertical support to the perimeter of the floor design zone. This usually results in these beams being fire protected.

The MACS+ software calculates the design effect of actions on these perimeter beams and the room temperature moment of resistance of the beam, in order to calculate the degree of utilisation for each perimeter beam, which is calculated using the guidance given in EN 1993-1-2 §4.2.4, as shown below.

$$\mu_0 = \frac{E_{\mathrm{fi,d}}}{R_{\mathrm{fi,d,0}}}$$

where:

$E_{\mathrm{fi,d}}$ is the design effect of actions on the beam in fire

$R_{\mathrm{fi,d,0}}$ is the design resistance of the beam at time $t = 0$.

Having calculated the degree of utilisation, the software can compute the critical temperature of the bottom flange of the perimeter beams. This critical temperature is reported in the MACS+ software output for use when specifying the fire protection required by each of the perimeter beams on the floor design zone. Full details of the calculation method can be obtained from the MACS+ Background document[7].

For perimeter beams with floor design zones on both sides, the lower value of critical temperature given by the design of the adjacent floor design zones should be used to design the fire protection for that perimeter beam. The method of design for a perimeter beam that is shared by two floor design zones is illustrated in the work example, see Section 5.

When specifying fire protection for the perimeter beams, the fire protection supplier must be given the section factor for the member to be protected and the period of fire resistance required and the critical temperature of the member. Most reputable fire protection manufacturers will have a multi temperature assessment for their product which will have been assessed in accordance with EN 13381-4[17] for non-reactive materials or EN 13381-8[18] for reactive materials (intumescent). Design tables for fire protection which relate section factor to protection thickness are based on a single value of assessment temperature. This assessment temperature should be less than or equal to the critical temperature of the member.

3.3 Reinforcement details

The yield strength and ductility of the reinforcing steel material should be specified in accordance with the requirements of EN 10080. The characteristic yield strength of reinforcement to EN 10080 will be between 400 MPa and 600 MPa, depending on the national market.

In most countries, national standards for the specification of reinforcement may still exist as non-contradictory complimentary information (NCCI), as a common range of steel grades have not been agreed for EN 10080.

In composite slabs, the primary function of the mesh reinforcement is to control the cracking of the concrete. Therefore the mesh reinforcement tends to be located as close as possible to the surface of the concrete while maintaining the minimum depth of concrete cover required to provide adequate durability, in accordance with EN 1992-1-1[19]. In fire conditions, the position of the mesh will affect the mesh temperature and the lever arm when calculating the bending resistance. Typically, adequate fire performance is achieved with the mesh placed between 15 mm and 45 mm below the top surface of the concrete.

Section 3.3.1 gives general information regarding reinforcement details. Further guidance and information can be obtained from EN 1994-1-1[9] and EN 1994-1-2[6] or any national specifications such as those given in reference[20].

3.3.1 Detailing mesh reinforcement

Typically, sheets of mesh reinforcement are 4.8 m by 2.4 m and therefore must be lapped to achieve continuity of the reinforcement. Sufficient lap lengths must therefore be specified and adequate site control must be put in place to ensure that such details are implemented on site. Recommended lap lengths are given in section 8.7.5 of EN 1992-1-1[19] or can be in accordance with Table 3-3. The minimum lap length for mesh reinforcement should be 250 mm. Ideally, mesh should be specified with 'flying ends', as shown in Figure 3-5, to eliminate build up of bars at laps. It will often be economic to order 'ready fit fabric', to reduce wastage.

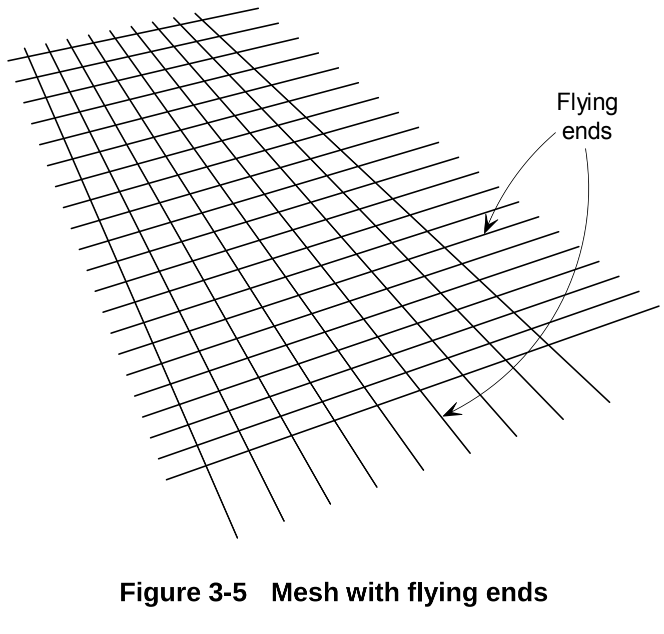


Figure 3-5 Mesh with flying ends

Table 3-3 Recommended tension laps and anchorage lengths for welded mesh

Reinforcement Type	Wire/Bar Type	Concrete class					
		LC 25/28	NC 25/30	LC 28/31	NC 28/35	LC 32/35	NC 32/40
Grade 500 Bar of diameter d	Ribbed	50d	40d	47d	38d	44d	35d
6 mm wires	Ribbed	300	250	300	250	275	250
7 mm wires	Ribbed	350	300	350	275	325	250
8 mm wires	Ribbed	400	325	400	325	350	300
10 mm wires	Ribbed	500	400	475	400	450	350

Notes:
These recommendations can be conservatively applied to design in accordance with EN 1992-1-1.
Where a lap occurs at the top of a section and the minimum cover is less than twice the size of the lapped reinforcement, the lap length should be increased by a factor of 1.4.
Ribbed Bars/Wires are defined in EN 10080.
The minimum Lap/Anchorage length for bars and fabric should be 300 mm and 250 mm respectively.

3.3.2 Detailing requirements for the edge of a composite floor slab

The detailing of reinforcement at the edge of the composite floor slab will have a significant effect on the performance of the edge beams and the floor slab in fire conditions. The following guidance is based on the best practice recommendations for the design and construction of composite floor slabs to meet the requirements for room temperature design. The fire design method and guidance presented in this document assumes that the composite floor is constructed in accordance with these recommendations.

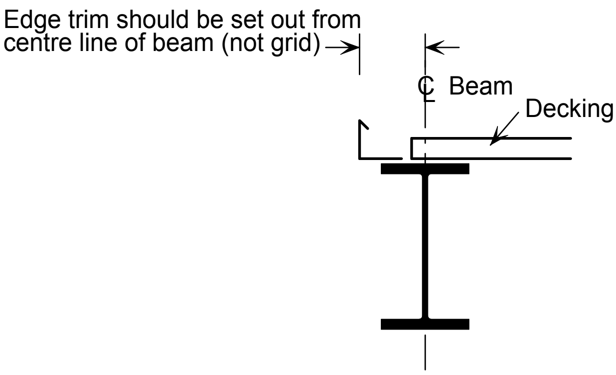


Figure 3-6 Setting out of edge trim

The edge of the composite slab is usually formed using ‘edge trims’ made from strips of light gauge galvanized steel fixed to the beam in the same way as the decking, as shown in Figure 3-6. In cases where the edge beam is designed to act compositely with the concrete slab, U shaped reinforcing bars are required to prevent longitudinal splitting of the concrete slab. These reinforcement bars also ensure that the edge beam is adequately anchored to the slab when using this simple design method.

Some typical slab edge details covering the two deck orientations are given in Figure 3-7. Where the decking ribs run transversely over the edge beam and cantilevers out a short distance, the edge trim can be fastened in the manner suggested in Figure 3-7 (a). The cantilever projection should be no more than 600 mm, depending on the depth of the slab and deck type used.

The more difficult case is where the decking ribs run parallel to the edge beam, and the finished slab is required to project a short distance, so making the longitudinal edge of the sheet unsupported Figure 3-7 (b). When the slab projection is more than approximately 200 mm (depending on the specific details), the edge trim should span between stub beams attached to the edge beam, as shown in Figure 3-7 (c). These stub beams are usually less than 3 m apart, and should be designed and specified by the structural designer as part of the steelwork package.

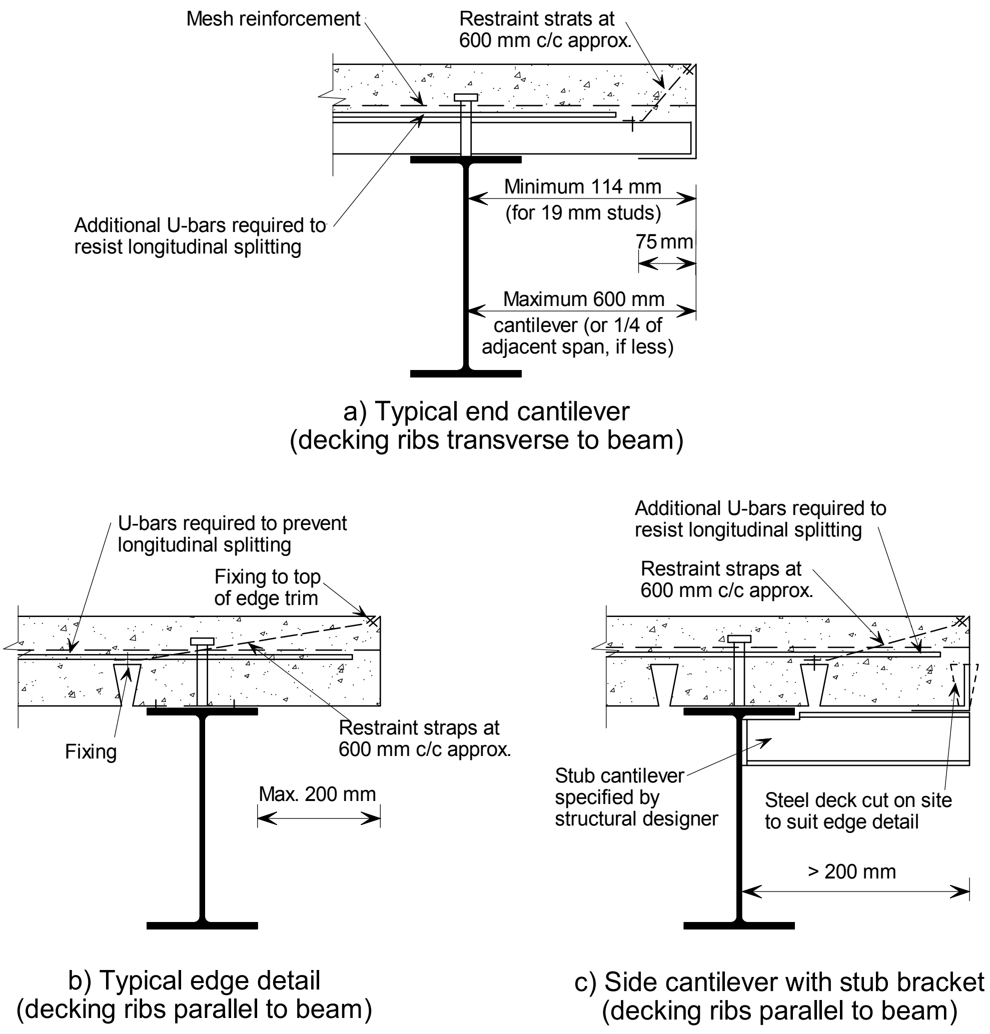


Figure 3-7 Typical edge details

3.4 Design of non composite edge beams

It is common practice for beams at the edge of floor slabs to be designed as non composite beams. This is because the costs of meeting the requirements for transverse shear reinforcement are more than the costs of installing a slightly heavier non composite beam. For fire design, it is important that the floor slab is adequately anchored to the edge beams, as these beams will be at the edge of floor design zones. Although not usually required for room temperature design of non composite edge beams, this guide recommends that shear connectors are provided at not more than 300 mm centres and U shaped reinforcing bars positioned around the shear connectors, as described in Section 3.3.2.

Edge beams often serve the dual function of supporting both the floors and the cladding. It is important that the deformation of edge beams should not affect the stability of cladding as it might increase the danger to fire fighters and others in the vicinity. This does not refer to the hazard from falling glass that results from thermal shock, which can only be addressed by use of special materials or sprinklers. Excessive deformation of the façade could increase the hazard, particularly when a building is tall and clad in masonry, by causing bricks to be dislodged.

3.5 Columns

The design guidance in this document is devised to confine structural damage and fire spread to the fire compartment itself. In order to achieve this, columns (other than those in the top storey) should be designed for the required period of fire resistance or designed to withstand the selected natural (parametric) fire.

In case of steel columns, any applied fire protection should extend over the full height of the column, including the connection zone (see Figure 3-8). This will ensure that no local squashing of the column occurs and that structural damage is confined to one floor.

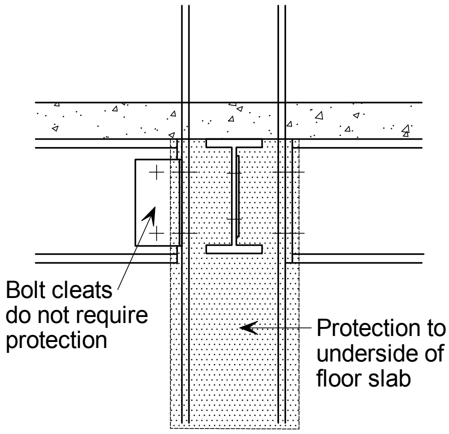


Figure 3-8 Extent of fire protection to columns

If steel and concrete composite columns are used, the fire protection applied to steel beams connected to these columns have to cover the connection zone of each column over a height corresponding to the maximum height of all connected steel beams. The thickness of fire protection should be the maximum one applied to all connected steel beams.

3.6 Joints

As stated in Section 2.2.1 the values given by the design method relate to 'simple' joints such as those with flexible end plates, fin plates and web cleats.

The steel frame building tested at Cardington contained flexible end plate and fin plate connections. Partial and full failures of some of the joints were observed during the cooling phase of the Cardington fire tests; however, no failure of the structure occurred as a result.

In the case where the plate was torn off the end of the beam, no collapse occurred because the floor slab transferred the shear to other load paths. This highlights the important role of the composite floor slab, which can be achieved with proper lapping of the reinforcement.

The resistances of the simple joints should be verified using the rules given in EN 1993-1-8[(23)].

3.6.1 Joint classification

Joint details should be such that they fulfill the assumptions made in the design model. Three joint classifications are given in EN 1993-1-8:

- nominally pinned
  - joints that transfer internal shear forces without transferring significant moments
- semi-rigid
  - joints that do not satisfy the nominally pinned nor the rigid joint criteria
- rigid
  - joints that provide full continuity.

EN 1993-1-8 §5.2 gives principles for the classification of joints based on their stiffness and strength; the rotation capacity (ductility) of the joint should also be considered.

As stated in Section 2.2.1 the values given by the simple design method have been prepared assuming the use of nominally pinned (simple) joints. To ensure that a joint does not transfer significant bending moments and so that it is a 'simple' joint it must have sufficient ductility to allow a degree of rotation. This can be achieved by detailing the joint such that it meets geometrical limits. Guidance on geometrical limits and initial sizing to ensure sufficient ductility of the joint is given in Access-steel documents[(25)].

3.6.2 End plates

There are two basic types of end plate connections; partial depth; and full depth. SN013 recommends the use of:

partial end plates when $V_{Ed} \leq 0.75\ V_{c,Rd}$

full depth end plates when $0.75\ V_{c,Rd} < V_{Ed} \leq V_{c,Rd}$

where:

V_{Ed} is the design shear force applied to the joint

$V_{c,Rd}$ is the design shear resistance of the supported beam.

The resistance of the components of the joint should be verified against the requirements given in EN 1993-1-8. For persistent and transient design situations the following design resistances need to be verified at ambient temperatures:

- supporting member in bearing

- end plate in shear (gross section)
- end plate in shear (net section)
- end plate in shear (block shear)
- end plate in bending
- beam web in shear*.

For completeness, all the design verifications given above should be carried out. However, in practice, for 'normal' joints, the verifications marked * will usually be critical. Guidance on meeting the requirements of EN 1993-1-8 is given in Access-steel documents(26).

EN 1993-1-8 does not give any guidance on design for tying resistance of end plates. Guidance is given in SN015(26) for the determination of the tying resistance of an end plate.

3.6.3 Fin plates

Single and double vertical lines of bolts may be used in fin plates. SN014(26) recommends the use of:

Single vertical lines of bolts when: $V_{\mathrm{Ed}} \leq 0.50\ V_{\mathrm{c,Rd}}$

Two vertical lines of bolts when: $0.50\ V_{\mathrm{c,Rd}} < V_{\mathrm{Ed}} \leq 0.75\ V_{\mathrm{c,Rd}}$

Use an end plate when: $0.75\ V_{\mathrm{c,Rd}} < V_{\mathrm{Ed}}$

where:

V_{Ed} is the design shear force applied to the joint

$V_{\mathrm{c,Rd}}$ is the design shear resistance of the supported beam.

For persistent and transient design situations, the following fin plate design resistances need to be verified at ambient temperature:

- bolts in shear*
- fin plate in bearing*
- fin plate in shear (gross section)
- fin plate in shear (net section)
- fin plate in shear (block shear)
- fin plate in bending
- fin plate in buckling (LTB)
- beam web in bearing*
- beam web in shear (gross section)
- beam web in shear (net section)
- beam web in shear (block shear)

- supporting element (punching shear) (this mode is not appropriate for fin plates connected to column flanges).

For completeness, all the design verifications given above should be carried out. However, in practice, for ‘normal’ joints, the verifications marked * will usually be critical. Guidance on meeting the requirements of EN 1993-1-8 is given in Access Steel documents[27].

As for end plates EN1993-1-8 does not give any guidance on design for tying resistance of fin plates. Therefore, alternative guidance such as that given in SN018[27] may be used to determine the tying resistance of a fin plate.

3.6.4 Web cleats

Although there were no cleated joints used in the Cardington frame, SCI has conducted a number of tests on composite and non-composite cleated joints in fire[28]. These joints consisted of two steel angles bolted to either side of the beam web using two bolts in each angle leg, then attached to the flange of the column also using two bolts. The joints were found to be rotationally ductile under fire conditions and large rotations occurred. This ductility was due to plastic hinges that formed in the leg of the angle adjacent to the column face. No failure of bolts occurred during the fire test. The composite cleated joint had a better performance in fire than the non-composite joint.

For non-composite web cleat joints it is recommended that single vertical lines of bolts should only be used when:

$V_{\mathrm{Ed}} \leq 0.50\ V_{\mathrm{c,Rd}}$

The design resistance of the cleated joint should be verified using the design rules given in Section 3 of EN 1993-1-8. Table 3.3 of EN 1993-1-8 gives the maximum and minimum values for the edge, end and spacing distances that should be met when detailing the position of bolts.

3.6.5 Fire protection

In cases where both structural elements to be connected are fire protected, the protection appropriate to each element should be applied to the parts of the plates or angles in contact with that element. If only one element requires fire protection, the plates or angles in contact with the unprotected elements may be left unprotected.

3.7 Overall building stability

In order to avoid sway collapse, the building should be braced by shear walls or other bracing systems. Masonry or reinforced concrete shear walls should be constructed with the appropriate fire resistance.

If bracing plays a major part in maintaining the overall stability of the building it should be protected to the appropriate standard.

In two-storey buildings, it may be possible to ensure overall stability without requiring fire resistance for all parts of the bracing system. In taller buildings, all parts of the bracing system should be appropriately fire protected.

One way in which fire resistance can be achieved without applied protection is to locate the bracing system in a protected shaft such as a stairwell, lift shaft or service core. It is important that the walls enclosing such shafts have adequate fire resistance to prevent the spread of any fire. Steel beams, columns and bracing totally contained within the shaft may be unprotected. Other steelwork supporting the walls of such shafts should have the appropriate fire resistance.

4 COMPARTMENTATION

National regulations require that compartment walls separating one fire compartment from another shall have stability, integrity and insulation for the required fire resistance period.

Stability is the ability of a wall not to collapse. For load bearing walls, the load bearing capacity must be maintained.

Integrity is the ability to resist the penetration of flames and hot gases.

Insulation is the ability to resist excessive transfer of heat from the side exposed to fire to the unexposed side.

4.1 Beams above fire resistant walls

When a beam is part of a fire resisting wall, the combined wall/beam separating element must have adequate insulation and integrity as well as stability. For optimum fire performance, compartment walls should, whenever possible, be located beneath and in line with beams.

Beams in the wall plane

The Cardington tests demonstrated that unprotected beams above and in the same plane as separating walls (see Figure 4-1), which are heated from one side only, do not deflect to a degree that would compromise compartment integrity, and normal movement allowances are sufficient. Insulation requirements must be fulfilled and protection for 30 or 60 minutes will be necessary; all voids and service penetrations must be fire stopped. Beams protected with intumescent coatings require additional insulation because the temperature on the non fire side is likely to exceed the limits required in the fire resistance testing standards[29,30].

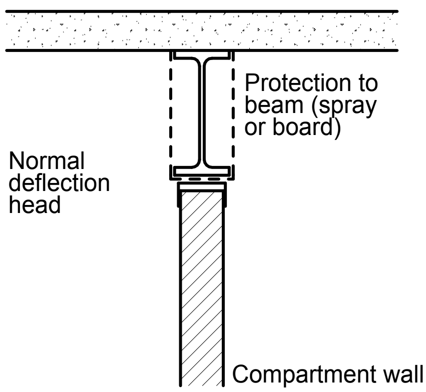


Figure 4-1 Beams above and in line with walls

Beams through walls

The Cardington tests showed that floor stability can be maintained even when unprotected beams suffer large deflections. However, when walls are located off the column grid, large deflections of unprotected beams can compromise integrity by displacing or cracking the walls through which they pass. In such cases, the beams should either be protected or sufficient movement allowance provided. It is recommended that a deflection allowance of span/30 should be provided in walls crossing the middle half of an unprotected beam. For walls crossing the end quarters of the beam, this allowance may be reduced linearly to zero at end supports (see Figure 4-2). The compartment wall should extend to the underside of the floor.

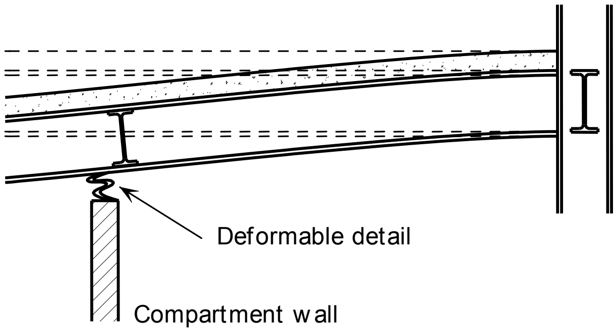


Figure 4-2 Deformation of beams crossing walls

4.2 Stability

Walls that divide a storey into more than one fire compartment must be designed to accommodate expected structural movements without collapse (stability). Where beams span above and in the plane of the wall, movements, even of unprotected beams, may be small and the normal allowance for deflection should be adequate. If a wall is not located at a beam position, the floor deflection that the wall will be required to accommodate may be large. It is therefore recommended that fire compartment walls should be located at a beam positions whenever possible.

In some cases, the deflection allowance may be in the form of a sliding joint. In other cases, the potential deflection may be too large and some form of deformable blanket or curtain may be required, as illustrated in Figure 4-2.

National recommendations should be consulted for the structural deformations which should be considered when ensuring that compartmentation is maintained.

4.3 Integrity and insulation

Steel beams above fire compartment walls are part of the wall and are required to have the same separating characteristics as the wall. A steel beam without penetrations will have integrity. However, any service penetrations must be properly fire stopped and all voids above composite beams should also be fire stopped.

An unprotected beam in the plane of a compartment wall may not have the required insulation and will normally require applied fire protection. It is recommended that all beams at compartment boundaries should be fire protected, as shown in Figure 4-1.

5 WORKED EXAMPLE

In order to illustrate the application of the output from the MACS+ software, this Section contains a worked example based on a realistic composite floor plate and composite floor plate with cellular beams.

The building considered is a 4 storey steel framed office building. The building requires 60 minutes fire resistance for a given National Building Regulation.

The floor plate for each storey consists of a composite floor slab constructed using Cofraplus 60 trapezoidal metal decking, normal weight concrete and a single layer of mesh reinforcement. The slab spans between 9 m long secondary beams designed to act compositely with the floor slab. These secondary beams are also in turn supported on composite primary beams of 9 m and 12 m spans. The beams on the edge of the building are designed as non-composite in accordance with EN 1993-1-1. Some of the internal beams (part 1 to 2) are plain composite profiles and beams located in part 2 to 3 are composite cellular beams.

The construction of the floor plate is shown in Figure 5-3 to Figure 5-6.

Figure 5-3 shows the general arrangement of steelwork at floor level across the full width of the building and two bays along its length. It is assumed that this general arrangement is repeated in adjoining bays along the length of the building. The columns are HD320×158, designed as non-composite columns in accordance with EN 1993-1-1.

The floor loading considered was as follows:

- variable action due to occupancy: 4 kN/m^2
- variable action due to light weight partitions: 1 kN/m^2
- permanent action due to ceilings and services: 0.7 kN/m^2
- self weight of beam: 0.5 kN/m^2

For the edge beams, an additional cladding load of 2 kN/m was considered in the design.

The beam sizes required to fulfil the normal stage checks for these values of actions are shown in Figure 5-3. The internal beams are composite and the degree of shear connection for each beam is shown in Table 5-1.

Figure 5-4 shows a cross section through the composite slab. The slab is C25/30 normal weight concrete with an overall thickness of 130 mm. The slab is reinforced with ST 15C mesh reinforcement with a yield strength of 500 MPa, this meets the requirements for normal temperature design but the mesh size may need to be increased in size if the performance in fire conditions is inadequate.

The floor Zone E has been designed using Composite Cellular beams with circular openings made from a hot rolled IPE 300 in S355 (see Figure 5-1 hereafter).

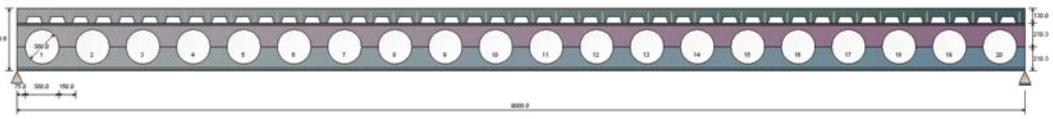


Figure 5-1 Geometry of the Cellular Beam composite section

The floor Zone D and F have been designed using Composite Angelina^{TM} beams with sinusoidal openings made from a hot rolled IPE 270 in S355 (see Figure 5-2 hereafter).

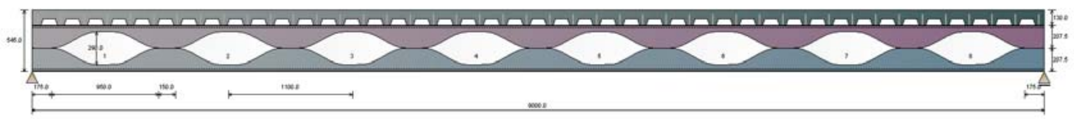


Figure 5-2 Geometry of the ANGELINA^{TM} beam composite section

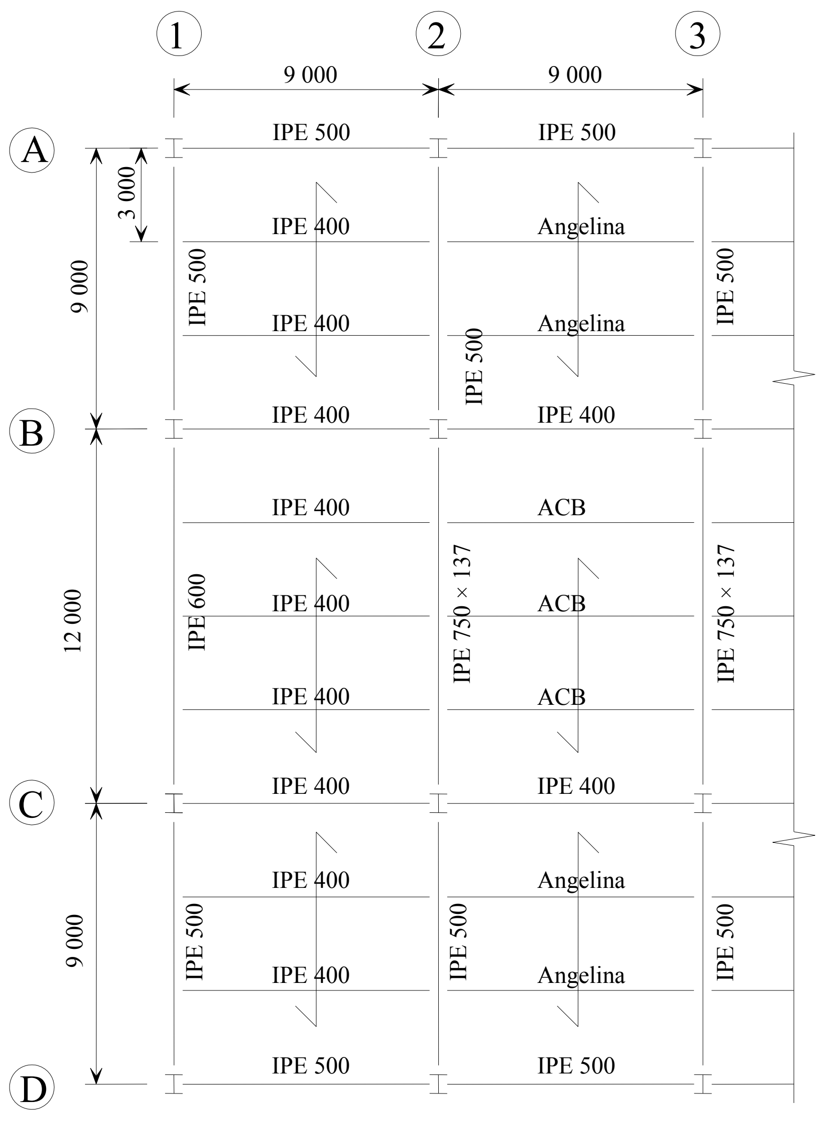


Figure 5-3 General arrangement of steelwork at floor level

Table 5-1 Beam details

Beam Section (S355)	Location of beam	Construction Type	Degree of Shear Connection (%)	Number of shear studs per group and spacing
IPE 400	Secondary internal beam	Composite	51	1 @ 207mm
IPE 500	Secondary edge beam	Non composite	-	
IPE 500	Primary internal beam	Composite	72	2 @ 207mm
IPE 750 × 137	Primary internal beam	Composite	71	2 @ 207 mm
IPE 600	Primary edge beam	Non composite	-	
ACB IPE 300+IPE 300	Secondary internal beam	Composite	52	2 @ 207 mm
Angelina IPE270 + IPE 270	Secondary internal beam	Composite	52	2 @ 207 mm

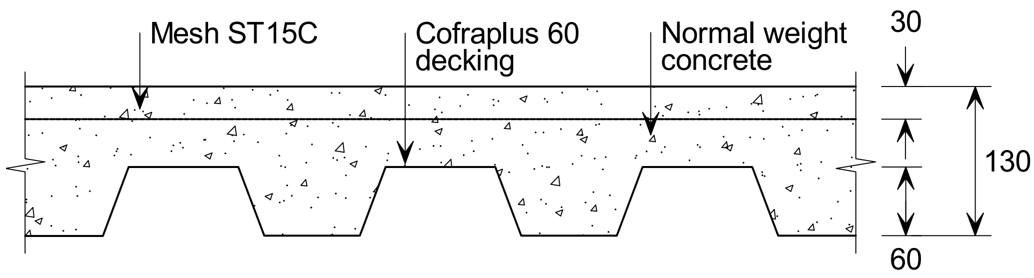
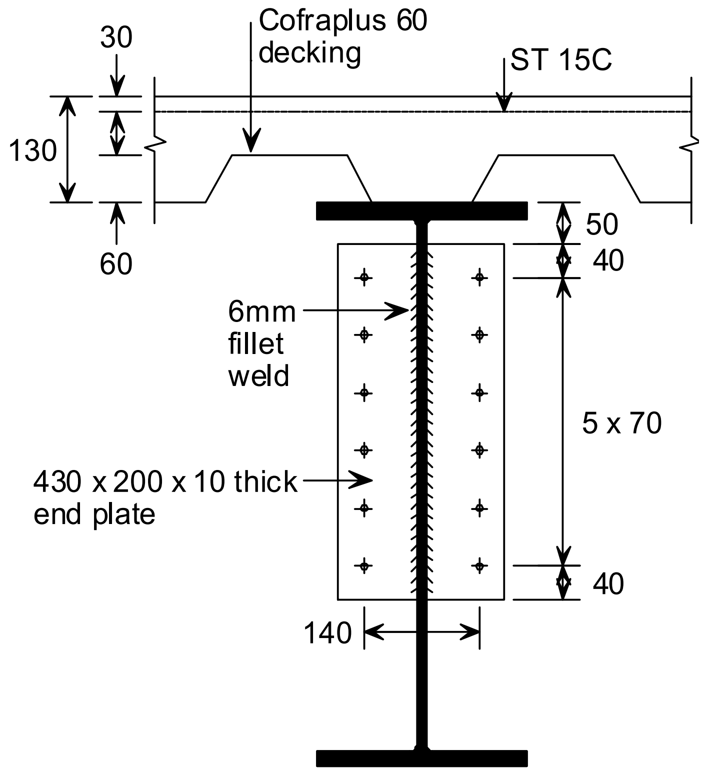
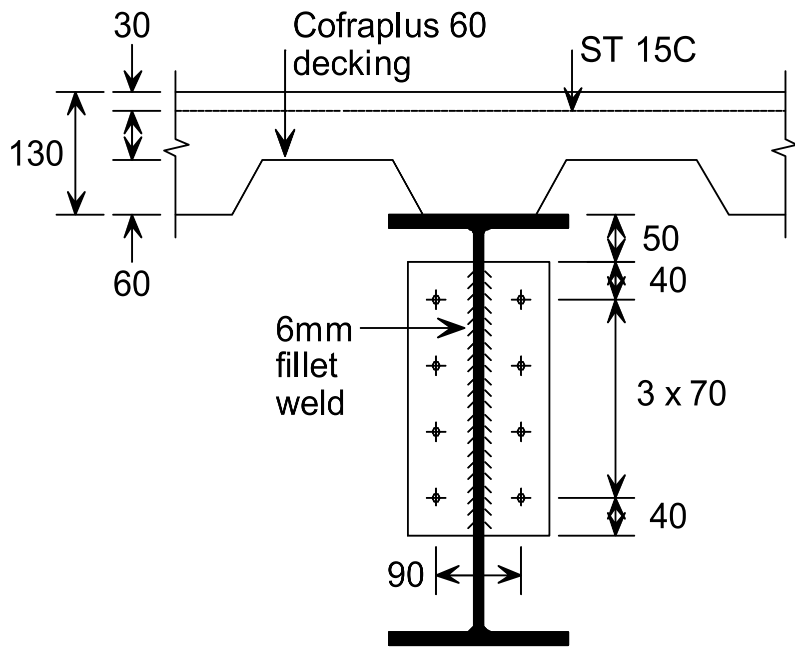


Figure 5-4 Construction of floor slab

All joints between the main steelwork elements use flexible end plate details and are designed as nominally pinned in accordance with EN 1993-1-8. Figure 5-5(a) shows the joint used between the primary beams and the columns. The beam-to-column joints for secondary beams are as shown in Figure 5-5(b). Figure 5-6 shows the endplate connection between the secondary beams and the primary beams.



(a) Primary beam-to-column joint



(b) Secondary beam to column joint

Figure 5-5 Beam-to-column joints

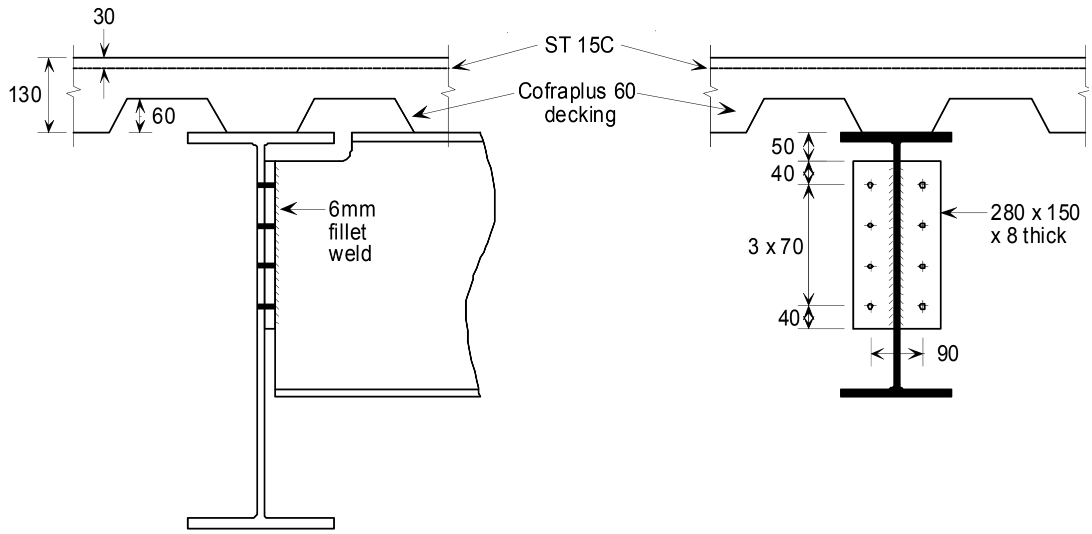


Figure 5-6 Secondary beam to primary beam connection

Figure 5-7 shows the floor plate divided into floor design zones. It is likely that floor design zones A and B will give the most onerous design conditions. The design of both of these zones will be considered.

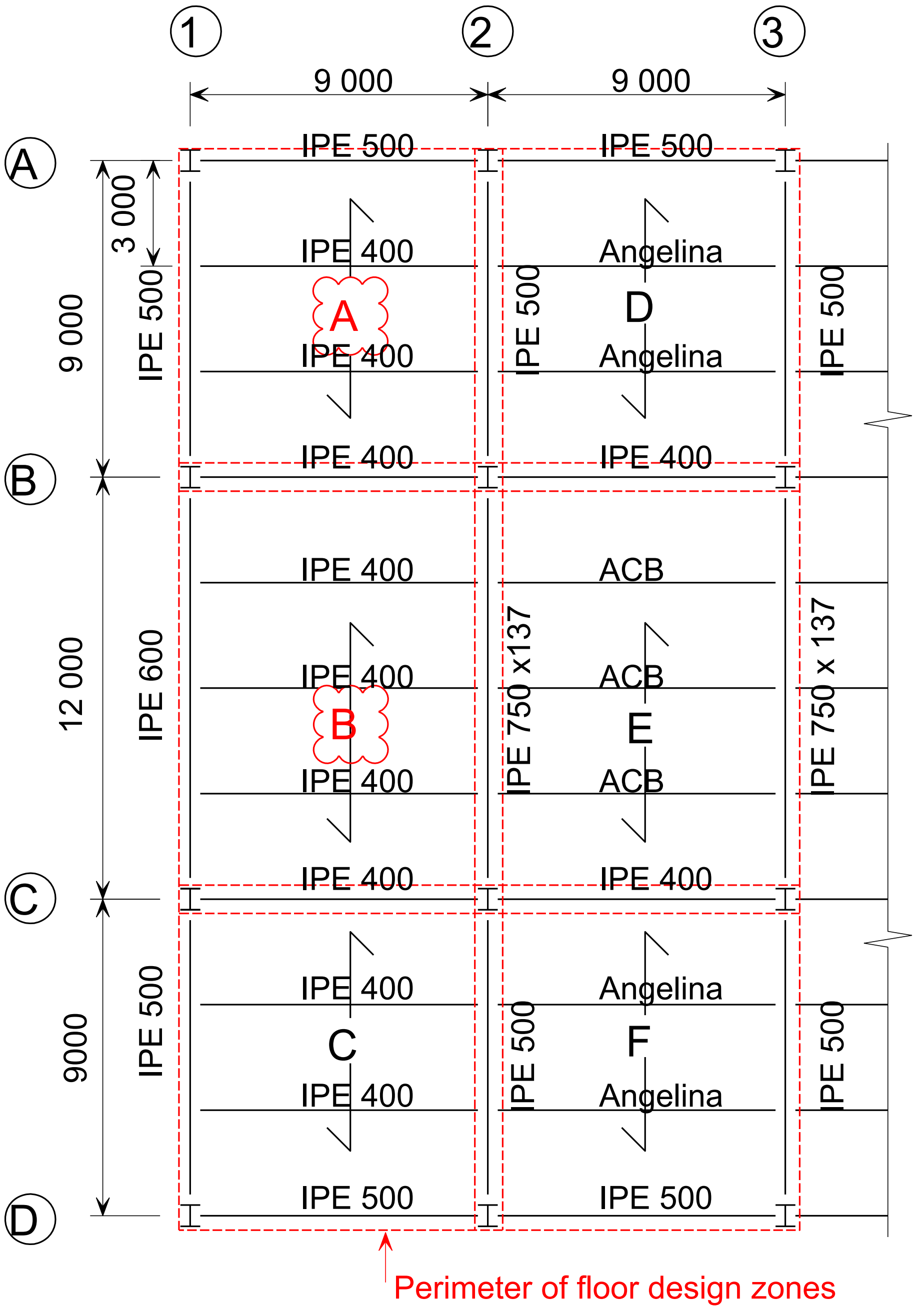


Figure 5-7 Floor design zones (A – F)

5.1 Design of composite slab in fire conditions

The following design checks carried out on the floor design zones are based on the floor construction required for room temperature design checks. If this construction proves to be inadequate for fire conditions then the mesh size and/or the floor depth will be increased to improve the performance in fire conditions. As the design Zone B seems more critical than design Zone A due to its lager span, we run the program with design Zone B first.

5.1.1 Floor design: Zone B

Table 5-2 shows the input data for floor design Zone B, which is 9 m by 12 m with the mesh size of ST 15C. Within this floor design zone, there are 3 unprotected composite beams.

Table 5-2 Input data for floor design Zone B

L (mm)	ℓ (mm)	f_c (MPa)	A_s (mm²/m)	f_{sy} (MPa)	Unprotected beams	Steel decking	Total thickness of the slab (mm)	d: mesh axis distance (mm)
12 000	9 000	25	142	500	IPE400	Cofraplus60	130	30

Figure 5-8 to Figure 5-11 show the same information in the input windows of the MACS+ Software.

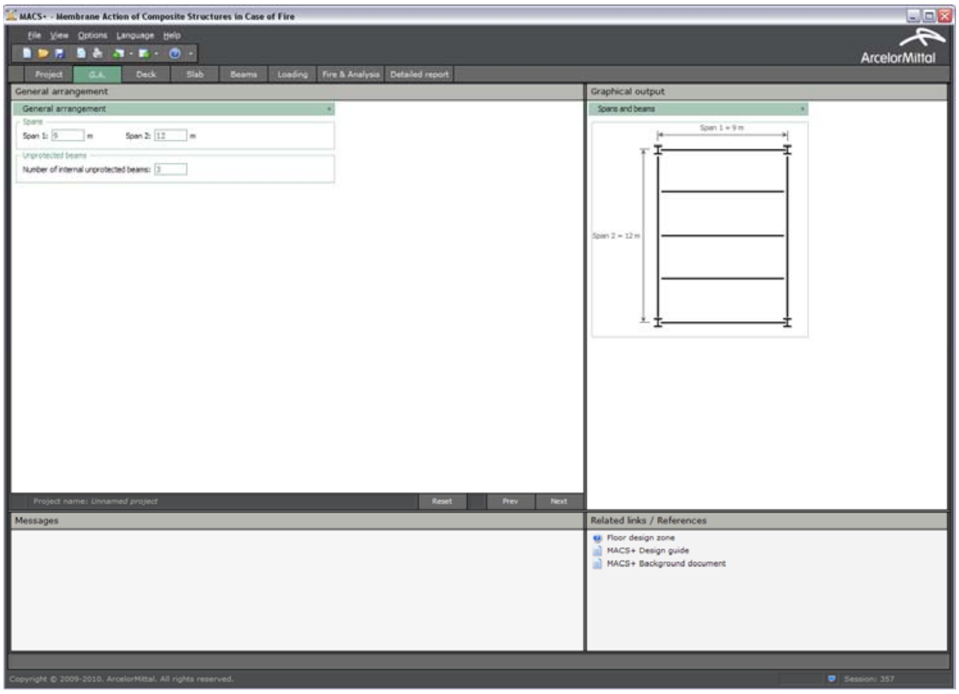


Figure 5-8 Input data using the MACS+ software – General arrangement

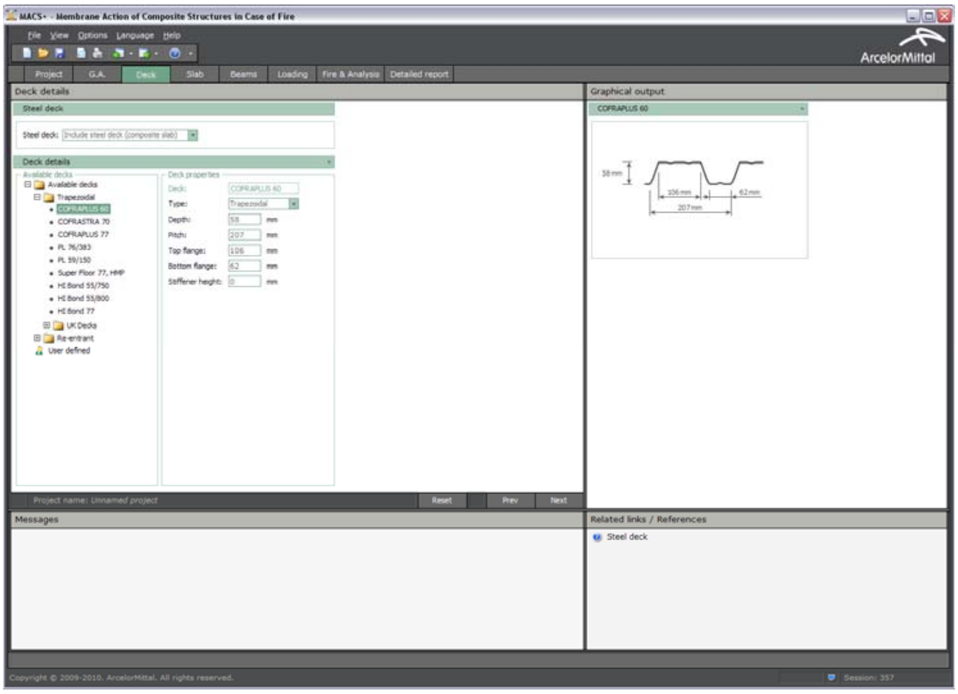


Figure 5-9 Input data using the MACS+ software - Deck

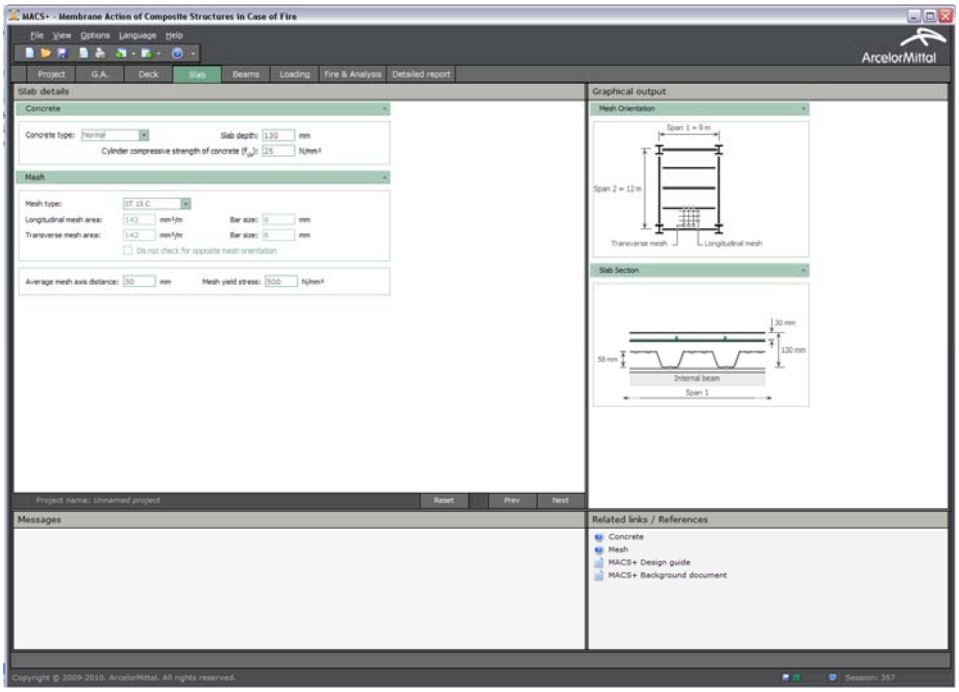


Figure 5-10 Input data using the MACS+ software - Slab

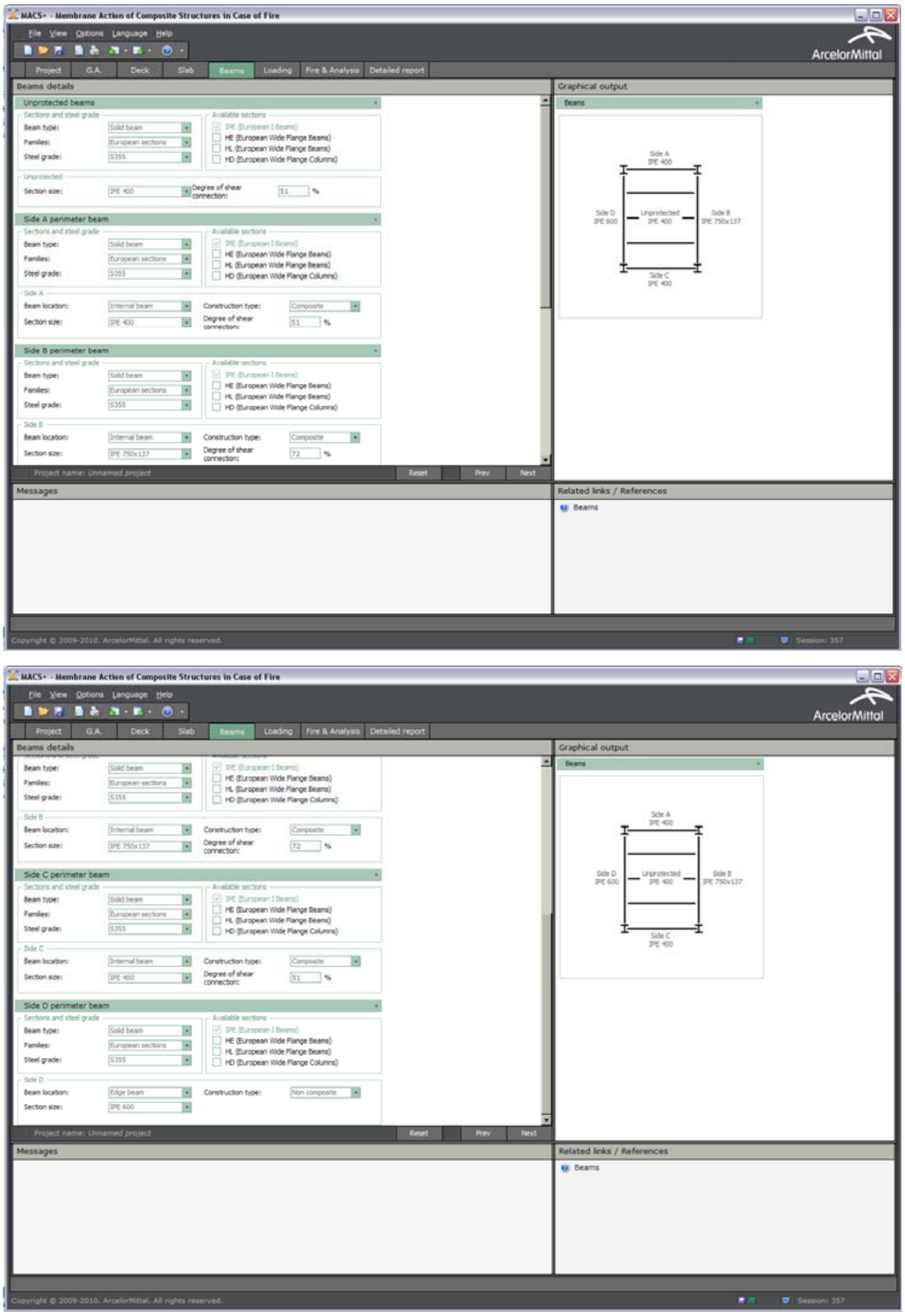


Figure 5-11 Input data using the MACS+ software – Beams in Zone B

The application of the simplified model is done in several steps as followed:

Step 1: Calculation of the applied load on the slab in case of fire

The applied load on the slab in case of fire with a self weight of 2.28 kN/m² for the slab can be determined by:

$$q_{fi,Sd} = G + 0.5Q = (2.28 + 0.7 + 0.5) + 0.5 \times (4.0 + 1.0) = 5.98 \text{ kN/m}^2$$

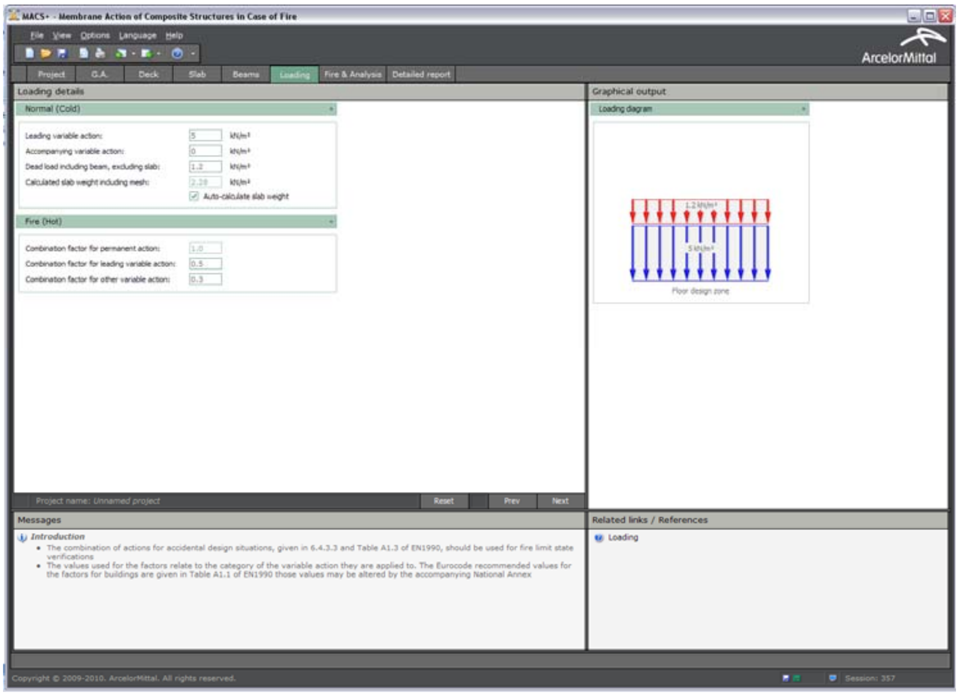


Figure 5-12 Input data using the MACS+ software - Loading

Step 2: Calculation of the heat transfer into the composite slab Cofraplus 60

From the relation D.15a of the Annex D of the EN 1994-1-2[16], the effective thickness of the slab can be expressed by:

$$h_{eff} = h_1 + 0.5\, h_2\left(\frac{\ell_1 + \ell_2}{\ell_1 + \ell_3}\right) = 72 + 0.5 \times 58 \times \left(\frac{101+62}{101+106}\right) \approx 95 \text{ mm}$$

This effective thickness allows to verify that the slab fulfill the criteria EI60 which request an effective thickness with creed of minimum 80 mm for the composite slab.

Moreover, this effective thickness leads to the following temperatures θ_1, θ_2 and θ_s (see Table 3-1). For a time exposure of 60 minutes to normalized fire:

θ_1 = 99 °C; θ_2 = 831 °C and θ_s = 288 °C.

Following Table 3-4 of EN 1994-1-2, there is no reduction of the effective steel strength for the welded steel mesh:

$$f_{sy,\theta_s} = 500 \text{ MPa}$$

$$\gamma_{M,fi,s} = 1.0$$

Moreover, there is also:

$$\gamma_{M,fi,c} = 1.0$$

Step 3: Calculation of the moment resistance of the slab section $M_{fi,0}$

For this calculation zone:

L_1 = 9 000 mm (span of the secondary beams)

L_2 = 12 000 mm (span of the primary beams)

So, $L = \max\{L_1; L_2\}$ = 12 000 mm and $\ell = \min\{L_1; L_2\}$ = 9 000 mm.

It can be obtained:

$$(g_0)_1 = 1 - \frac{2KA_s\, f_{sy,\theta_s}/\gamma_{M,fi,s}}{0.85\, f_c/\gamma_{M,fi,c}\, d} = 1 - \frac{2\times 1.0\times\frac{142}{1000}\times 500/1.0}{0.85\times 25/1.0\times 30} = 0.777$$

$$(g_0)_2 = 1 - \frac{2A_s\, f_{sy,\theta_s}/\gamma_{M,fi,s}}{0.85\, f_c/\gamma_{M,fi,c}\, d} = 1 - \frac{2\times 1.0\times\frac{142}{1000}\times 500/1.0}{0.85\times 25/1.0\times 30} = 0.777$$

It is to be noticed that the parameter K is equal to 1.0 because the reinforcing mesh has the same cross section in both dimensions.

So, the positive moment resistance of the slab section is:

$$M_{fi,0} = A_s\, f_{sy,\theta_s}/\gamma_{M,fi,s}\, d\,\frac{3+(g_0)_2}{4} = \frac{142}{1000}\times 500/1.0\times 30\times\frac{3+0.777}{4} = 2\,011.4 \text{ Nmm/mm}$$

In parallel, it is also possible to determine the other necessary parameters:

$$\mu = K\frac{3+(g_0)_1}{3+(g_0)_2} = 1.0\times\frac{3+0.777}{3+0.777} = 1.0$$

$$a = \frac{L}{\ell} = \frac{12\,000}{9\,000} = 1.333$$

$$n = \frac{1}{2\mu a^2}\left(\sqrt{3\mu a^2+1}-1\right) = \frac{1}{2\times 1.0\times 1.333^2}\times\left(\sqrt{3\times 1.0\times 1.333^2+1}-1\right) = 0.427$$

Step 4: Determination of the reference bearing capacity of the slab

The reference bearing capacity of the slab can be determined from:

$$p_{fi} = 6\frac{M_{fi,0}}{n^2 a^2 \ell^2} = 6\times\frac{2\,011.4}{0.427^2\times 1.333^2\times 9\,000^2} = 0.461\times 10^{-3}\text{ N/mm}^2 = 0.461\text{ kN/m}^2$$

Step 5: Determination of the deflection for the calculation of the membrane action

The deflection of the slab in fire situation to take into account membrane action can be obtained from:

$$w = \min\left\{\frac{\alpha(\theta_2-\theta_1)\ell^2}{19.2h_{eff}} + \min\left[\sqrt{\left(\frac{0.5f_{sy}}{E_a\gamma_{M,fi,s}}\right)\frac{3L^2}{8}};\ \frac{\ell}{30}\right];\ \frac{L+\ell}{30}\right\}$$

$$= \min\left\{\frac{1.2\times 10^{-5}(831-99)\times 9\,000^2}{19.2\times 95} + \min\left[\sqrt{\left(\frac{0.5\times 500}{210\,000\times 1.0}\right)\frac{3\times 12\,000^2}{8}};\frac{9\,000}{30}\right];\frac{12\,000+9\,000}{30}\right\}$$

$$= \min\{391.0+\min[253.5\,;300];700\} = 644.6\text{ mm}$$

Step 6: Calculation of the parameters to determine the membrane action

The determination of the different multiplication factors for the membrane action are based on the different parameters α_1, α_2, β_1, β_2, A, B, C, D, k and b that need to be determined. The values of theses parameters are summarized in Table 5-3.

Table 5-3 Parameters used for the assessment of the membrane action in Zone B

Equation	Obtained value
$\alpha_1 = \frac{2(g_0)_1}{3+(g_0)_1}$	0.412
$\beta_1 = \frac{1-(g_0)_1}{3+(g_0)_1}$	0.059
$\alpha_2 = \frac{2(g_0)_2}{3+(g_0)_2}$	0.412
$\beta_2 = \frac{1-(g_0)_2}{3+(g_0)_2}$	0.059
$k = \frac{4na^2(1-2n)}{4n^2a^2+1}+1$	1.194
$A = \frac{1}{2(1+k)}\left[\frac{\ell^2}{8n} - \left(\frac{1-2n}{2n} + \frac{1}{3(1+k)}\right)\left((nL)^2 + (\ell/2)^2\right)\right]$	1 978 359 mm^2
$B = \frac{k^2}{2(1+k)}\left[\frac{nL^2}{2} - \frac{k}{3(1+k)}\left((nL)^2 + (\ell/2)^2\right)\right]$	7 242 376 mm^2
$C = \frac{\ell^2}{16n}(k-1)$	2 305 602 mm^2
$D = \frac{L^2}{8}(1-2n)^2$	388 465 mm^2
$b = \min\left[\begin{array}{l}\frac{\ell^2}{8K(A+B+C-D)}, \\ \frac{\gamma_{M,fi,s}}{kKA_s f_{sy,\theta s}}\left(0.85\frac{f_c}{\gamma_{M,fi,c}} \times 0.45d - A_s \frac{f_{sy,\theta s}}{\gamma_{M,fi,s}} \frac{K+1}{2}\right)\end{array}\right]$	0.909

Step 7: Calculation of the enhancement factors for the membrane action

The multiplication factors e_{1b}, e_{2b}, e_{1m} and e_{2m} can be determined:

Table 5-4 Enhancement factors the assessment of the membrane action in Zone B

Equation	Obtained value
$e_{1b} = 2n\left(1+\alpha_1 b\frac{k-1}{2}-\frac{\beta_1 b^2}{3}\left(k^2-k+1\right)\right)+(1-2n)\left(1-\alpha_1 b-\beta_1 b^2\right)$	0.952
$e_{1m} = \frac{4b}{3+(g_0)_1}\frac{w}{d}\left((1-2n)+n\frac{2+3k-k^3}{3(1+k)^2}\right)$	5.407
$e_1 = e_{1b} + e_{1m}$	6.360
$e_{2b} = 1+\frac{\alpha_2 bK}{2}(k-1)-\frac{\beta_2 b^2 K}{3}\left(k^2-k+1\right)$	1.016
$e_{2m} = \frac{4bK}{3+(g_0)_2}\frac{w}{d}\frac{2+3k-k^3}{6(1+k)^2}$	2.777
$e_2 = e_{2b} + e_{2m}$	3.794

Then, the global enhancement factor e is determined by:

$$e = e_1 - \frac{e_1 - e_2}{1+2\mu a^2} = 6.360 - \frac{6.360 - 3.7948}{1+2\times 1.0\times 1.333^2} = 5.796$$

Step 8: Total bearing capacity of the slab in fire condition

The total bearing capacity of the slab in fire condition taking into account the membrane action can be obtained from:

$$q_{fi,Rd,slab} = e \times p_{fi} = 5.796 \times 0.461 = 2.670 \text{ kN/m}^2$$

Step 9: Bearing capacity of the slab taking into account the contribution of the unprotected composite beams

From paragraph 4.3.4.2.2 of EN 1994-1-2, it is possible to determine the temperature of the unprotected composite beams. In a first step, it is necessary to calculate the section factor of the steel section IPE400. The calculated values are summarised in Table 5-5.

From Table 3-2, the temperatures of the steel part of the composite section are the following:

- temperature of the flanges: 938.6°C;
- temperature of the web: 941.5°C in Table 3-2 but taken as 938.6°C because the depth of the steel section is not greater than 500 mm;
- temperature of the studs (see 4.3.4.2.5 of EN 1994-1-2): 938.6 × 0.8 = 750.9°C

Table 5-5 Section factor of the unprotected composite beam

Steel section member	$k_{sh} = 0.9\left(\dfrac{H + 0.5B}{H + 1.5B - t_w}\right)$	$\left(\dfrac{A_i}{V_i}\right)$ (m^{-1})	$k_{sh}\left(\dfrac{A_i}{V_i}\right)$ (m^{-1})
Lower flange	0.668	$\dfrac{2(B + t_f)}{Bt_f} = 159$	106
Web		$\dfrac{2}{t_w} = 233$	155
Upper flange		$\dfrac{2(B + t_f)}{Bt_f} = 159$	106
With: H: depth of the steel section; B: width of the steel section; t_f: thickness of the flange; t_w: thickness of the web.			

The temperatures of the steel section and of the steel studs allow determining the moment resistance of the internal non composite unprotected beams. The calculated values are given in Table 5-6.

Table 5-6 Moment resistance for unprotected composite beams in Zone B

Parameters	Calculated values
Effective with of the slab	$b_{eff} = \min\{9\,000/4\,;3\,000\} = 2\,250$ mm
Area of the steel section A_i	$A_i = 8\,446$ mm²
Reduction factor for the steel strength properties	$k_{y,\theta} = 0.0523$
Reduction factor for the stud strength properties	$k_{u,\theta} = 0.17$
Thickness of the slab in compression in fire situation $h_u = \dfrac{\sum A_i f_y k_{y,\theta} / \gamma_{M,fi,a}}{b_{eff} f_c / \gamma_{M,fi,c}}$	$h_u = \dfrac{8\,446 \times 355 \times 0.0523/1.0}{2\,250 \times 25/1.0} = 2.787$ mm
Connection degree of the beam at 20°C	$n_{c,20°C} = 0.51$
Connection degree of the beam in fire situation $n_{c,\theta} = \dfrac{n_{c,20°C} k_{u,\theta} \gamma_{M,v}}{k_{y,\theta} \gamma_{M,fi,v}}$	$n_{c,\theta} = \dfrac{0.51 \times 0.17 \times 1.25}{0.0523 \times 1.0} = 2.09 > 1.0$ So full shear connection
Positive moment resistance $M_{fi,Rd} = \dfrac{A_i f_y k_{y,\theta}}{\gamma_{M,fi,a}} \left(\dfrac{H}{2} + h_c - \dfrac{h_u}{2} \right)$	$M_{fi,Rd} = \dfrac{8\,446 \times 355 \times 0.0523}{1.0} \left(\dfrac{400}{2} + 130 - \dfrac{2.787}{2} \right)$ $= 51.51 \times 10^6$ Nmm $= 51.51$ kNm
With: h_c: total thickness of the slab; $\gamma_{M,fi,a}$, $\gamma_{M,v}$ and $\gamma_{M,fi,v}$ partial safety factor for the steel profile, the steel stud in normal conditions and in fire conditions.	

Then, the bearing capacity of the slab thanks to the contribution of the unprotected composite beam can be obtained from:

$$q_{fi,Rd,ub} = \frac{8M_{fi,Rd}}{L_1^2} \frac{1+n_{ub}}{L_2} = \frac{8 \times 51.5}{9^2} \times \frac{(1+3)}{12} = 1.70 \text{ kN/m}^2$$

Step 10: Total bearing capacity of the slab in fire conditions and verification of the fire resistance of the slab

The total bearing capacity of the slab is:

$$q_{fi,Rd} = q_{fi,Rd,slab} + q_{fi,Rd,ub} = 2.67 + 1.70 = 4.37 \text{ kN/m}^2$$

With regards to the applied load on the slab in fire situation:

$$q_{fi,Sd} = 5.98 \text{ kN/m}^2 > q_{fi,Rd} = 4.37 \text{ kN/m}^2$$

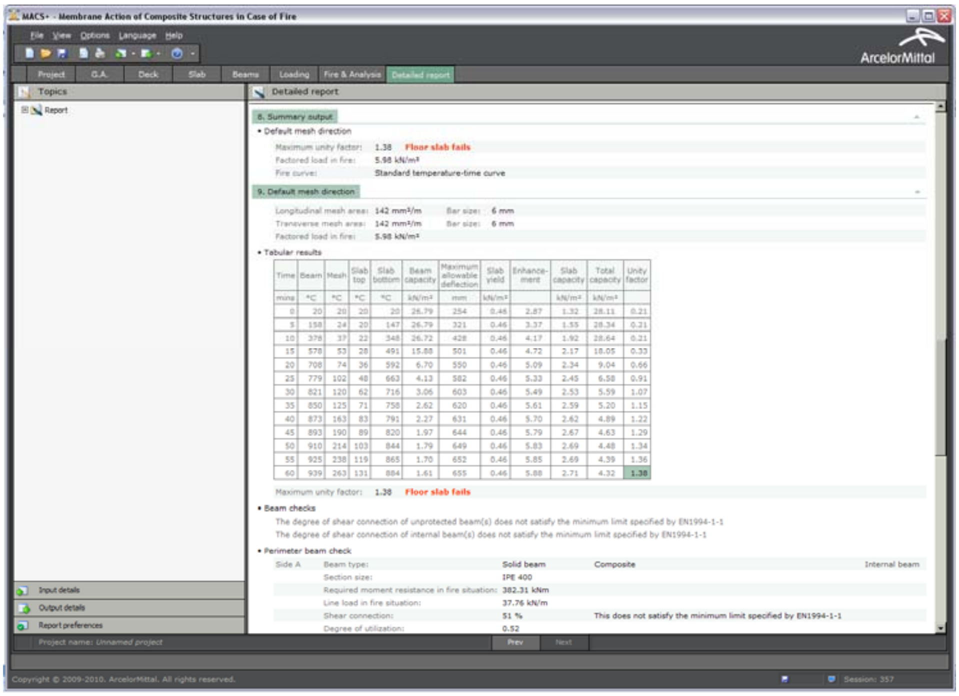


Figure 5-13 Output data using the MACS+ software - Detailed report

Conclusion 1

In conclusion, the stability of the slab system cannot be ensured for R60 with its actual dimensions in Zone B. So, it is necessary to modify the constructive parameters.

An adequate solution could be to increase the size of the reinforcing mesh to bring more resistance to the slab. So, the size of the welded mesh was increased from ST 15C (142 mm²/m) to ST 25C (257 mm²/m).

A new calculation needs to be performed with the new input data. But, it is only necessary to recalculate the bearing capacity of the slab because the unprotected composite beams remain unchanged.

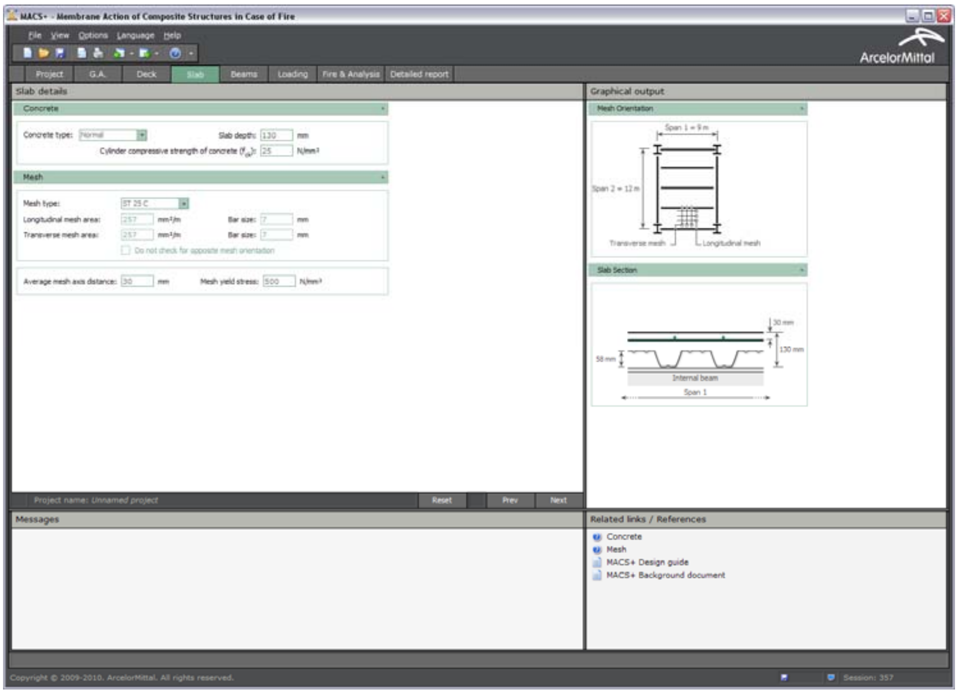


Figure 5-14 Input data using the MACS+ software - Slab

Step 2a: Calculation of the heat transfer into the composite slab Cofraplus 60

The results are identical to the step 2 because the overall dimensions of the slab remain unchanged.

Step 3a: Calculation of the resisting bending moment of the slab section $M_{fi,0}$

It can be obtained:

$$(g_0)_1 = 1 - \frac{2KA_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}}{0.85\, f_c / \gamma_{M,fi,c}\, d} = 1 - \frac{2 \times 1.0 \times \frac{257}{1000} \times 500/1.0}{0.85 \times 25/1.0 \times 30} = 0.597$$

$$(g_0)_2 = 1 - \frac{2A_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}}{0.85\, f_c / \gamma_{M,fi,c}\, d} = 1 - \frac{2 \times 1.0 \times \frac{257}{1000} \times 500/1.0}{0.85 \times 25/1.0 \times 30} = 0.597$$

It is to be noticed that the parameter *K* is equal to 1.0 because the reinforcing mesh has the same cross section in both dimensions.

So, the positive moment resistance of the slab section is:

$$M_{fi,0} = A_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}\, d\, \frac{3 + (g_0)_2}{4} = \frac{257}{1000} \times 500/1.0 \times 30 \times \frac{3 + 0.597}{4} = 3\,466.5 \text{ Nmm/mm}$$

In parallel, it is also possible to determine the other necessary parameters:

$$\mu = K\frac{3 + (g_0)_1}{3 + (g_0)_2} = 1.0 \times \frac{3 + 0.597}{3 + 0.597} = 1.0$$

$$a = \frac{L}{\ell} = \frac{12\,000}{9\,000} = 1.333$$

$$n = \frac{1}{2\mu a^2}\left(\sqrt{3\mu a^2 + 1} - 1\right) = \frac{1}{2 \times 1.0 \times 1.333^2} \times \left(\sqrt{3 \times 1.0 \times 1.333^2 + 1} - 1\right) = 0.427$$

Step 4a: Determination of the reference bearing capacity of the slab

The reference bearing capacity of the slab can be determined from:

$$p_{fi} = 6\frac{M_{fi,0}}{n^2 a^2 \ell^2} = 6 \times \frac{3\,466.5}{0.427^2 \times 1.333^2 \times 9\,000^2} = 0.794 \times 10^{-3} \text{ N/mm}^2 = 0.794 \text{ kN/m}^2$$

Step 5a: Determination of the deflection for the calculation of the membrane action

The deflection of the slab in fire situation to take into account membrane action can be obtained from:

$$w = \min\left\{\frac{\alpha(\theta_2 - \theta_1)\ell^2}{19.2h_{eff}} + \min\left[\sqrt{\left(\frac{0.5f_{sy}}{E_a\gamma_{M,fi,s}}\right)\frac{3L^2}{8}};\quad \frac{\ell}{30}\right];\quad \frac{L+\ell}{30}\right\}$$

$$= \min\left\{\frac{1.2\times10^{-5}(831-992)\times 9\,000^2}{19.2\times 95} + \min\left[\sqrt{\left(\frac{0.5\times 500}{210\,000\times 1.0}\right)\frac{3\times 12\,000^2}{8}};\frac{9\,000}{30}\right];\frac{12\,000+9\,000}{30}\right\}$$

$$= \min\{391.0 + \min[253.5\,;300];700\} = 644.5\text{ mm}$$

Step 6a: Calculation of the parameters to determine the membrane action

The determination of the different multiplication factors for the membrane action are based on the different parameters α_1, α_2, β_1, β_2, A, B, C, D, k and b that need to be determinedro. The values of theses parameters are summarized in Table 5-7.

Table 5-7 Parameters used for the assessment of the membrane action in Zone B

Equation	Obtained values
$\alpha_1 = \frac{2(g_0)_1}{3+(g_0)_1}$	0.332
$\beta_1 = \frac{1-(g_0)_1}{3+(g_0)_1}$	0.112
$\alpha_2 = \frac{2(g_0)_2}{3+(g_0)_2}$	0.332
$\beta_2 = \frac{1-(g_0)_2}{3+(g_0)_2}$	0.112
$k = \frac{4na^2(1-2n)}{4n^2a^2+1}+1$	1.194
$A = \frac{1}{2(1+k)}\left[\frac{\ell^2}{8n} - \left(\frac{1-2n}{2n}+\frac{1}{3(1+k)}\right)\left((nL)^2+(\ell/2)^2\right)\right]$	1 978 359 mm^2
$B = \frac{k^2}{2(1+k)}\left[\frac{nL^2}{2} - \frac{k}{3(1+k)}\left((nL)^2+(\ell/2)^2\right)\right]$	7 242 376 mm^2
$C = \frac{\ell^2}{16n}(k-1)$	2 305 602 mm^2
$D = \frac{L^2}{8}(1-2n)^2$	388 465 mm^2
$b = \min\left[\frac{\ell^2}{8K(A+B+C-D)}, \frac{\gamma_{M,fi,s}}{kKA_s f_{sy,\theta s}}\left(0.85\frac{f_c}{\gamma_{M,fi,c}}\times 0.45d - A_s\frac{f_{sy,\theta s}}{\gamma_{M,fi,s}}\frac{K+1}{2}\right)\right]$	0.909

Step 7a: Calculation of the enhancement factors for the membrane action

The multiplication factors e_{1b}, e_{2b}, e_{1m} and e_{2m} can be determined:

Table 5-8 Enhancement factors the assessment of the membrane action in Zone B

Equation	Obtained values
$e_{1b}=2n\left(1+\alpha_1 b\frac{k-1}{2}-\frac{\beta_1 b^2}{3}(k^2-k+1)\right)+(1-2n)(1-\alpha_1 b-\beta_1 b^2)$	0.935
$e_{1m}=\frac{4b}{3+(g_0)_1}\frac{w}{d}\left((1-2n)+n\frac{2+3k-k^3}{3(1+k)^2}\right)$	5.679
$e_1=e_{1b}+e_{1m}$	6.614
$e_{2b}=1+\frac{\alpha_2 bK}{2}(k-1)-\frac{\beta_2 b^2 K}{3}(k^2-k+1)$	0.991
$e_{2m}=\frac{4bK}{3+(g_0)_2}\frac{w}{d}\frac{2+3k-k^3}{6(1+k)^2}$	2.917
$e_2=e_{2b}+e_{2m}$	3.908

Then, the global enhancement factor e is determined by:

$$e=e_1-\frac{e_1-e_2}{1+2\mu a^2}=6.614-\frac{6.614-3.908}{1+2\times 1.0\times 1.333^2}=6.020$$

Step 8a: Total bearing capacity of the slab in fire condition

The total bearing capacity of the slab in fire condition taking into account the membrane action can be obtained from:

$$q_{fi,Rd,slab}=e\times p_{fi}=6.020\times 0.794=4.78\text{ kN/m}^2$$

Step 9a: Bearing capacity of the slab taking into account the contribution of the unprotected composite beams

Same as Step 9

Step 10a: Total bearing capacity of the slab in fire conditions and verification of the fire resistance of the slab

The total bearing capacity of the slab is:

$$q_{fi,Rd} = q_{fi,Rd,slab} + q_{fi,Rd,ub} = 4.78 + 1.70 = 6.48 \text{ kN/m}^2$$

With regards to the applied load on the slab in fire situation:

$$q_{fi,Sd} = 5.98 \text{ kN/m}^2 < q_{fi,Rd} = 6.48 \text{ kN/m}^2$$

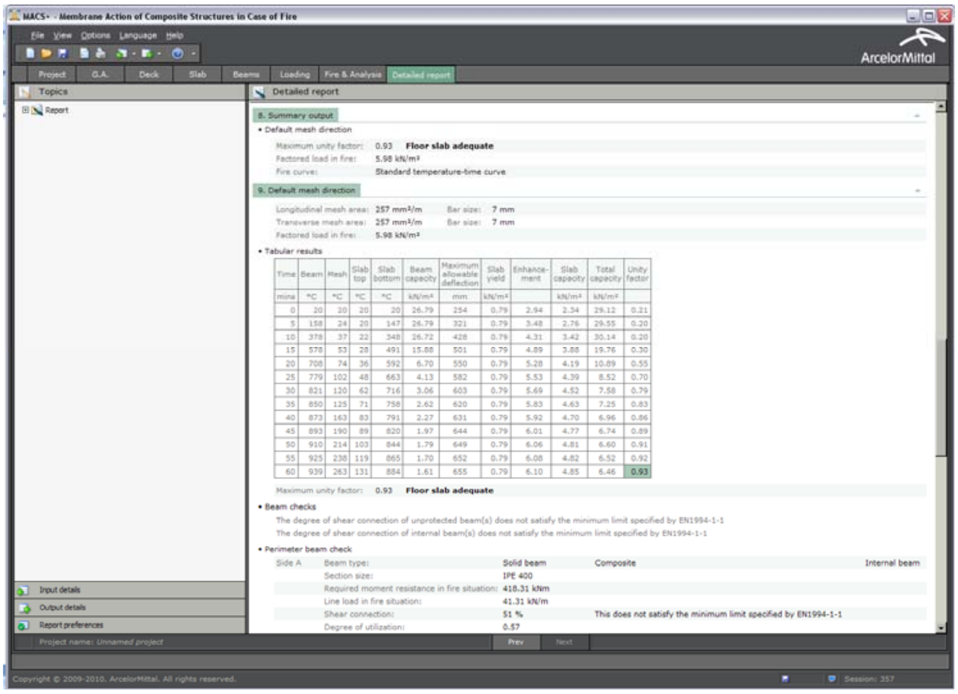


Figure 5-15 Output data using the MACS+ software – Detailed report

Conclusion 2

In conclusion, the stability of the slab system is ensured for R60 with its actual dimensions in Zone B.

Step 11: Applied load in fire situation for perimeter beams

The applied loads in fire situation on the secondary beams and perimeter beams of Zone B are calculated from relations 3.24 to 3.37:

- For the secondary perimeter beams

$$M_{fi,Sd,b,1} = \frac{q_{fi,Rd}L_1^{\,2}L_2 - 8\left(M_{fi,0}\left(L_2 - n_{ub}b_{eff,ub} - \sum_{i=1}^{2} b_{eff,1,i}\right) + n_{ub}M_{fi,Rd}\right)}{c_M}$$

$$= \frac{6.48 \times 9^2 \times 12 - 8 \times \{3\,466.5 \times 10^{-3} \times [12 - 3 \times 2.25 - (0 + 2.25/2)] + 3 \times 51.5\}}{12}$$

$$= 412.3 \text{ kNm}$$

$$V_{fi,Sd,b,1} = \frac{4M_{fi,Sd,b,1}}{L_1} = \frac{4 \times 412.3}{9} = 183.3 \text{ kN}$$

- For the primary perimeter beams

$$M_{fi,Sd,b,2} = \frac{q_{fi,Rd} L_1 L_2^{\,2} - 8\mu M_{fi,0}\left(L_1 - \sum_{i=1}^{2} b_{eff,2,i}\right)}{c_M} = \frac{6.48\times9\times12^2 - 8\times1.0\times3466.5\times10^{-3}\times(9-(12/8+12/8))}{12}$$
$$= 686.0 \text{ kNm}$$

$$V_{fi,Sd,b,2} = \frac{4M_{fi,Sd,b,2}}{L_2} = \frac{4\times686.0}{12} = 228.7 \text{ kN}$$

One of the primary beams of this zone is an edge beam at the façade level, it must support an additional load coming from the façade elements of 2.0 kN/m, which implies a modification of the applied load in fire condition following the next relations:

$$M_{fi,Sd,b,2} = 686.0 + \frac{2.0\times12^2}{8} = 722.0 \text{ kNm}$$
$$V_{fi,Sd,b,2} = 222.8 + \frac{2.0\times12}{2} = 234.8 \text{ kN}$$

So, the fire protection of this beam must be determined to ensure that the calculated bearing capacity in fire situation is not lower than the applied loads for the requested fire duration.

5.1.2 Floor design: Zone A

The applied calculation procedure is the same as the one applied for Zone B. Here, the dimensions are 9 m by 9 m. In order to simplify the construction, the mesh ST 25C will also be used in this area in order to have the same section for the entire slab surface. In consequence, Zone A will be also verified with this mesh section. This calculation zone is composed of 2 unprotected composite beams. The details of the calculation are given below:

Step 1: Calculation of the applied load on the slab in case of fire

Same as the calculation for Zone B

Step 2: Calculation of the heat transfer into the composite slab Cofraplus 60

Same as the calculation for Zone B

Step 3: Calculation of the moment resistance of the slab section $M_{fi,0}$

For this calculation zone:

$L_1 = 9\ 000$ mm

$L_2 = 9\ 000$ mm

So, $L = \max\{L_1; L_2\} = 9\ 000$ mm and $\ell = \min\{L_1; L_2\} = 9\ 000$ mm.

It can be obtained:

$$(g_0)_1 = 1 - \frac{2KA_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}}{0.85\, f_c / \gamma_{M,fi,c}\, d} = 1 - \frac{2 \times 1.0 \times \frac{257}{1000} \times 500/1.0}{0.85 \times 25/1.0 \times 30} = 0.597$$

$$(g_0)_2 = 1 - \frac{2A_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}}{0.85\, f_c / \gamma_{M,fi,c}\, d} = 1 - \frac{2 \times 1.0 \times \frac{257}{1000} \times 500/1.0}{0.85 \times 25/1.0 \times 30} = 0.597$$

It is to be noticed that the parameter K is equal to 1.0 because the reinforcing mesh has the same cross section in both dimensions.

So, the positive moment resistance of the slab section is:

$$M_{fi,0} = A_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}\, d\, \frac{3+(g_0)_2}{4} = \frac{257}{1000} \times 500/1.0 \times 30 \times \frac{3+0.597}{4} = 3\,466.5 \text{ Nmm/mm}$$

In parallel, it is also possible to determine the other necessary parameters:

$$\mu = K\frac{3+(g_0)_1}{3+(g_0)_2} = 1.0 \times \frac{3+0.597}{3+0.597} = 1.0$$

$$a = \frac{L}{\ell} = \frac{9\,000}{9\,000} = 1.0$$

$$n = \frac{1}{2\mu a^2}\left(\sqrt{3\mu a^2+1}-1\right) = \frac{1}{2 \times 1.0 \times 1.0^2} \times \left(\sqrt{3 \times 1.0 \times 1.0^2 + 1} - 1\right) = 0.50$$

Step 4: Determination of the reference bearing capacity of the slab

The reference bearing capacity of the slab can be determined from:

$$p_{fi} = 6\frac{M_{fi,0}}{n^2 a^2 \ell^2} = 6 \times \frac{3\,466.5}{0.5^2 \times 1.0^2 \times 9\,000^2} = 1.027 \times 10^{-3} \text{ N/mm}^2 = 1.027 \text{ kN/m}^2$$

Step 5: Determination of the deflection for the calculation of the membrane action

The deflection of the slab in fire situation to take into account membrane action can be obtained from:

$$\begin{aligned} w &= \min\left\{\frac{\alpha(\theta_2-\theta_1)\ell^2}{19.2 h_{eff}} + \min\left[\sqrt{\left(\frac{0.5 f_{sy}}{E_a \gamma_{M,fi,s}}\right)\frac{3L^2}{8}};\ \frac{\ell}{30}\right];\ \frac{L+\ell}{30}\right\} \\ &= \min\left\{\frac{1.2 \times 10^{-5}(831-99) \times 9\,000^2}{19.2 \times 95} + \min\left[\sqrt{\left(\frac{0.5 \times 500}{210\,000 \times 1.0}\right)\frac{3 \times 9\,000^2}{8}};\frac{9\,000}{30}\right];\frac{9\,000+9\,000}{30}\right\} \\ &= \min\{391.0 + \min[190.2\,;300];\,600\} = 581.2 \text{ mm} \end{aligned}$$

Step 6: Calculation of the parameters to determine the membrane action

The determination of the different multiplication factors for the membrane action are based on the different parameters α_1, α_2, β_1, β_2, A, B, C, D, k and b that need to be determinedro. The values of theses parameters are summarized in Table 5-9.

Table 5-9 Parameters used for the assessment of the membrane action in Zone A

Equation	Obtained value
$\alpha_1 = \dfrac{2(g_0)_1}{3+(g_0)_1}$	0.332
$\beta_1 = \dfrac{1-(g_0)_1}{3+(g_0)_1}$	0.112
$\alpha_2 = \dfrac{2(g_0)_2}{3+(g_0)_2}$	0.332
$\beta_2 = \dfrac{1-(g_0)_2}{3+(g_0)_2}$	0.112
$k = \dfrac{4na^2(1-2n)}{4n^2a^2+1}+1$	1.0
$A = \dfrac{1}{2(1+k)}\left[\dfrac{\ell^2}{8n} - \left(\dfrac{1-2n}{2n} + \dfrac{1}{3(1+k)}\right)\left((nL)^2 + (\ell/2)^2\right)\right]$	3 375 000 mm^2
$B = \dfrac{k^2}{2(1+k)}\left[\dfrac{nL^2}{2} - \dfrac{k}{3(1+k)}\left((nL)^2 + (\ell/2)^2\right)\right]$	3 375 000 mm^2
$C = \dfrac{\ell^2}{16n}(k-1)$	0 mm^2
$D = \dfrac{L^2}{8}(1-2n)^2$	0 mm^2
$b = \min\left[\dfrac{\ell^2}{8K(A+B+C-D)}, \dfrac{\gamma_{M,fi,s}}{kKA_s f_{sy,\theta s}}\left(0.85\dfrac{f_c}{\gamma_{M,fi,c}} \times 0.45d - A_s \dfrac{f_{sy,\theta s}}{\gamma_{M,fi,s}}\dfrac{K+1}{2}\right)\right]$	1.232

Step 7: Calculation of the enhancement factors for the membrane action

The multiplication factors e_{1b}, e_{2b}, e_{1m} and e_{2m} can be determined:

Table 5-10: Enhancement factors the assessment of the membrane action in Zone A

Equation	Obtained Value
$e_{1b} = 2n\left(1+\alpha_1 b\frac{k-1}{2}-\frac{\beta_1 b^2}{3}\left(k^2-k+1\right)\right)+(1-2n)\left(1-\alpha_1 b-\beta_1 b^2\right)$	0.943
$e_{1m} = \frac{4b}{3+(g_0)_1}\frac{w}{d}\left((1-2n)+n\frac{\left(2+3k-k^3\right)}{3(1+k)^2}\right)$	4.425
$e_1 = e_{1b} + e_{1m}$	5.368
$e_{2b} = 1+\frac{\alpha_2 bK}{2}(k-1)-\frac{\beta_2 b^2 K}{3}\left(k^2-k+1\right)$	0.943
$e_{2m} = \frac{4bK}{3+(g_0)_2}\frac{w}{d}\frac{\left(2+3k-k^3\right)}{6(1+k)^2}$	4.425
$e_2 = e_{2b} + e_{2m}$	5.368

Then, the global enhancement factor e is determined by:

$$e = e_1 - \frac{e_1 - e_2}{1+2\mu a^2} = 5.368 - \frac{5.368-5.368}{1+2\times 1.0\times 1.0^2} = 5.368$$

Step 8: Total bearing capacity of the slab in fire condition

The total bearing capacity of the slab in fire condition taking into account the membrane action can be obtained from:

$$q_{fi,Rd,slab} = e \times p_{fi} = 5.368 \times 1.027 = 5.51 \text{ kN/m}^2$$

Step 9: Bearing capacity of the slab taking into account the contribution of the unprotected composite beams

The moment resistance of the beams has the same value as in Zone A, but the calculation of their bearing capacity is modified due to a different number of internal unprotected beams, and a different span of the primary beams:

$$q_{fi,Rd,ub} = \frac{8M_{fi,Rd}}{L_1^2}\frac{1+n_{ub}}{L_2} = \frac{8\times 51.5}{9^2}\times\frac{(1+2)}{9} = 1.70 \text{ kN/m}^2$$

Step 10: Total bearing capacity of the slab in fire conditions and verification of the fire resistance of the slab

The total bearing capacity of the slab is:

$$q_{fi,Rd} = q_{fi,Rd,slab} + q_{fi,Rd,ub} = 5.51 + 1.70 = 7.21 \text{ kN/m}^2$$

With regards to the applied load on the slab in fire situation

$$q_{fi,Sd} = 5.98 \text{ kN/m}^2 < q_{fi,Rd} = 7.21 \text{ kN/m}^2$$

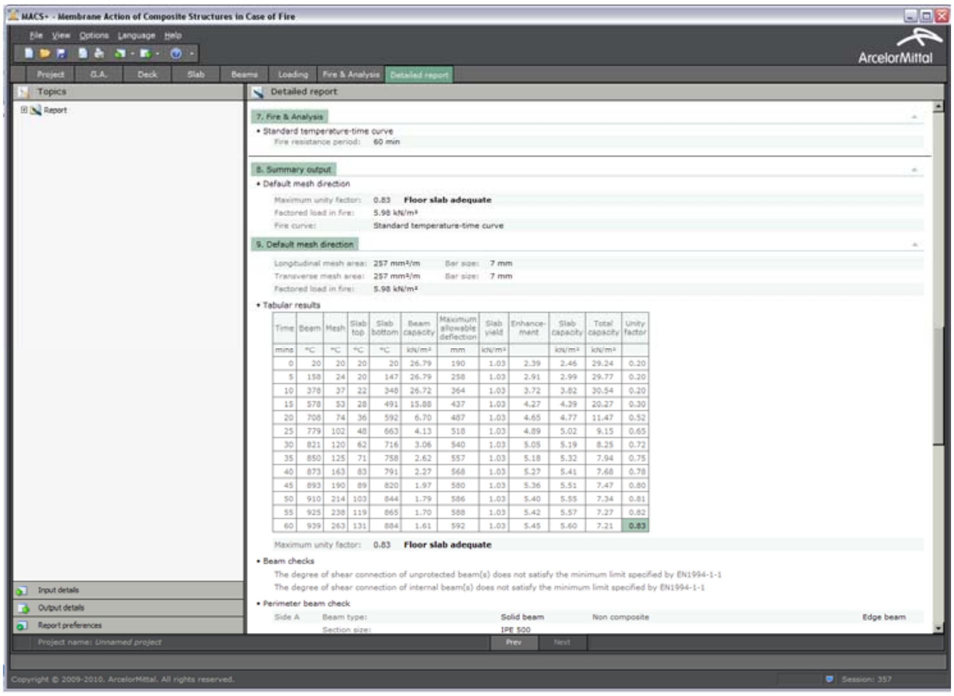


Figure 5-16 Output data using the MACS+ software – Detailed report

In conclusion, the stability of the slab system is ensured for R60 with its actual dimensions in Zone A.

Step 11: Applied load in fire situation for perimeter beams

The applied loads in fire situation on the secondary beams and perimeter beams of Zone A are calculated from relations 3.24 to 3.37:

- For the secondary perimeter beams

$$M_{fi,Sd,b,1} = \frac{q_{fi,Rd} L_1^2 L_2 - 8\left(M_{fi,0}\left(L_2 - n_{ub} b_{eff,ub} - \sum_{i=1}^{2} b_{eff,1,i}\right) + n_{ub} M_{fi,Rd}\right)}{c_M}$$

$$= \frac{7.21 \times 9^2 \times 9 - 8 \times \{3466.5 \times 10^{-3} \times [9 - 2 \times 2.25 - (0 + 2.25/2)] + 2 \times 51.5\}}{12}$$

$$= 361.5 \text{ kNm}$$

$$V_{fi,Sd,b,1} = \frac{4 M_{fi,Sd,b,1}}{L_1} = \frac{4 \times 361.5}{9} = 160.7 \text{ kN}$$

- For the primary perimeter beams

$$M_{fi,Sd,b,2} = \frac{q_{fi,Rd} L_1 L_2^{\ 2} - 8\mu M_{fi.0}\left(L_1 - \sum_{i=1}^{2} b_{eff,2,i}\right)}{c_M} = \frac{7.21 \times 9 \times 9^2 - 8 \times 1.0 \times 3466.5 \times 10^{-3} \times \left(9 - (0 + 9/8)\right)}{12}$$

$$= 419.8 \text{ kNm}$$

$$V_{fi,Sd,b,2} = \frac{4M_{fi,Sd,b,2}}{L_2} = \frac{4 \times 419.8}{9} = 186.6 \text{ kN}$$

Two of the perimeter beams of this zone are corner beams at the façade level, they must support an additional load coming from the façade elements of 2.0 kN/m, which implies a modification of the applied load in fire condition following the next relations:

- For the secondary perimeter edge beam

$$M_{fi,Sd,b,1} = 361.5 + \frac{2.0 \times 9^2}{8} = 381.7 \text{ kNm} \quad \text{and} \quad V_{fi,Sd,b,1} = 160.7 + \frac{2.0 \times 9}{2} = 169.7 \text{ kN}$$

- For the primary perimeter edge beam

$$M_{fi,Sd,b,2} = 419.8 + \frac{2.0 \times 9^2}{8} = 440.0 \text{ kNm} \quad \text{and} \quad V_{fi,Sd,b,2} = 186.6 + \frac{2.0 \times 9}{2} = 195.6 \text{ kN}$$

So, the fire protection of these beams must be determined to ensure that the calculated bearing capacity in fire situation is not lower than the applied loads for the requested fire duration.

5.1.3 Floor design: Zone E

In Zone E, the dimensions of the composite slab and the spans of the beams have the same values as in Zone B. However, solid beams are replaced by IPE 300+IPE 300 ACB beams (see cross-section in Figure 5-18).

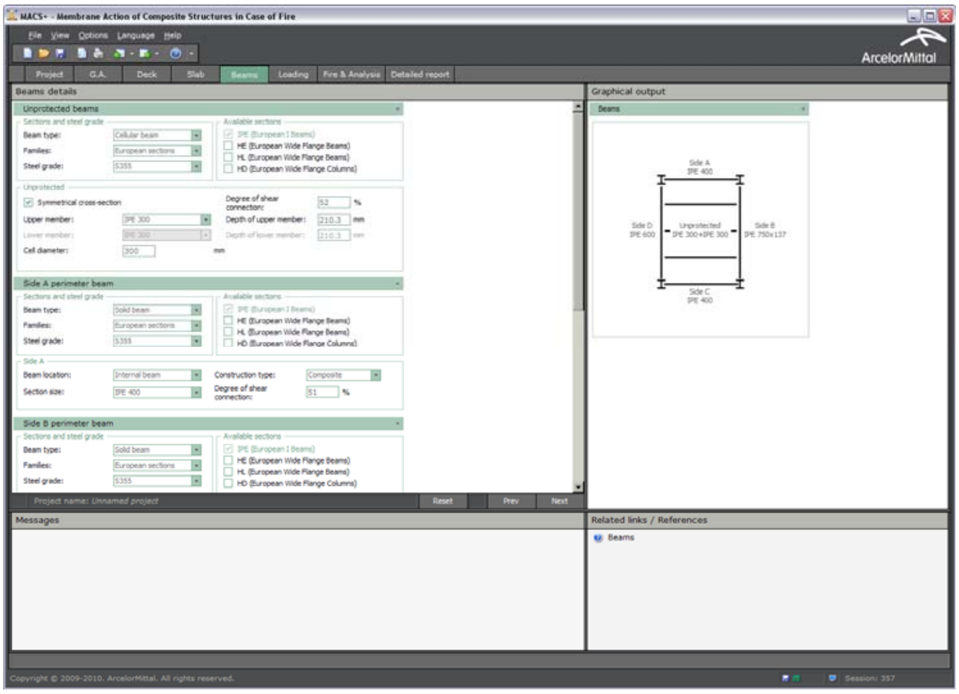


Figure 5-17 Input data using the MACS+ software – Beams in Zone E

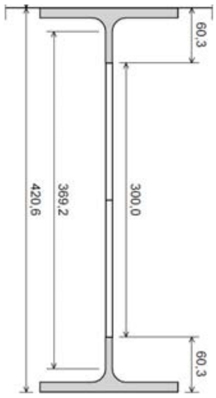


Figure 5-18 Net cross-section of ACB beam in Zone E

In consequence, only the load-bearing capacity of the unprotected beams needs to be determined.

Steps 1 to 8: same as Zone B

Step 9: Bearing capacity of the slab taking into account the contribution of the unprotected composite beams

The values of the section factors of the steel section are summarized in Table 5-11.

From Table 3-2, the temperatures of the steel part of the composite section are the following:

- temperature of the flanges: 940.0°C;
- temperature of the lower web: 942.1°C in Table 3-2 but taken as 940.0°C because the depth of the steel section is not bigger than 500 mm;
- temperature of the upper web: 942.1°C;
- temperature of the studs (see 4.3.4.2.5 of EN 1994-1-2): 940.0×0.8 = 752.0°C

Table 5-11 Section factor of the unprotected composite beam

Steel section member	$k_{sh} = 0.9\left(\dfrac{0.5B_1 + t_{f1} + t_{f2} + \sqrt{h_w^2 + (B_1 - B_2)^2/4}}{H + B_1 + B_2/2 - (t_{w1} + t_{w2})/2}\right)$	$\left(\dfrac{A_i}{V_i}\right)$ (m^{-1})	$k_{sh}\left(\dfrac{A_i}{V_i}\right)$ (m^{-1})
Lower flange	0.699	$\dfrac{2(B_1 + t_{f1})}{B_1 t_{f1}} = 200$	140
Lower web		$\dfrac{2h_{w1} + t_{w1}}{h_{w1} t_{w1}} = 302$	211
Upper web		$\dfrac{2h_{w2} + t_{w2}}{h_{w2} t_{w2}} = 302$	211
Upper flange		$\dfrac{2(B_2 + t_{f_2})}{B_2 t_{f2}} = 200$	140

With: H: depth of the steel section; h_w: overall depth of the web; B_1: width of the lower flange; t_{f1}: thickness of the lower flange; t_{w1}: thickness of the lower web; h_{w1}: depth of the lower web (net cross-section); B_2: width of the upper flange; t_{f2}: thickness of the upper flange; t_{w2}: thickness of the upper web; h_{w2}: depth of the upper web (net cross-section).

The temperatures of the steel section and of the steel studs allow determining the moment resistance of the internal non composite unprotected beams. For Cellular Beams, the contribution of the lower member is neglected as its temperature exceeds 600°C. The calculated values are given in Table 5-12.

Table 5-12 Moment resistance for unprotected composite beams in Zone E

Parameters	Calculated values
Effective with of the slab	$b_{eff} = \min\{9\,000/4\,;\,3\,000\} = 2\,250$ mm
Area of the upper flange A_{f2}	$A_{f2} = 1605$ mm²
Area of the upper web A_{w2}	$A_{w2} = 352$ mm²
Reduction factor for the steel strength properties	$k_{y,\theta} = 0.052$
Reduction factor for the stud strength properties	$k_{u,\theta} = 0.17$
Tensile force $T^+ = \sum A_i f_y k_{y,\theta} / \gamma_{M,fi,a}$	$T^+ = (1605 + 352) \times 355 \times 0.052/1.0$ $= 36.08$ kN
Thickness of the slab in compression in fire situation $h_u = \dfrac{T^+}{b_{eff} f_c / \gamma_{M,fi,c}}$	$h_u = \dfrac{36.08}{2\,250 \times 25/1.0} = 0.641$ mm
Connection degree of the beam at 20°C	$n_{c,20°C} = 0.52$
Connection degree of the beam in fire situation $n_{c,\theta} = \dfrac{n_{c,20°C} k_{u,\theta} \gamma_{M,v}}{k_{y,\theta} \gamma_{M,fi,v}}$	$n_{c,\theta} = \dfrac{0.52 \times 0.17 \times 1.25}{0.052 \times 1.0} = 2.05 > 1.0$ So full shear connection
Tensile force application point $y_T = \dfrac{\sum A_i y_i f_y k_{y,\theta}}{T^+ \gamma_{M,fi,a}}$	$y_T = \dfrac{(352 \times 6.45 + 1605 \times 29.63) \times 355 \times 0.052}{36.08 \times 1.0}$ $= 409.86$ mm
Compressive force application point $y_F = H + h_c - h_u/2$	$y_F = 420.6 + 130 - 0.641/2 = 550.28$ mm
Positive moment resistance $M_{fi,Rd} = T^+(y_F - y_T)$	$M_{fi,Rd} = 36.08 \times (550.28 - 409.86)$ $= 5.07 \times 10^6$ Nmm $= 5.07$ kNm
With: h_c: total thickness of the slab; $\gamma_{M,fi,a}$, $\gamma_{M,v}$ and $\gamma_{M,fi,v}$ partial safety factor for the steel profile, the steel stud in normal conditions and in fire conditions.	

Then, the bearing capacity of the slab thanks to the contribution of the unprotected composite beam can be obtained from:

$$q_{fi,Rd,ub} = \frac{8M_{fi,Rd}}{L_1^2} \frac{1+n_{ub}}{L_2} = \frac{8 \times 5.07}{9^2} \times \frac{(1+3)}{12} = 0.17 \text{ kN/m}^2$$

Step 10: Total bearing capacity of the slab in fire conditions and verification of the fire resistance of the slab

The total bearing capacity of the slab is:

$$q_{fi,Rd} = q_{fi,Rd,slab} + q_{fi,Rd,ub} = 4.78 + 0.17 = 4.95 \text{ kN/m}^2$$

With regards to the applied load on the slab in fire situation:

$$q_{fi,Sd} = 5.98 \text{ kN/m}^2 > q_{fi,Rd} = 4.95 \text{ kN/m}^2$$

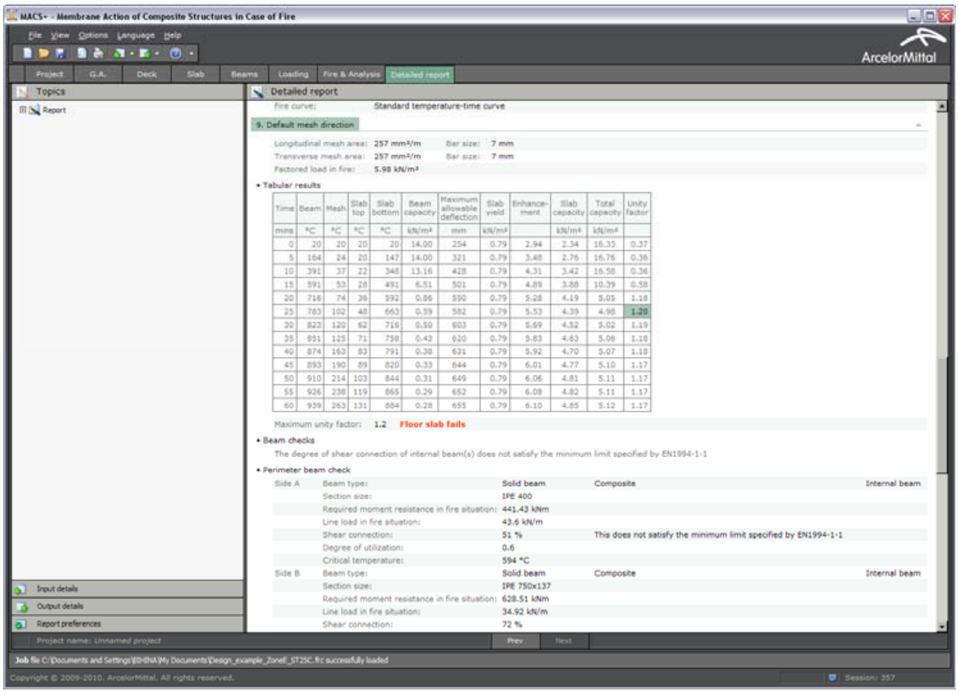


Figure 5-19 Output data using the MACS+ software – Detailed report

Conclusion 1

In conclusion, the stability of the slab system cannot be ensured for R60 with its actual dimensions in Zone E. So, it is necessary to modify the constructive parameters.

An adequate solution could be to increase or the mesh axis distance or the mesh size.

The closest mesh area in the current mesh range is equal to 385 mm^2/m, i.e. much greater than that of the current ST 25C mesh. So, the first option is to increase the mesh axis distance in such a way to as to keep its temperature below 400°C for a minimum yield strength reduction. The mesh axis distance was increased from 30 mm to 40 mm. In this case, the temperature of the reinforcement mesh increases from 288°C to 363°C. According to Table 3-4 of EN 1994-1-2, the effective yield strength of the reinforcement mesh is reduced to 96% of its value at room temperature.

For information purpose, using this increased mesh axis distance leads to the following load bearing capacities:

- Zone A: $q_{\text{fi,Rd}} = q_{\text{fi,Rd,slab}} + q_{\text{fi,Rd,ub}} = 6.60 + 1.70 = 8.30$ kN/m^2 > 7.21 kN/m^2;
- Zone B: $q_{\text{fi,Rd}} = q_{\text{fi,Rd,slab}} + q_{\text{fi,Rd,ub}} = 4.88 + 1.70 = 6.58$ kN/m^2 > 6.48 kN/m^2.

In consequence, increasing this mesh axis distance does increase the overall load bearing capacity of Zone A and Zone B.

Step 2a

Following Table 3-4 of EN 1994-1-2, the effective steel strength for the welded steel mesh is reduced as follows:

$$f_{sy,\theta_s} = 500 \times 0{,}962 = 481 \text{ MPa}$$

Step 3a: Calculation of the moment resistance of the slab section $M_{\text{fi,0}}$

For this calculation zone:

L_1 = 9 000 mm (span of the secondary beams)

L_2 = 12 000 mm (span of the primary beams)

So, $L = \max\{L_1; L_2\}$ = 12 000 mm and $\ell = \min\{L_1; L_2\}$ = 9 000 mm.

It can be obtained:

$$(g_0)_1 = 1 - \frac{2KA_s\ f_{sy,\theta_s} / \gamma_{M,fi,s}}{0.85\, f_c / \gamma_{M,fi,c}\ d} = 1 - \frac{2 \times 1.0 \times \frac{257}{1\,000} \times 4810/1.0}{0.85 \times 25/1.0 \times 40} = 0.709$$

$$(g_0)_2 = 1 - \frac{2A_s\ f_{sy,\theta_s} / \gamma_{M,fi,s}}{0.85\, f_c / \gamma_{M,fi,c}\ d} = 1 - \frac{2 \times 1.0 \times \frac{257}{1\,000} \times 481/1.0}{0.85 \times 25/1.0 \times 40} = 0.709$$

So, the positive moment resistance of the slab section is:

$$M_{fi,0} = A_s\ f_{sy,\theta_s} / \gamma_{M,fi,s}\ d\, \frac{3 + (g_0)_2}{4} = \frac{257}{1\,000} \times 0.962 \times 500/1.0 \times 40 \times \frac{3 + 0.709}{4} = 4\,586.51 \text{ Nmm/mm}$$

In parallel, it is also possible to determine the other necessary parameters:

$$\mu = K\, \frac{3 + (g_0)_1}{3 + (g_0)_2} = 1.0 \times \frac{3 + 0.709}{3 + 0.709} = 1.0$$

$$a = \frac{L}{\ell} = \frac{12\,000}{9\,000} = 1.333$$

$$n = \frac{1}{2\mu a^2}\left(\sqrt{3\mu a^2 + 1} - 1\right) = \frac{1}{2 \times 1.0 \times 1.333^2} \times \left(\sqrt{3 \times 1.0 \times 1.333^2 + 1} - 1\right) = 0.427$$

Step 4a: Determination of the reference bearing capacity of the slab

The reference bearing capacity of the slab can be determined from:

$$p_{fi} = 6\, \frac{M_{fi,0}}{n^2 a^2 \ell^2} = 6 \times \frac{4\,586.51}{0.427^2 \times 1.333^2 \times 9\,000^2} = 1.050 \times 10^{-3} \text{ N/mm}^2 = 1.050 \text{ kN/m}^2$$

Step 5a: same as Step 5

Step 6a: Calculation of the parameters to determine the membrane action

The determination of the different multiplication factors for the membrane action are based on the different parameters α_1, α_2, β_1, β_2, A, B, C, D, k and b that need to be determinedro. The values of theses parameters are summarized in Table 5-13.

Table 5-13 Parameters used for the assessment of the membrane action in Zone E

Equation	Obtained values
$\alpha_1 = \frac{2(g_0)_1}{3+(g_0)_1}$	0.382
$\beta_1 = \frac{1-(g_0)_1}{3+(g_0)_1}$	0.078
$\alpha_2 = \frac{2(g_0)_2}{3+(g_0)_2}$	0.382
$\beta_2 = \frac{1-(g_0)_2}{3+(g_0)_2}$	0.078
$k = \frac{4na^2(1-2n)}{4n^2a^2+1}+1$	1.194
$A = \frac{1}{2(1+k)}\left[\frac{\ell^2}{8n} - \left(\frac{1-2n}{2n} + \frac{1}{3(1+k)}\right)\left((nL)^2 + (\ell/2)^2\right)\right]$	1 978 359 mm^2
$B = \frac{k^2}{2(1+k)}\left[\frac{nL^2}{2} - \frac{k}{3(1+k)}\left((nL)^2 + (\ell/2)^2\right)\right]$	7 242 376 mm^2
$C = \frac{\ell^2}{16n}(k-1)$	2 305 602 mm^2
$D = \frac{L^2}{8}(1-2n)^2$	388 465 mm^2
$b = \min\left[\frac{\ell^2}{8K(A+B+C-D)}, \frac{\gamma_{M,fi,s}}{kKA_s f_{sy,\theta s}}\left(0.85\frac{f_c}{\gamma_{M,fi,c}} \times 0.45d - A_s \frac{f_{sy,\theta s}}{\gamma_{M,fi,s}}\frac{K+1}{2}\right)\right]$	0.909

Step 7a: Calculation of the enhancement factors for the membrane action

The multiplication factors e_{1b}, e_{2b}, e_{1m} and e_{2m} can be determined:

Table 5-14 Enhancement factors the assessment of the membrane action in Zone E

Equation	Obtained values
$e_{1b} = 2n\left(1+\alpha_1 b\frac{k-1}{2}-\frac{\beta_1 b^2}{3}\left(k^2-k+1\right)\right)+(1-2n)\left(1-\alpha_1 b-\beta_1 b^2\right)$	0.946
$e_{1m} = \frac{4b}{3+(g_0)_1}\frac{w}{d}\left((1-2n)+n\frac{2+3k-k^3}{3(1+k)^2}\right)$	4.130
$e_1 = e_{1b} + e_{1m}$	5.076
$e_{2b} = 1+\frac{\alpha_2 bK}{2}(k-1)-\frac{\beta_2 b^2 K}{3}\left(k^2-k+1\right)$	1.007
$e_{2m} = \frac{4bK}{3+(g_0)_2}\frac{w}{d}\frac{2+3k-k^3}{6(1+k)^2}$	2.121
$e_2 = e_{2b} + e_{2m}$	3.129

Then, the global enhancement factor *e* is determined by:

$$e = e_1 - \frac{e_1 - e_2}{1+2\mu a^2} = 5.076 - \frac{5.076-3.129}{1+2\times 1.0\times 1.333^2} = 4.648$$

Step 8a: Total bearing capacity of the slab in fire condition

The total bearing capacity of the slab in fire condition taking into account the membrane action can be obtained from:

$$q_{fi,Rd,slab} = e\times p_{fi} = 4.648\times 1.050 = 4.88 \text{ kN/m}^2$$

Step 9a: Bearing capacity of the slab taking into account the contribution of the unprotected composite beams

Same as Step 9

Step 10a: Total bearing capacity of the slab in fire conditions and verification of the fire resistance of the slab

The total bearing capacity of the slab is:

$$q_{fi,Rd} = q_{fi,Rd,slab} + q_{fi,Rd,ub} = 4.88+0.17 = 5.05 \text{ kN/m}^2$$

With regards to the applied load on the slab in fire situation:

$$q_{fi,Sd} = 5.98 \text{ kN/m}^2 > q_{fi,Rd} = 5.05 \text{ kN/m}^2$$

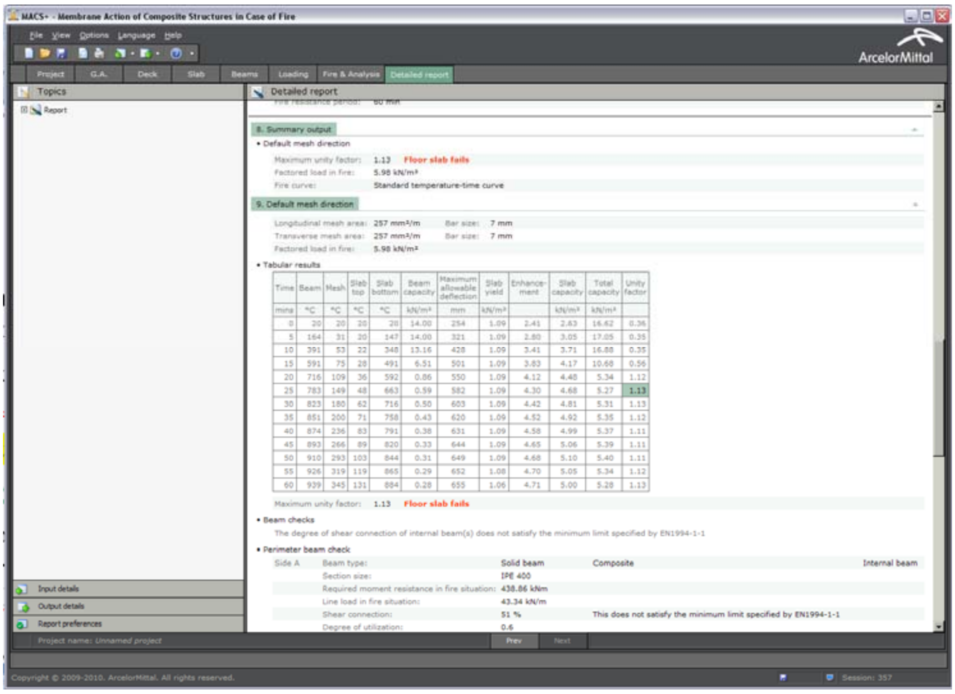


Figure 5-20 Output data using the MACS+ software – Detailed report

Conclusion 2

In conclusion, the stability of the slab system cannot be ensured for R60 with its actual dimensions in Zone E. So, it is necessary to modify the constructive parameters, for instance by increasing the reinforcement mesh area.

The size of the welded mesh was increased from ST 25C (257 mm²/m) to ST 40C (385 mm²/m).

Step 2b: same as Step 2a

Step 3b: Calculation of the moment resistance of the slab section $M_{fi,0}$

For this calculation zone:

L_1 = 9 000 mm (span of the secondary beams)

L_2 = 12 000 mm (span of the primary beams)

So, $L = \max\{L_1; L_2\}$ = 12 000 mm and $\ell = \min\{L_1; L_2\}$ = 9 000 mm.

It can be obtained:

$$(g_0)_1 = 1 - \frac{2KA_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}}{0.85\, f_c / \gamma_{M,fi,c}\, d} = 1 - \frac{2 \times 1.0 \times \dfrac{385}{1000} \times 481/1.0}{0.85 \times 25/1.0 \times 40} = 0.564$$

$$(g_0)_2 = 1 - \frac{2A_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}}{0.85\, f_c / \gamma_{M,fi,c}\; d} = 1 - \frac{2 \times 1.0 \times \frac{385}{1000} \times 481/1.0}{0.85 \times 25/1.0 \times 40} = 0.564$$

So, the positive moment resistance of the slab section is:

$$M_{fi,0} = A_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}\; d\, \frac{3 + (g_0)_2}{4} = \frac{385}{1000} \times 0.962 \times 500/1.0 \times 40 \times \frac{3 + 0.564}{4} = 6\,602.40 \text{ Nmm/mm}$$

In parallel, it is also possible to determine the other necessary parameters:

$$\mu = K\, \frac{3 + (g_0)_1}{3 + (g_0)_2} = 1.0 \times \frac{3 + 0.564}{3 + 0.564} = 1.0$$

$$a = \frac{L}{\ell} = \frac{12\,000}{9\,000} = 1.333$$

$$n = \frac{1}{2\mu a^2}\left(\sqrt{3\mu a^2 + 1} - 1\right) = \frac{1}{2 \times 1.0 \times 1.333^2} \times \left(\sqrt{3 \times 1.0 \times 1.333^2 + 1} - 1\right) = 0.427$$

Step 4b: Determination of the reference bearing capacity of the slab

The reference bearing capacity of the slab can be determined:

$$p_{fi} = 6\, \frac{M_{fi,0}}{n^2 a^2 \ell^2} = 6 \times \frac{6\,602.40}{0.427^2 \times 1.333^2 \times 9\,000^2} = 1.512 \times 10^{-3} \text{ N/mm}^2 = 1.512 \text{ kN/m}^2$$

Step 5b: same as Step 5

Step 6b: Calculation of the parameters to determine the membrane action

The determination of the different multiplication factors for the membrane action are based on the different parameters α_1, α_2, β_1, β_2, A, B, C, D, k and b that need to be determinedro. The values of theses parameters are summarized in Table 5-15.

Table 5-15   Parameters used for the assessment of the membrane action in Zone E

Equation	Obtained values
$\alpha_1 = \frac{2(g_0)_1}{3+(g_0)_1}$	0.317
$\beta_1 = \frac{1-(g_0)_1}{3+(g_0)_1}$	0.122
$\alpha_2 = \frac{2(g_0)_2}{3+(g_0)_2}$	0.317
$\beta_2 = \frac{1-(g_0)_2}{3+(g_0)_2}$	0.122
$k = \frac{4na^2(1-2n)}{4n^2a^2+1}+1$	1.194
$A = \frac{1}{2(1+k)}\left[\frac{\ell^2}{8n} - \left(\frac{1-2n}{2n} + \frac{1}{3(1+k)}\right)\left((nL)^2 + (\ell/2)^2\right)\right]$	1 978 359 mm^2
$B = \frac{k^2}{2(1+k)}\left[\frac{nL^2}{2} - \frac{k}{3(1+k)}\left((nL)^2 + (\ell/2)^2\right)\right]$	7 242 376 mm^2
$C = \frac{\ell^2}{16n}(k-1)$	2 305 602 mm^2
$D = \frac{L^2}{8}(1-2n)^2$	388 465 mm^2
$b = \min\left[\frac{\ell^2}{8K(A+B+C-D)}, \frac{\gamma_{M,fi,s}}{kKA_s f_{sy,\theta s}}\left(0.85\frac{f_c}{\gamma_{M,fi,c}} \times 0.45d - A_s\frac{f_{sy,\theta s}}{\gamma_{M,fi,s}}\frac{K+1}{2}\right)\right]$	0.892

Step 7b: Calculation of the enhancement factors for the membrane action

The multiplication factors e_{1b}, e_{2b}, e_{1m} and e_{2m} can be determined:

Table 5-16 Enhancement factors the assessment of the membrane action in Zone E

Equation	Obtained values
$e_{1b} = 2n\left(1+\alpha_1 b\frac{k-1}{2}-\frac{\beta_1 b^2}{3}\left(k^2-k+1\right)\right)+\left(1-2n\right)\left(1-\alpha_1 b-\beta_1 b^2\right)$	0.934
$e_{1m} = \frac{4b}{3+\left(g_0\right)_1}\frac{w}{d}\left(\left(1-2n\right)+n\frac{2+3k-k^3}{3\left(1+k\right)^2}\right)$	4.216
$e_1 = e_{1b}+e_{1m}$	5.150
$e_{2b} = 1+\frac{\alpha_2 bK}{2}\left(k-1\right)-\frac{\beta_2 b^2 K}{3}\left(k^2-k+1\right)$	0.988
$e_{2m} = \frac{4bK}{3+\left(g_0\right)_2}\frac{w}{d}\frac{2+3k-k^3}{6\left(1+k\right)^2}$	2.165
$e_2 = e_{2b}+e_{2m}$	3.153

Then, the global enhancement factor e is determined by:

$$e = e_1 - \frac{e_1 - e_2}{1+2\mu a^2} = 5.150 - \frac{5.150-3.153}{1+2\times 1.0\times 1.333^2} = 4.711$$

Step 8b: Total bearing capacity of the slab in fire condition

The total bearing capacity of the slab in fire condition taking into account the membrane action can be obtained from:

$$q_{fi,Rd,slab} = e\times p_{fi} = 4.711\times 1.512 = 7.123 \text{ kN/m}^2$$

Step 9b: Bearing capacity of the slab taking into account the contribution of the unprotected composite beams

Same as Step 9

Step 10b: Total bearing capacity of the slab in fire conditions and verification of the fire resistance of the slab

The total bearing capacity of the slab is:

$$q_{fi,Rd} = q_{fi,Rd,slab} + q_{fi,Rd,ub} = 7.12 + 0.17 = 7.29 \text{ kN/m}^2$$

With regards to the applied load on the slab in fire situation:

$$q_{fi,Sd} = 5.98 \text{ kN/m}^2 < q_{fi,Rd} = 7.29 \text{ kN/m}^2$$

Conclusion 3

In conclusion, the stability of the slab system is ensured for R60 with its actual dimensions in Zone E.

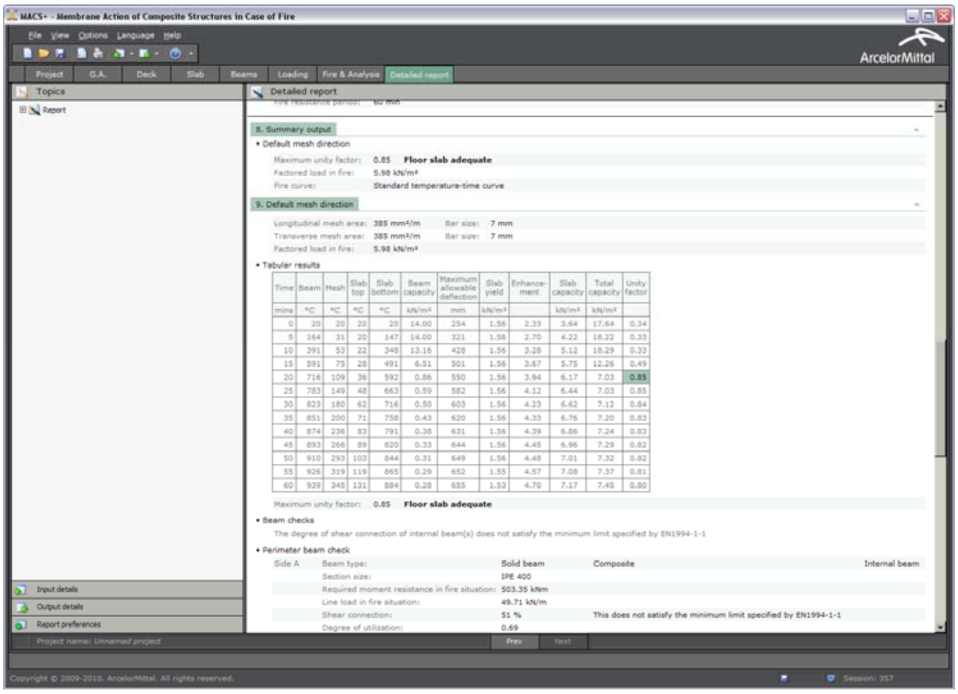


Figure 5-21 Output data using the MACS+ software – Detailed report

Step 11: Applied load in fire situation for perimeter beams

The applied loads in fire situation on the secondary beams and perimeter beams of Zone E are calculated as follows:

- For the secondary perimeter beams

$$M_{fi,Sd,b,1} = \frac{q_{fi,Rd} L_1^{\,2} L_2 - 8\left(M_{fi,0}\left(L_2 - n_{ub} b_{eff,ub} - \sum_{i=1}^{2} b_{eff,1,i}\right) + n_{ub} M_{fi,Rd}\right)}{c_M}$$

$$= \frac{7.29 \times 9^2 \times 12 - 8 \times \{6\,602.40 \times 10^{-3} \times [12 - 3 \times 2.25 - (2.25/2 + 2.25/2)] + 3 \times 5.1\}}{12}$$

$$= 567.08 \text{ kNm}$$

$$V_{fi,Sd,b,1} = \frac{4 M_{fi,Sd,b,1}}{L_1} = \frac{4 \times 567.08}{9} = 252.04 \text{ kN}$$

- For the primary perimeter beams

$$M_{fi,Sd,b,2} = \frac{q_{fi,Rd} L_1 L_2^{\,2} - 8\mu M_{fi,0}\left(L_1 - \sum_{i=1}^{2} b_{eff,2,i}\right)}{c_M} = \frac{7.29\times 9\times 12^2 - 8\times 1.0\times 6\,602.40\times 10^{-3}\times\left(9-\left(12/8+12/8\right)\right)}{12}$$

$$= 760.91 \text{ kNm}$$

$$V_{fi,Sd,b,2} = \frac{4M_{fi,Sd,b,2}}{L_2} = \frac{4\times 760.91}{12} = 253.64 \text{ kN}$$

So, the fire protection of this beam must be determined to ensure that the calculated bearing capacity in fire situation is not lower than the applied loads for the requested fire duration.

5.1.4 Floor design: Zone D

In Zone D, the dimensions of the composite slab and the spans of the beams have the same values as in Zone A. However, solid beams are replaced by IPE 270+IPE 270 Angelina^{TM} beams (see cross-section in Figure 5-23).

In consequence, only the load-bearing capacity of the unprotected beams needs to be determined.

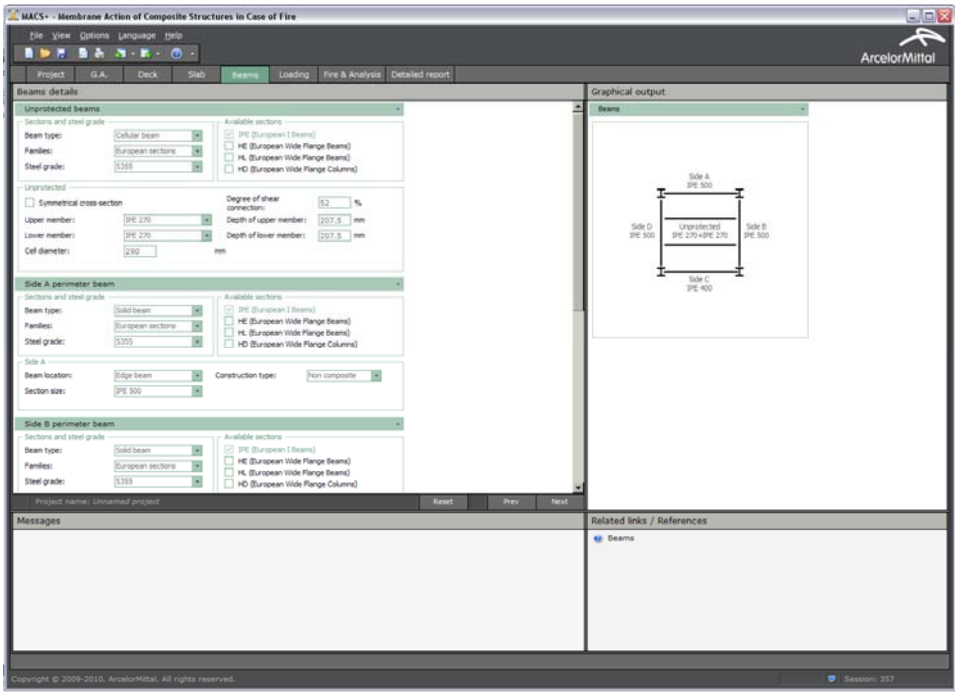


Figure 5-22 Input data using MACS+ software – Beams in Zone D

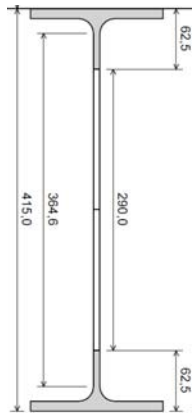


Figure 5-23 Net cross-section of Angelina beam in Zone D

Step 2: same as Zone E

Steps 3 to 8: same as Zone A

Step 9: Bearing capacity of the slab taking into account the contribution of the unprotected composite beams

The values of the section factors of the steel section are summarized in Table 5-17.

From Table 3-3, the temperatures of the steel part of the composite section are the following:

- temperature of the flanges: 941.0°C;
- temperature of the lower web: 942.2°C in Table 3-3 but taken as 941.0°C because the depth of the steel section is not greater than 500 mm;
- temperature of the upper web: 942.2°C;
- temperature of the studs (see 4.3.4.2.5 of EN 1994-1-2): 941.0×0.8 = 752.8°C.

Table 5-17 Section factor of the unprotected composite beam in Zone D

Steel section member	$k_{sh} = 0.9\left(\frac{0.5B_1 + t_{f1} + t_{f2} + \sqrt{h_w^{\ 2} + (B_1 - B_2)^2/4}}{H + B_1 + B_2/2 - (t_{w1} + t_{w2})/2}\right)$	$\left(\frac{A_i}{V_i}\right)$ (m^{-1})	$k_{sh}\left(\frac{A_i}{V_i}\right)$ (m^{-1})
Lower flange	0.711	$\frac{2(B_1 + t_{f1})}{B_1 t_{f1}} = 211$	150
Lower web		$\frac{2h_{w1} + t_{w1}}{h_{w1} t_{w1}} = 322$	229
Upper web		$\frac{2h_{w2} + t_{w2}}{h_{w2} t_{w2}} = 322$	229
Upper flange		$\frac{2(B_2 + t_{f_2})}{B_2 t_{f2}} = 211$	150
With: H: depth of the steel section; h_w: overall depth of the web; B_1: width of the lower flange; t_{f1}: thickness of the lower flange; t_{w1}: thickness of the lower web; h_{w1}: depth of the lower web (net cross-section); B_2: width of the upper flange; t_{f2}: thickness of the upper flange; t_{w2}: thickness of the upper web; h_{w2}: depth of the upper web (net cross-section).			

The temperatures of the steel section and of the steel studs allow determining the moment resistance of the internal non composite unprotected beams. For Cellular

Beams, the contribution of the lower member is neglected as its temperature exceeds 600°C. The calculated values are given in Table 5-18.

Table 5-18 Moment resistance for unprotected composite beams in Zone D

Parameters		Calculated values
Effective with of the slab		$b_{eff} = \min\{9\,000/4\,;3\,000\} = 2\,250$ mm
Area of the upper flange A_{f2}		$A_{f2} = 1377$ mm²
Area of the upper web A_{w2}		$A_{w2} = 229.0$ mm²
Reduction factor for the steel strength properties		$k_{y,\theta} = 0.052$
Reduction factor for the stud strength properties		$k_{u,\theta} = 0.17$
Tensile force	$T^+ = \sum A_i f_y k_{y,\theta} / \gamma_{M,fi,a}$	$T^+ = (1377 + 229) \times 355 \times 0.052/1.0$ $= 31.64$ kN
Thickness of the slab in compression in fire situation	$h_u = \dfrac{T^+}{b_{eff} f_c / \gamma_{M,fi,c}}$	$h_u = \dfrac{31.64}{2\,250 \times 25/1.0} = 0.562$ mm
Connection degree of the beam at 20°C		$n_{c,20°C} = 0.52$
Connection degree of the beam in fire situation	$n_{c,\theta} = \dfrac{n_{c,20°C} k_{u,\theta} \gamma_{M,v}}{k_{y,\theta} \gamma_{M,fi,v}}$	$n_{c,\theta} = \dfrac{0.52 \times 0.17 \times 1.25}{0.052 \times 1.0} = 2.04 > 1.0$ So full shear connection
Tensile force application point	$y_T = \dfrac{\sum A_i y_i f_y k_{y,\theta}}{T^+ \gamma_{M,fi,a}}$	$y_T = \dfrac{(229 \times 6.32 + 1377 \times 25.32) \times 355 \times 0.052}{31.64 \times 1.0}$ $= 403.66$ mm
Compressive force application point	$y_F = H + h_c - h_u/2$	$y_F = 415 + 130 - 0.562/2 = 544.72$ mm
Positive moment resistance	$M_{fi,Rd} = T^+(y_F - y_T)$	$M_{fi,Rd} = 31.64 \times (544.72 - 403.66)$ $= 4.46 \times 10^6$ Nmm $= 4.46$ kNm
With: h_c: total thickness of the slab; $\gamma_{M,fi,a}$, $\gamma_{M,v}$ and $\gamma_{M,fi,v}$ partial safety factor for the steel profile, the steel stud in normal conditions and in fire conditions.		

Then, the bearing capacity of the slab thanks to the contribution of the unprotected composite beam can be obtained from:

$$q_{fi,Rd,ub} = \frac{8M_{fi,Rd}}{L_1^2}\frac{1+n_{ub}}{L_2} = \frac{8 \times 4.46}{9^2} \times \frac{(1+2)}{9} = 0.15 \text{ kN/m}^2$$

Step 10: Total bearing capacity of the slab in fire conditions and verification of the fire resistance of the slab

The total bearing capacity of the slab is:

$$q_{fi,Rd} = q_{fi,Rd,slab} + q_{fi,Rd,ub} = 5.51 + 0.15 = 5.66 \text{ kN/m}^2$$

With regards to the applied load on the slab in fire situation:

$q_{fi,Sd} = 5.98$ kN/m² $> q_{fi,Rd} = 5.66$ kN/m²

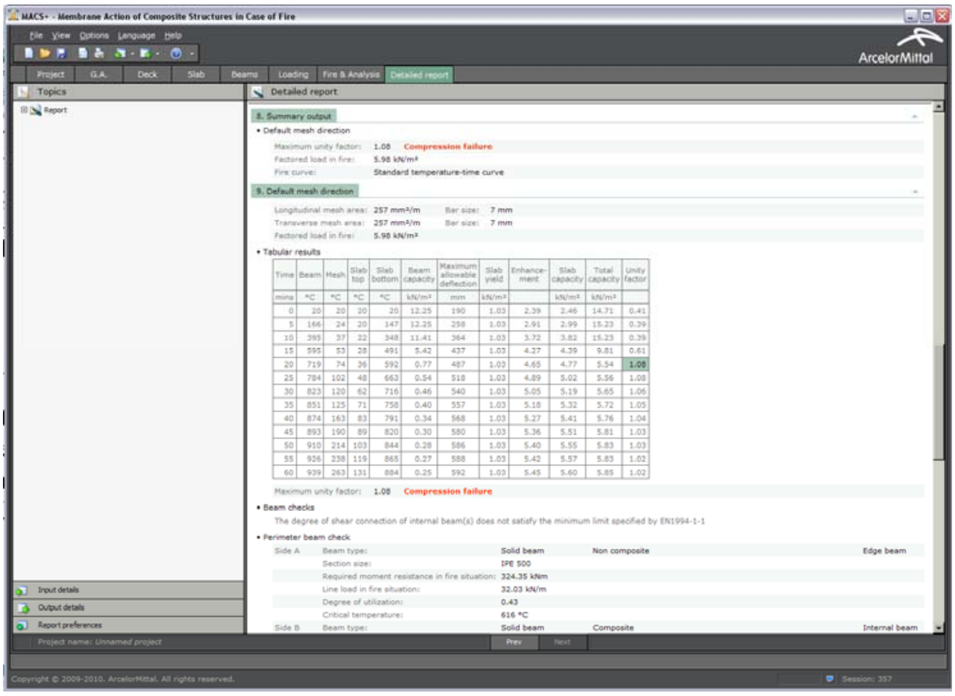


Figure 5-24 Output data using the MACS+ software – Detailed report

Conclusion 1

In conclusion, the stability of the slab system cannot be ensured for R60 with its actual dimensions in Zone D. So, it is necessary to modify the constructive parameters.

An adequate solution could be to increase or the mesh axis distance or the mesh size.

So, the mesh axis distance was increased from 30 mm to 40 mm, modifying the welded mesh temperature from 288 °C to 362 °C.

Step 2a

Following Table 3-4 of EN 1994-1-2, the effective steel strength for the welded steel mesh is reduced as follows:

$$f_{sy,\theta_s} = 500 \times 0{,}962 = 481 \text{ MPa}$$

Step 3a: Calculation of the moment resistance of the slab section $M_{fi,0}$

For this calculation zone:

L_1 = 9 000 mm (span of the secondary beams)

L_2 = 9 000 mm (span of the primary beams)

So, $L = \max\{L_1; L_2\}$ = 9 000 mm and $\ell = \min\{L_1; L_2\}$ = 9 000 mm.

It can be obtained:

$$(g_0)_1 = 1 - \frac{2KA_s\, f_{sy,\theta_s}/\gamma_{M,fi,s}}{0.85\, f_c/\gamma_{M,fi,c}\, d} = 1 - \frac{2\times 1.0\times \frac{257}{1\,000}\times 481/1.0}{0.85\times 25/1.0\times 40} = 0.709$$

$$(g_0)_2 = 1 - \frac{2A_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}}{0.85\, f_c / \gamma_{M,fi,c}\, d} = 1 - \frac{2\times 1.0 \times \frac{257}{1000} \times 481/1.0}{0.85 \times 25/1.0 \times 40} = 0.709$$

So, the positive moment resistance of the slab section is:

$$M_{fi,0} = A_s\, f_{sy,\theta_s} / \gamma_{M,fi,s}\, d\, \frac{3+(g_0)_2}{4} = \frac{257}{1000} \times 481/1.0 \times 40 \times \frac{3+0.709}{4} = 4\,586.51 \text{ Nmm/mm}$$

In parallel, it is also possible to determine the other necessary parameters:

$$\mu = K\, \frac{3+(g_0)_1}{3+(g_0)_2} = 1.0 \times \frac{3+0.709}{3+0.709} = 1.0$$

$$a = \frac{L}{\ell} = \frac{9\,000}{9\,000} = 1.0$$

$$n = \frac{1}{2\mu a^2}\left(\sqrt{3\mu a^2 + 1} - 1\right) = \frac{1}{2\times 1.0 \times 1.0^2} \times \left(\sqrt{3\times 1.0 \times 1.0^2 + 1} - 1\right) = 0.5$$

Step 4a: Determination of the reference bearing capacity of the slab

The reference bearing capacity of the slab can be determined from:

$$p_{fi} = 6\, \frac{M_{fi,0}}{n^2 a^2 \ell^2} = 6 \times \frac{4\,586.51}{0.427^2 \times 1.0^2 \times 9\,000^2} = 1.359 \times 10^{-3} \text{ N/mm}^2 = 1.359 \text{ kN/m}^2$$

Step 5a: same as Step 5

Step 6a: Calculation of the parameters to determine the membrane action

The determination of the different multiplication factors for the membrane action are based on the different parameters α_1, α_2, β_1, β_2, A, B, C, D, k and b that need to be determinedrofrom. The values of theses parameters are summarized in Table 5-19.

Table 5-19   Parameters used for the assessment of the membrane action in Zone D

Equation	Obtained values
$\alpha_1 = \frac{2(g_0)_1}{3+(g_0)_1}$	0.382
$\beta_1 = \frac{1-(g_0)_1}{3+(g_0)_1}$	0.078
$\alpha_2 = \frac{2(g_0)_2}{3+(g_0)_2}$	0.382
$\beta_2 = \frac{1-(g_0)_2}{3+(g_0)_2}$	0.078
$k = \frac{4na^2(1-2n)}{4n^2a^2+1}+1$	1.0
$A = \frac{1}{2(1+k)}\left[\frac{\ell^2}{8n} - \left(\frac{1-2n}{2n} + \frac{1}{3(1+k)}\right)\left((nL)^2 + (\ell/2)^2\right)\right]$	3 375 000 mm^2
$B = \frac{k^2}{2(1+k)}\left[\frac{nL^2}{2} - \frac{k}{3(1+k)}\left((nL)^2 + (\ell/2)^2\right)\right]$	3 375 000 mm^2
$C = \frac{\ell^2}{16n}(k-1)$	0 mm^2
$D = \frac{L^2}{8}(1-2n)^2$	0 mm^2
$b = \min\left[\frac{\ell^2}{8K(A+B+C-D)}, \frac{\gamma_{M,fi,s}}{kKA_s f_{sy,\theta s}}\left(0.85\frac{f_c}{\gamma_{M,fi,c}} \times 0.45d - A_s \frac{f_{sy,\theta s}}{\gamma_{M,fi,s}}\frac{K+1}{2}\right)\right]$	1.5

Step 7a: Calculation of the enhancement factors for the membrane action

The multiplication factors e_{1b}, e_{2b}, e_{1m} and e_{2m} can be determined:

Table 5-20 Enhancement factors the assessment of the membrane action in Zone D

Equation	Obtained values
$e_{1b} = 2n\left(1+\alpha_1 b\frac{k-1}{2}-\frac{\beta_1 b^2}{3}\left(k^2-k+1\right)\right)+\left(1-2n\right)\left(1-\alpha_1 b-\beta_1 b^2\right)$	0.941
$e_{1m} = \frac{4b}{3+\left(g_0\right)_1}\frac{w}{d}\left(\left(1-2n\right)+n\frac{2+3k-k^3}{3\left(1+k\right)^2}\right)$	3.917
$e_1 = e_{1b} + e_{1m}$	4.858
$e_{2b} = 1+\frac{\alpha_2 bK}{2}\left(k-1\right)-\frac{\beta_2 b^2 K}{3}\left(k^2-k+1\right)$	0.941
$e_{2m} = \frac{4bK}{3+\left(g_0\right)_2}\frac{w}{d}\frac{2+3k-k^3}{6\left(1+k\right)^2}$	3.917
$e_2 = e_{2b} + e_{2m}$	4.858

Then, the global enhancement factor e is determined by:

$$e = e_1 - \frac{e_1 - e_2}{1+2\mu a^2} = 4.858 - \frac{4.858-4.858}{1+2\times 1.0\times 1.0^2} = 4.858$$

Step 8a: Total bearing capacity of the slab in fire condition

The total bearing capacity of the slab in fire condition taking into account the membrane action can be obtained from:

$$q_{fi,Rd,slab} = e\times p_{fi} = 4.858\times 1.359 = 6.60 \text{ kN/m}^2$$

Step 9a: Bearing capacity of the slab taking into account the contribution of the unprotected composite beams

Same as Step 9

Step 10a: Total bearing capacity of the slab in fire conditions and verification of the fire resistance of the slab

The total bearing capacity of the slab is:

$$q_{fi,Rd} = q_{fi,Rd,slab} + q_{fi,Rd,ub} = 6.60 + 0.15 = 6.75 \text{ kN/m}^2$$

With regards to the applied load on the slab in fire situation:

$q_{fi,Sd} = 5.98$ kN/m² < $q_{fi,Rd} = 6.75$ kN/m²

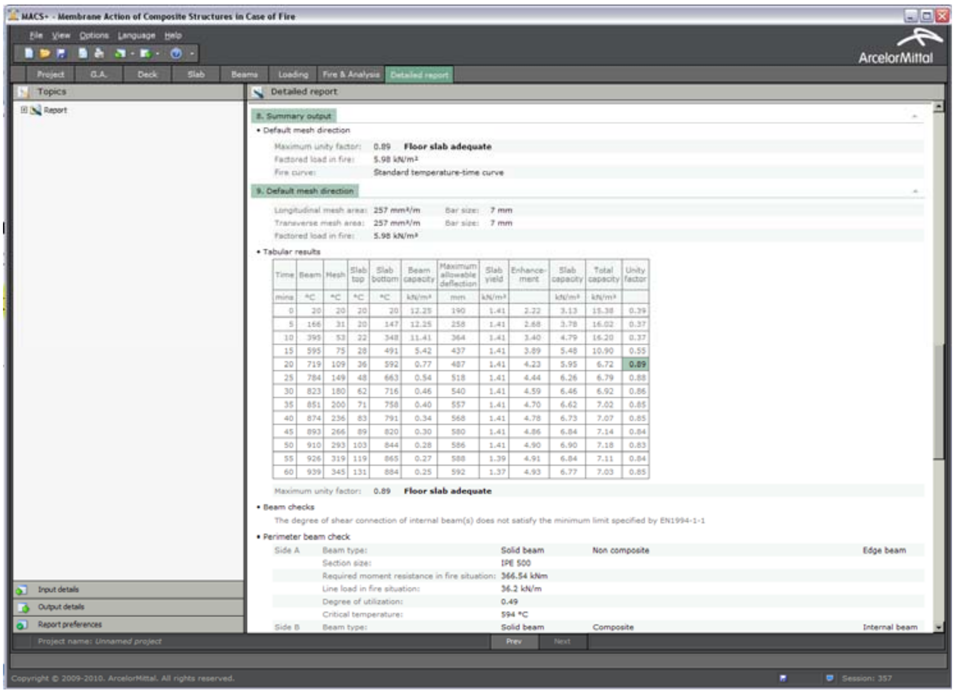


Figure 5-25 Output data using the MACS+ software – Detailed report

Conclusion 2

In conclusion, the stability of the slab system is ensured for R60 with its actual dimensions in Zone D.

Step 11: Applied load in fire situation for perimeter beams

The applied loads in fire situation on the secondary beams and perimeter beams of Zone D are calculated as follows:

- For the secondary perimeter beams

$$M_{fi,Sd,b,1} = \frac{q_{fi,Rd} L_1{}^2 L_2 - 8\left(M_{fi,0}\left(L_2 - n_{ub} b_{eff,ub} - \sum_{i=1}^{2} b_{eff,1,i}\right) + n_{ub} M_{fi,Rd}\right)}{c_M}$$

$$= \frac{6.75 \times 9^2 \times 9 - 8 \times \{4\,586.51 \times 10^{-3} \times [9 - 2 \times 2.25 - (0 + 2.25/2)] + 2 \times 4.5\}}{12}$$

$$= 393.74 \text{ kNm}$$

$$V_{fi,Sd,b,1} = \frac{4 M_{fi,Sd,b,1}}{L_1} = \frac{4 \times 393.74}{9} = 175.00 \text{ kN}$$

- For the primary perimeter beams

$$M_{fi,Sd,b,2} = \frac{q_{fi,Rd} L_1 L_2^{\ 2} - 8\mu M_{fi,0}\left(L_1 - \sum_{i=1}^{2} b_{eff,2,i}\right)}{c_M} = \frac{6.75\times9\times9^2 - 8\times1.0\times4\,586.51\times10^{-3}\times(9-(9/8+9/8))}{12}$$

$$= 389.42 \text{ kNm}$$

$$V_{fi,Sd,b,2} = \frac{4M_{fi,Sd,b,2}}{L_2} = \frac{4\times389.42}{9} = 173.08 \text{ kN}$$

One of the perimeter beams of this zone is an edge beam at the façade level, it must support an additional load coming from the façade elements of 2.0 kN/m, which implies a modification of the applied load in fire condition following the next relations:

$$M_{fi,Sd,b,1} = 393.74 + \frac{2.0\times9^2}{8} = 414.00 \text{ kNm}$$

$$V_{fi,Sd,b,1} = 175.00 + \frac{2.0\times9}{2} = 184.00 \text{ kN}$$

So, the fire protection of this beam must be determined to ensure that the calculated bearing capacity in fire situation is not lower than the applied loads for the requested fire duration.

5.2 Reinforcement details

Since the output confirms that the load bearing capacity of zones A and B are both adequate, the ST 25C mesh provided is adequate for fire design.

This mesh has an area of 257 mm^2/m in both directions and has 7 mm wires spaced at 150 mm centres in both directions.

The mesh in this example has a yield strength of 500 N/mm^2. For fire design the Class of reinforcement should be specified as Class A in accordance with EN 10080.

At joints between sheets the mesh must be adequately lapped in order to ensure that its full tensile resistance can be developed in the event of a fire in the building. For the 7 mm diameter bars of the ST 25C mesh the minimum lap length required would be 300 mm, as shown in Table 3-3. In order to avoid the build up of bars at lapped joints, sheets of mesh with flying ends should be specified as shown in Figure 3-5.

Additional reinforcement in the form of U-shaped bars should be provided at the edge beams to ensure adequate tying between these beams and the composite slab.

5.3 Fire protection of columns

Fire protection should also be specified for all of the columns in this example. The following information should be provided when specifying the fire protection.

Fire resistance period	60 minutes
Section size	HD320×158
Section Factor	63 m^{-1} box protection heated on 4 sides
	89 m^{-1} profiled protection heated on 4 sides

Critical temperature	500°C or 80ºC less than the critical temperature calculated on the basis of the EN 1993-1-2 design rules, whichever is the lower.

The applied fire protection should extend over the full height of the column, up to the underside of the composite floor slab.

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Design Guide

Version 2012-1