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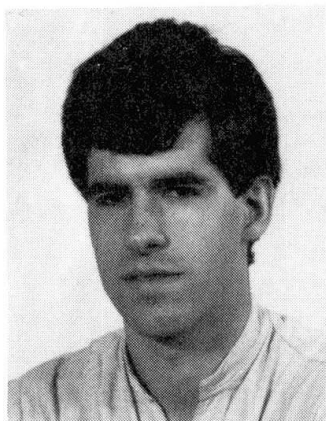
## Numerical Model for Fire-Exposed Composite Slabs

Modèle numérique pour planchers mixtes soumis au feu

Ein numerisches Verfahren zur Berechnung  
brandbeanspruchter Verbunddecken

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### SUMMARY

This paper deals with the influence of fire-exposure on the behaviour of composite steel-concrete slabs. A numerical model describing such behaviour has been developed. The model comprises a thermal model, a mechanical model for cross-sectional analysis and a mechanical model for structural analysis.

### RÉSUMÉ

Cet article traite du comportement au feu de planchers mixtes béton-acier. Un modèle de simulation numérique a été mis au point pour ce type de sollicitation. Il comprend un modèle thermique, un modèle de comportement mécanique en section et un modèle d'analyse structurale.

### ZUSAMMENFASSUNG

Dieser Text behandelt den Einfluss einer Brandbeanspruchung auf Verbunddecken aus Stahl und Beton. Ein numerisches Modell zur Beschreibung dieses Verhaltens wurde entwickelt. Dieses Verfahren beinhaltet ein thermisches Modell zur Querschnittsanalyse und ein mechanisches Modell für die Statik.



## 1. INTRODUCTION

Structures that are applied in buildings have to meet fire-safety requirements. Normally these structures are judged on the basis of standard fire tests. However, such tests are very expensive and time-consuming, for which reason tools have been designed to determine the fire resistance of steel and concrete structures by calculation [1], [2]. Recommendations for composite steel/concrete slabs, cf. Fig. 1, were introduced in 1983 [3]. With these recommendations the fire resistance of composite slabs can be quickly and simply calculated. Nevertheless, a mathematical model is needed, by means of which the behaviour of fire-exposed composite slabs can be predicted on a more fundamental basis. Such a model has been developed at Eindhoven University of Technology.

The model comprises a thermal model, by which means temperature profiles are calculated as a function of fire-exposure time ([4], [5]), and a mechanical model by which means the mechanical response of the slab structure is calculated [6].

The mechanical model is divided into two submodels:

- 1 a submodel for analysing the cross-sectional behaviour, providing moment-curvature diagrams as a function of time, and
- 2 a submodel for analysing the structural behaviour, providing moment distributions and deflections as a function of time.

In the scope of an international research program, which is currently being carried out, the developed models will be verified and adapted, if necessary. This research, which is co-sponsored by ECSC and carried out at TNO Institute for Building Materials and Structures, comprises tests for studying the thermal behaviour and loading tests on whole systems [7].

After verification of the model, practical design rules will be developed, for simple and rapid determination of the fire resistance. These rules will be proposed as European recommendations.

## 2. THERMAL ANALYSIS

A two-dimensional thermal model for the determination of temperature profiles in fire-exposed composite steel/concrete slabs has been developed. This model has been based on the finite difference method. It takes temperature-dependent material properties, evaporation of moisture, as well as arbitrary fire-exposure conditions into account. For a detailed description of the thermal model, see references [4] and [5].

## 3. CROSS-SECTIONAL ANALYSIS

At room temperature, the cross-sectional behaviour of composite slabs is determined by the geometry and the mechanical properties of the components (profiled steel sheet, concrete and possibly reinforcement) and the method of construction (application of props). This behaviour is analysed on the basis of nonlinear elastic theory, assuming that plane sections remain plane after bending (Bernoulli) and a complete interaction between concrete and steel (no slip).

The analysis yields an  $M-\kappa$  diagram, comprising both positive and negative bending as well as ascending and descending branches, cf. Fig. 2 (full line). The plastic moments  $M_p^-$  and  $M_p^+$  as well as the flexural stiffnesses  $EI^-$  and  $EI^+$  at room temperature (RT) have been indicated.

In comparison with room temperature, the cross-sectional behaviour of fire-exposed composite slabs changes due to the nonuniform temperature distribution in the cross-section. In each fibre of the cross-section, the temperature increase causes a thermal expansion and a decrease in strength and stiffness. In consequence of these changing material properties, thermal strains and stresses as well as a thermal curvature develop during fire exposure.

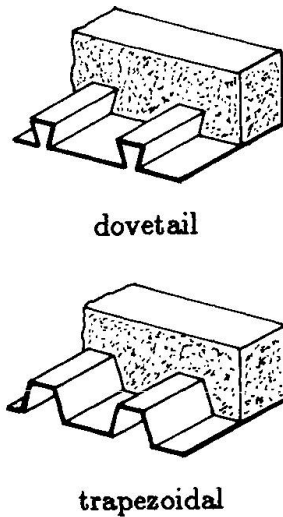


Fig. 1 Composite steel/concrete slabs

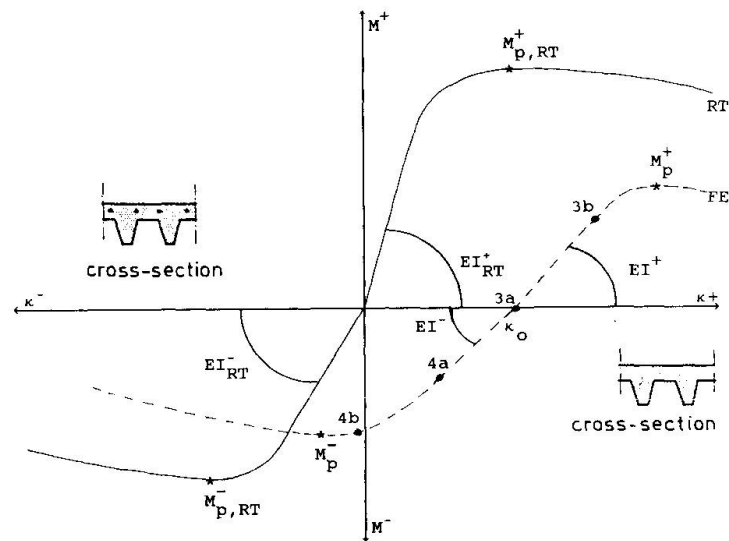


Fig. 2 The  $M-\kappa$  diagram for positive and negative bending Comparison of room temperature (RT) and fire-exposure (FE) conditions

The nonuniform temperature distribution causes thermal strains that vary in two directions. For simplicity, only one dimension, the variation of strains with the height of the cross-section, is considered in this paragraph.

First, a nonloaded cross-section is considered ( $M=0$ ). Owing to the nonuniform temperature distribution, a nonlinear (thermal) strain distribution would develop if all fibres could expand freely ( $\epsilon_t = \alpha \cdot T$ ). However, according to Bernoulli's hypothesis, a linear strain distribution is assumed, partly restraining the thermal strains. This restraint causes thermal stresses, cf. the hatched area in Fig. 3.a. In the upper and lower parts of the cross-section compressive stresses occur, in the middle part tensile stresses. A thermal curvature  $\kappa_0$  develops, for which the equilibrium conditions ( $\Sigma H=0$  and  $\Sigma M=0$ ) are fulfilled.

Second, a positive bending moment ( $M>0$ ) is considered. The curvature  $\kappa$  increases beyond  $\kappa_0$ , cf. Fig. 3.b. The stress-related strains increase accordingly, compressive strains in the upper parts and tensile strains in the lower parts, cf. the hatched area in Fig. 3.b. The stress-related strains  $\epsilon^\sigma$  are defined as the difference between the strains (according to the linear strain distribution) and the free thermal strains ( $\alpha \cdot T$ ).

Third, a negative bending moment ( $M<0$ ) is considered. The curvature  $\kappa$  decreases below  $\kappa_0$ . First, between  $\kappa=\kappa_0$  and  $\kappa=0$ , a negative bending moment corresponds to a positive curvature cf. Fig. 4.a. Then, below  $\kappa=0$ , a negative bending moment corresponds to a negative curvature cf. Fig. 4.b. The stress-related strains increase accordingly, compressive strains in the lower parts and tensile strains in the upper parts, cf. the hatched area in Fig. 4.

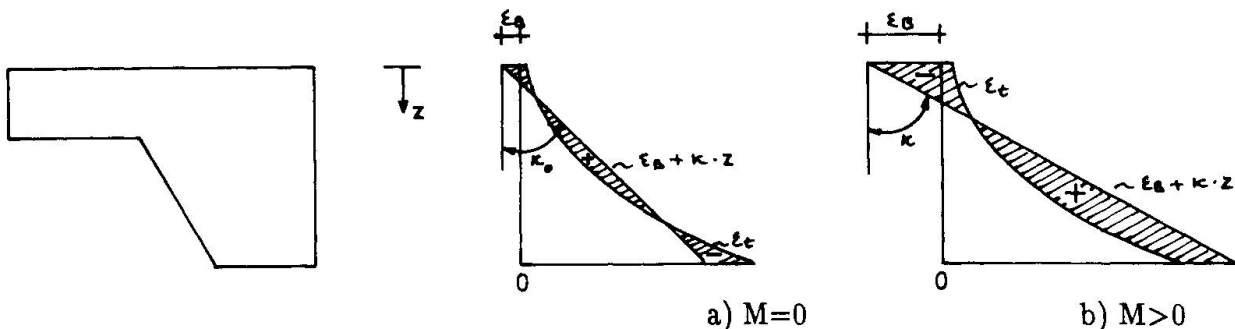


Fig. 3 Strain distribution in cross-sections without load (a) and at positive bending (b)

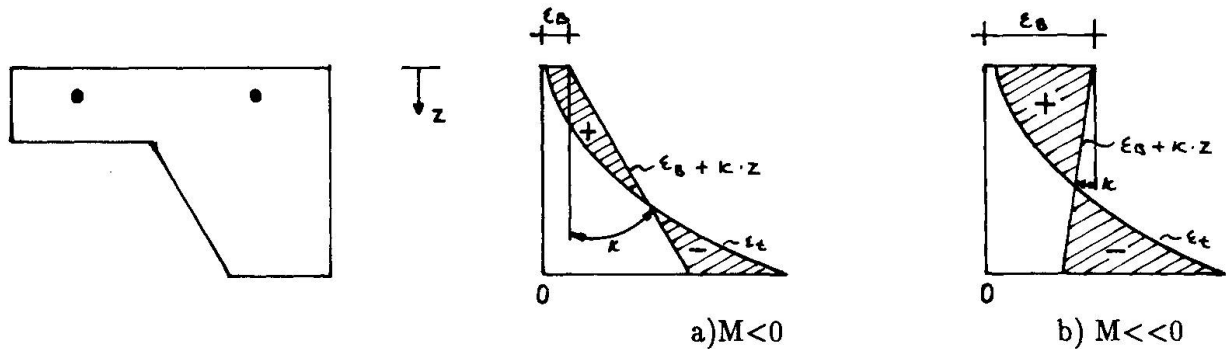


Fig. 4 Strain distribution in cross-sections at negative bending

A characteristic  $M-\kappa$  diagram under fire-exposure is compared with that at room temperature, cf. Fig. 2 (dotted line vs. full line). The points qualitatively described in Fig. 3 and 4 are indicated in Fig. 2. Plastic moments ( $M_p^-$  and  $M_p^+$ ) and flexural stiffnesses ( $EI^-$  and  $EI^+$ ) decrease as a function of fire-exposure time, while  $\kappa_0$  increases.

#### 4. STRUCTURAL ANALYSIS

On the basis of the  $M-\kappa$  diagrams, generated in the cross-sectional analysis, the structural behaviour is determined, yielding moment distributions and deflections as a function of time. Both statically determinate and statically indeterminate slabs can be analysed.

The determination of statically determinate slabs is fairly simple, as the moment distribution can be directly derived from the loading and the span. Therefore, the curvature distribution can be determined with the  $M-\kappa$  relations. From this, the deflections can be easily calculated. In Fig. 5 the hypothetical case of linear elastic behaviour is presented. During fire-exposure the stiffness  $EI$  decreases, causing larger deformations. Furthermore, deformations occur due to  $\kappa_0$ , that increases during fire-exposure. However, as the stiffness varies along the span, a nonlinear elastic model has to be applied.

Determination of the behaviour of statically indeterminate slabs is more complicated, because the moment distribution cannot be directly derived from the structural parameters (number and length of the spans, loading and supporting conditions). Moment distribution also depends on the stiffness distribution in the slab.

At room temperature a redistribution of moments takes place when the loading increases. For composite slabs, the nonlinear behaviour concentrates at the internal support sections. Accordingly, the stiffness of these sections decreases, yielding a relatively smaller contribution by the support moments to the load-bearing capacity, cf. Fig. 6. At a certain load level, the negative plastic moment at the mid support is reached. Depending on the rotational capacity at this support, loading can be further increased, during which the moments at mid span may increase until the positive plastic moment is reached.

During fire-exposure, a redistribution of moments also takes place, although the load remains the same, cf. Fig. 7. Owing to the temperature increase, the flexural stiffnesses decrease and the thermal curvature  $\kappa_0$  increases, cf. Fig. 2. Especially because of the latter effect, the support moment increases as a function of time, in contrast to the (relative) decrease during increased loading at room temperature, cf. Fig. 6 and 7.

In Fig. 8 the behaviour of a double-span continuous slab is schematically presented for the hypothetical case of linear elastic behaviour (constant stiffness of the whole slab). For reasons of symmetry, this static system can be substituted by a uniformly loaded simply supported slab with a support moment  $M_s$  acting at one support. A distinction is made between deformations due to the thermal curvature  $\kappa_0$ , to the loading  $q$  and the support moment  $M_s$ .

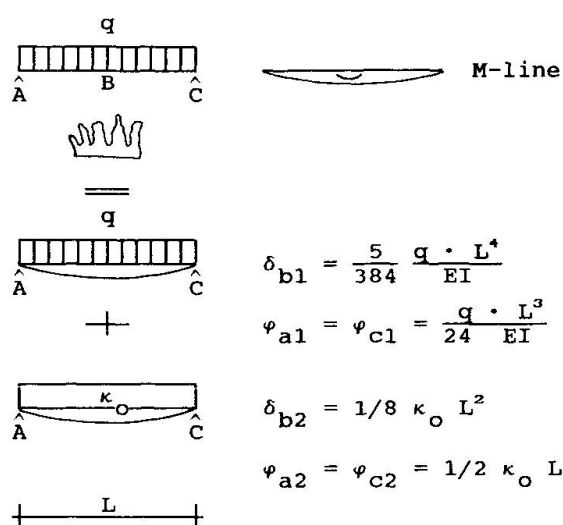


Fig. 5 Simply supported slab  
(linear elastic behaviour)

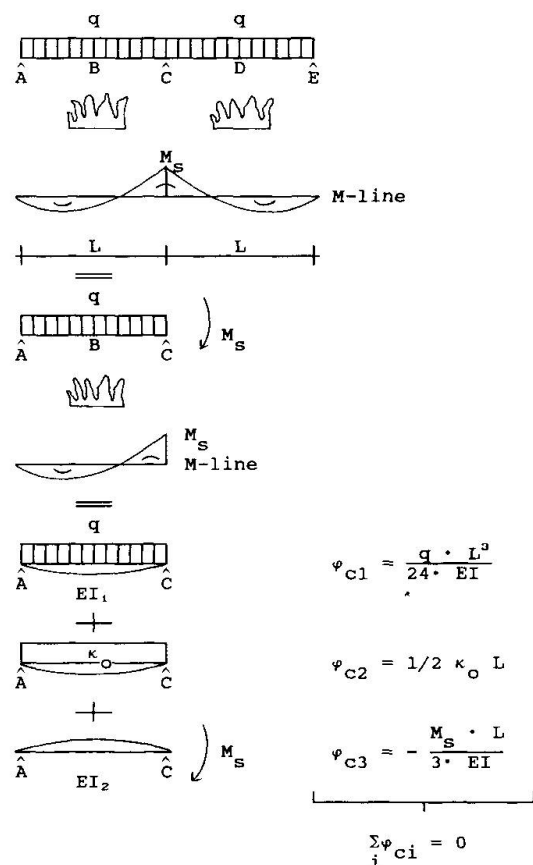


Fig. 8 Continuous slab  
(constant stiffness of  
the whole slab)

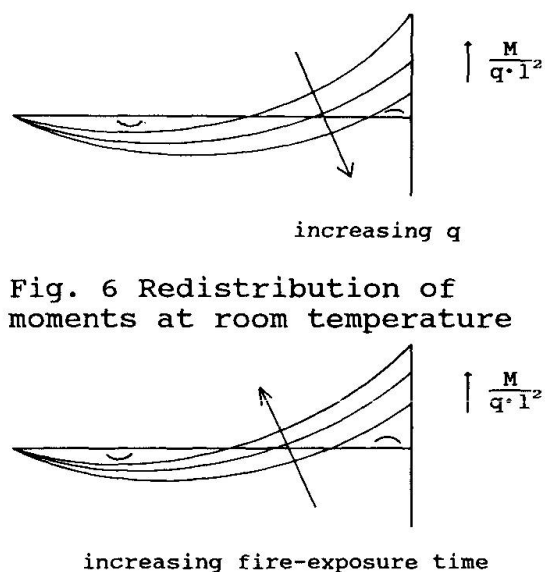


Fig. 6 Redistribution of moments at room temperature

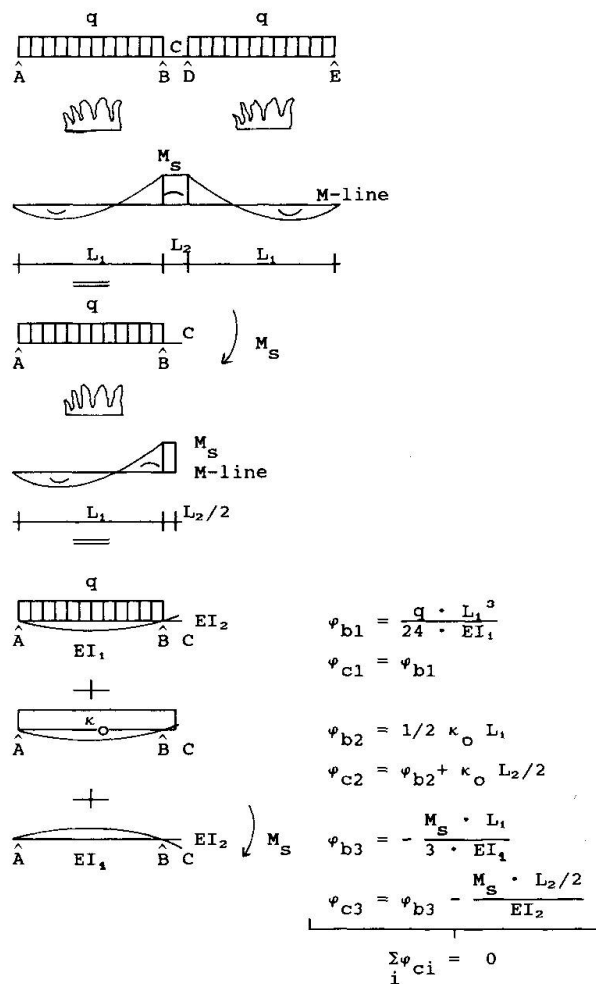


Fig. 9 Continuous slab with a constant moment area (constant stiffnesses of each span)



Compatibility requires the rotation at the mid support to be 0.

At an early stage of the fire,  $\kappa_0$  has increased to such an extent that  $M_s$  reaches the negative plastic moment. Hence, analogously to room-temperature conditions, the rotational capacity of the support section is decisive for the load-bearing capacity. From negative bending tests at room temperature it is known that the rotational capacity can be primarily attributed to one (or two) major crack(s) appearing at the mid-support section [8]. The stated compatibility condition ( $\Sigma \varphi_{\text{support}} = 0$ ) does not hold any longer. Therefore the static system of Fig. 8 is no longer valid after major crack formation.

It has been shown that the rotational capacity of the mid-support section can, under circumstances, be satisfactorily predicted assuming a constant moment area of approximately 100 mm at the mid support [8]. Under fire conditions, the rotational capacity is also simulated by choosing an appropriate length of the constant moment area.

The rotational capacity problem of a double-span continuous slab has thus been schematized as a triple-span continuous slab of which the (small) fictitious span represents the mid-support section of the actual slab, providing the rotational capacity, cf. Fig. 9. For symmetry reasons this static system can be substituted by a simply supported slab AB with a cantilever BC and a moment  $M_s$  at the end of the cantilever C.  $M_s$  is the constant moment acting at the mid-support section BD of the triple-span slab. Actual values of the length of the constant moment area are to be derived from calculations with a discrete crack model [9] and from experiments planned for the near future.

The hypothetical case of constant stiffnesses for each span does not occur. The stiffness varies along the span, which is the reason why a nonlinear elastic model has to be applied. Furthermore, an iteration process is necessary, in which the support moment and the stiffness distribution are determined in such a way that the compatibility condition is fulfilled.

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