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DESIGN OF FIRE EXPOSED CONCRETE STRUCTURES

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Josef Monier

PREFACE

Josef Monier was the first to use an iron netting imbedded in concrete, and he can thus be considered as the father of reinforced concrete constructions.

In his planters and garden structures the reinforcement was always placed in the middle of the cross-sections, and it is told that he was faithful to this point of view until he died in great misery in 1906.

Already within his lifetime other engineers developed the basic principles for construction in reinforced concrete and placed the reinforcement where it contributed most efficiently to the loadbearing capacitie of the cross-sections.

More and more slender concrete structures were developed and the protection of the reinforcing bars against influences from the environment became a problem of increasing significance.

Especially the influence of fire may lead to cover thicknesses indicating that Monier was almost right.

This report deals with the design of concrete structures for fire exposure.

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SUMMARY

A brief description is given dealing with the changes of the material properties of concrete exposed to high temperatures, and it is illustrated by curves showing the development of the residual compressive strength, the modulus of elasticity and Poisson's ratio for a typical Danish concrete.

A simple relation is indicated between the reductions due to heat exposure of the compressive strength and the modulus of elasticity of concrete, and idealized stress-strain curves are proposed.

A stress distribution factor is introduced, relating the mean value of the compressive strength of the concrete through a cross-section exposed to fire on the two sides to the reduction at the centre line.

Formulas for the ultimate limit state analyses at any time of any fire are developed for rectangular beams subjected to bending with the compression zone at the top and at the bottom respectively and subjected to shear, for slabs, T-shaped cross-sections, walls and rectangular columns with four sides exposed to fire, or one or more sides insulated.

The procedure is described for designing a structure for a certain fire exposure, and methods are indicated for converting a stress-distribution factor from one concrete quality or load to another.

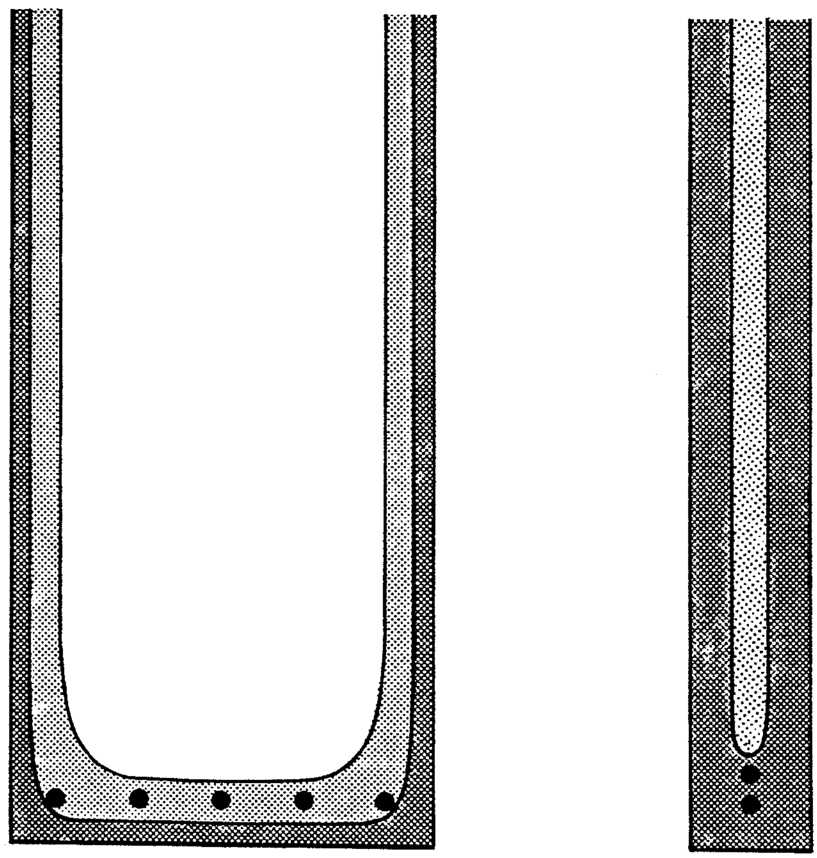
SYMBOLS

A	cross-section area
C	thickness of a cross-section
C_i	constant
c	modified thickness of a column
d	depth
E	modulus of elasticity
F	force
I	moment of inertia
ℓ	effective length of a column
M	moment
M'	moment reversed to M
n	ratio of the moduli of elasticity for steel and concrete respectively
T	temperature
t	time
V	shear force
x	depth of a neutral axis
y	depth of a compressive stress block
z	co-ordinate
ε	strain
ε_c'	ultimate compressive strain of concrete
ε_s	strain of steel
η	stress distribution factor
ν	Poisson's ratio
ξ_c'	reduction of ultimate compressive stress of concrete

ξ_s	reduction of yield stress of steel
σ	stress
σ'_c	ultimate compressive stress of concrete
σ_s	ultimate stress of steel
φ	ratio of reinforcement

Indices:

B	bottom of cross-section
c	concrete
cr	critical
i	index number
M	at the middle of a cross-section exposed on two sides
s	steel
T	top of cross-section
20	at $20^{o}C$



< 300°C 300-600°C > 600°C

Temperature distributions after 1 h. exposure to a standardfire.

INTRODUCTION

Ordinary concrete decrepitates by exposure to high temperatures.

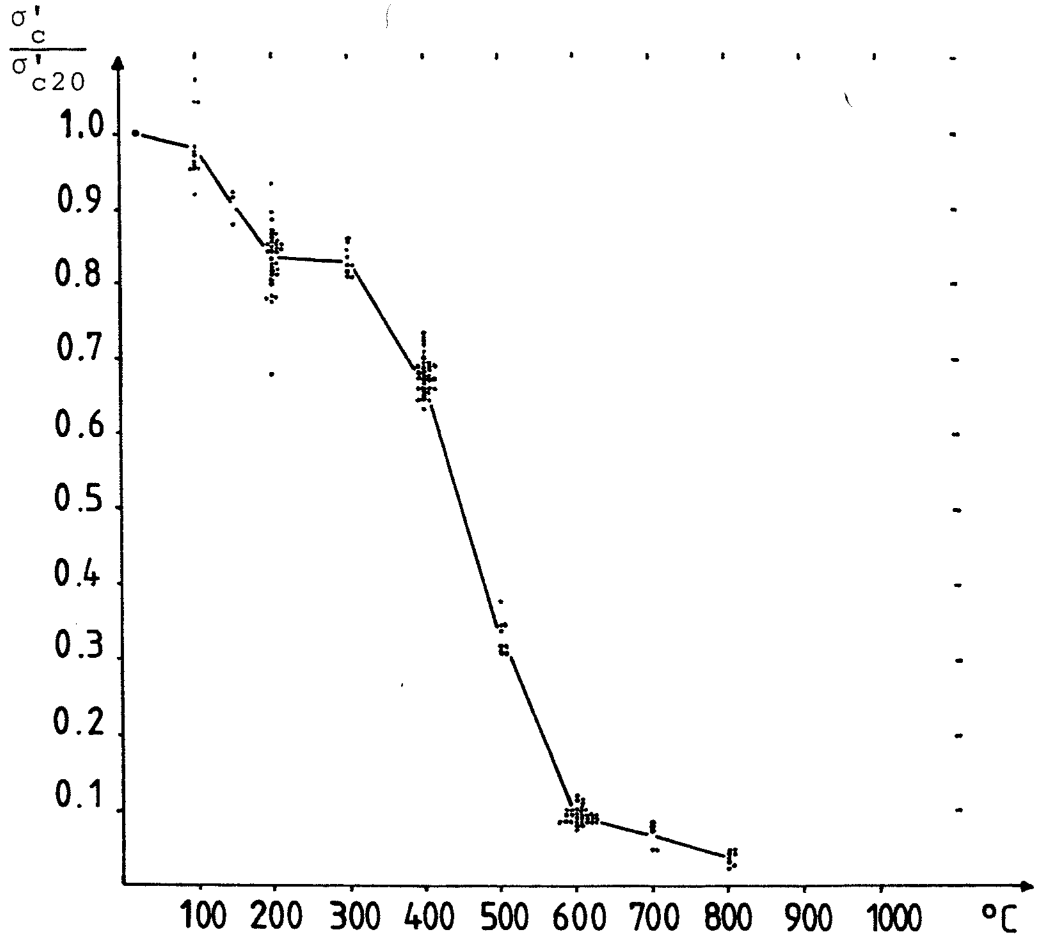
The reason why we have got the notion that concrete structures generally are less damaged than for example unprotected steel structures when exposed to fire, is not the heat resistance of the concrete, but its large heat capacity and relatively low thermal conductivity and the fact that concrete structures traditionally have been designed with large cross-sectional dimensions.

This means, that these cross-sections will only be heated to some extent at a rather thin surface layer, which is often about 5 to 10 centimetre in depth, and that the damages are limited to this layer.

However, the development in structural engineering leads to still more optimal constructions, and therefore the cross-sections used becomes more and more slender.

The more the materials are utilized statically, the more sensitive are the structures to various influences from the environment. One of these is the possible fire exposure.

The design of concrete structures will therefore be more and more dependent on the influence of fire, and the need for fast and precise methods for the theoretical design of fire exposed structures will increase. These methods must be based on a knowledge of the material properties during and after exposure to high temperatures, and test methods for determining these properties must be standardized.



Residual ultimate stress of concrete with Danish sea gravel, w/c = 0.87, σ'_{c20} = 19.5 MPa.

MATERIAL PROPERTIES

The decrepitation of concrete exposed to high temperatures is treated in more details in Hertz [6]. Only the most important causes of this effect are summarized in this context.

While concrete is heated the free water evaporates, and above approx. 150°C the water chemically combined in the hydrated calcium silicates starts to release.

This causes a shrinkage of the hydrated cement paste, while the aggregate and the reinforcing bars expand.

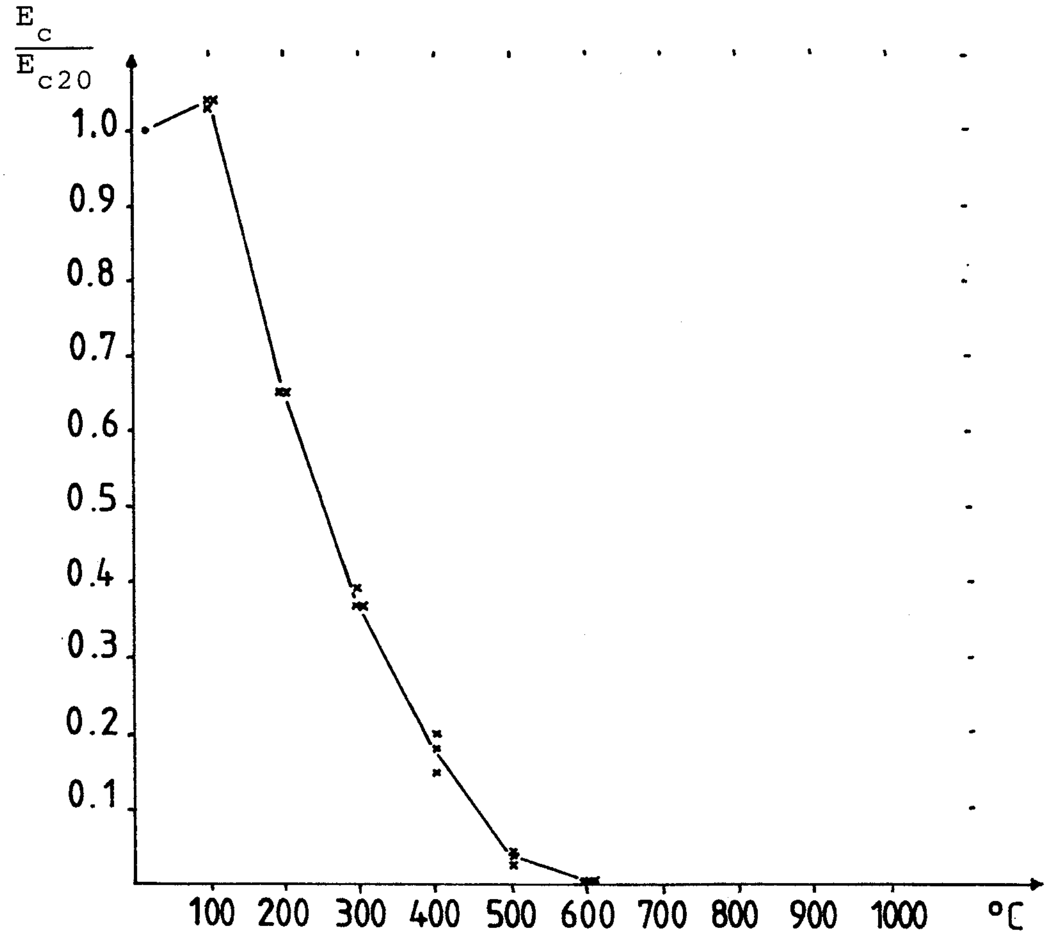
Stresses will therefore develop in the composite material, and from approx. 300°C microcracks pierce through the matrix.

The microcracks will cause a decrease of the compressive strength, the tensile strength and the modulus of elasticity and unloaded specimens will be subject to an irreversible expansion.

Above approx. 400°C the crystals of calcium hydroxide will start to decompose in calcium oxide and water, a process that reaches its highest intensity about 535°C.

This causes a weakening of the concrete, but while cooling, and during the first seven days after the heat exposure, the calcium oxide absorps water from the air giving rise to an expansion, that opens the cracks, which are already formed.

The weakening of the concrete is thus dependent on the temperature level, the load, the aggregate used and the amount of calcium hydroxide in the matrix.



Residual modulus of elasticity of concrete with Danish sea gravel, w/c = 0.87, E_{c20} = 27.9 GPa.

If curves for the development of the modulus of elasticity in temperature are compared to curves for the development of compressive strength in temperature for the same materials exposed to the same temperature -time course, and applied to the same load, it seems to be valid with a good approximation, that the reduction of the modulus of elasticity is the square of the reduction of the compressive strength. (See for example the 168 curves in Hertz [6]).

During an investigation comprising testing of the compressive strength of 230 cylindrical concrete specimens heated to various temperatures, the author has noticed a considerable increase of the ultimate strain in temperature. (The tests are described in more details in Hertz [6] and [5]).

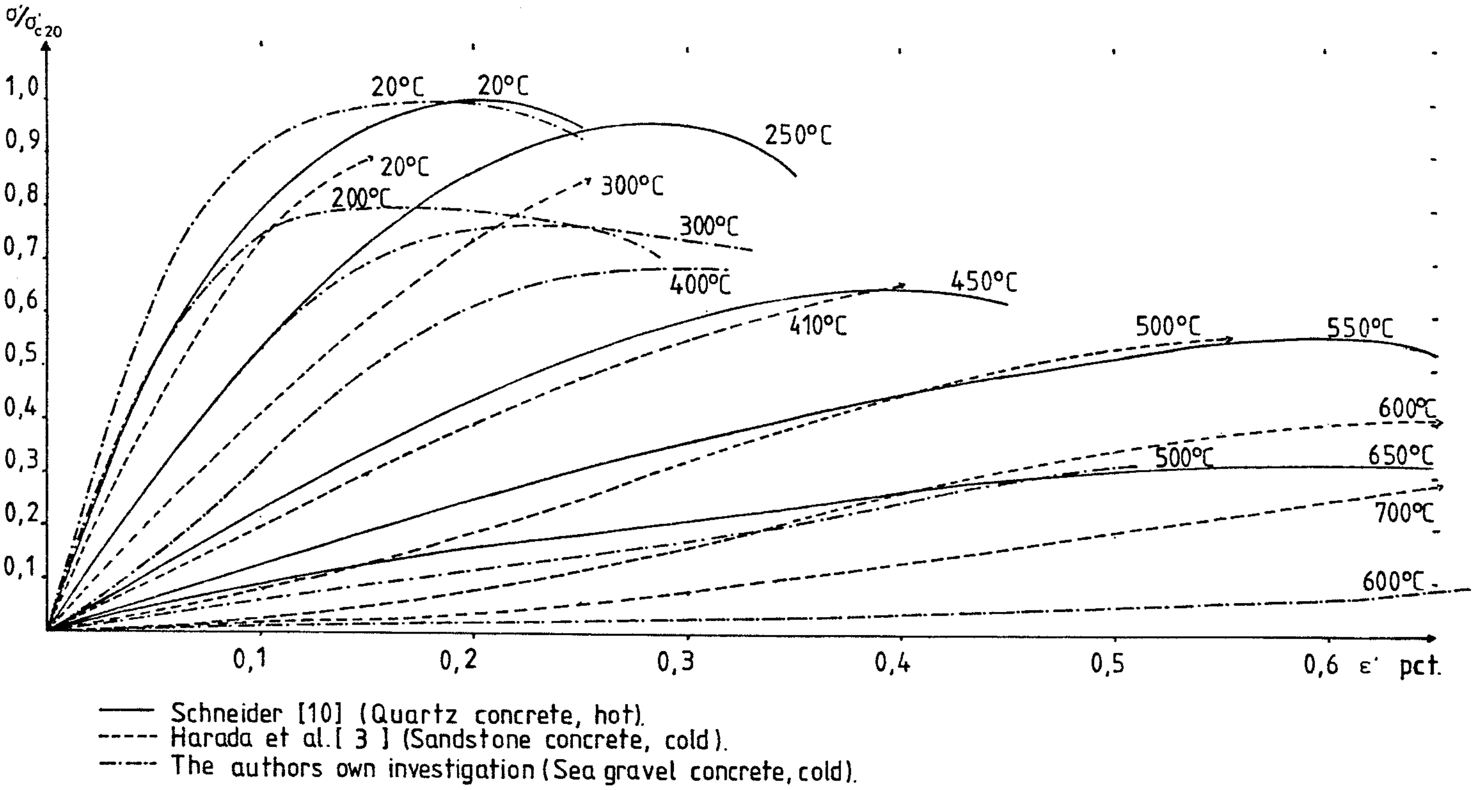
The same observation has been made from the stress -strain curves for various concretes with - or without application of load during heating and tested in a hot- or a cold condition, reported in the literature (see for example Schneider [10], Harmathy and Berndt [4], Harada et al. [3] and Fischer [2]) and the curves drawn upon the authors investigation.

From the stress-strain curves it can also be seen that the increase in strain follows the decrease in stress, and the simple model is proposed that the product of stress and strain remains constant for each point of the stress-strain curve for the concrete while the material is weakened due to heat.

This means that

$$\sigma\varepsilon = C_i$$

where C_i is a constant for each point of the stress-



Stress-strain curves from various investigations.

strain curve.

Especially, if ξ_c' is the reduction of the ultimate compressive stress, we have

$$\sigma_c' = \xi_c'\sigma_{c20}'$$

where the index "20" denotes the initial value before the heat exposure and

$$\varepsilon_c' = \frac{\varepsilon_{c20}'}{\xi_c'} = \frac{0.35\%}{\xi_c'}$$

And from idealized stress-strain curves the relation already mentioned appears:

$$E_c = (\xi_c')^2 E_{c20}$$

The many observations of this relationship between the reduction of the compressive strength and the modulus of elasticity support the theory well.

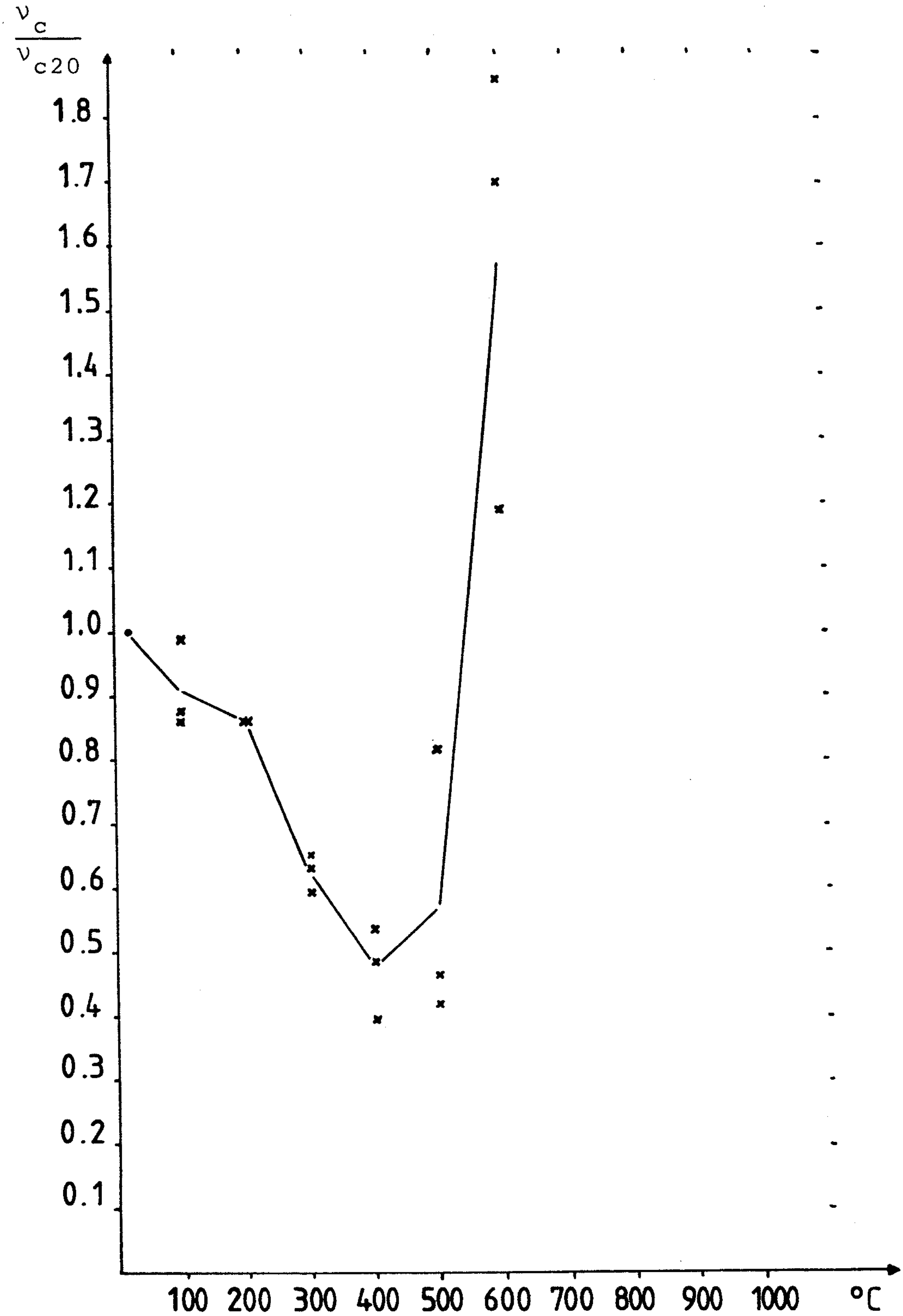
If corresponding idealizations should be made to facilitate the calculational treating of the stress-strain curve for the reinforcement, it must be remembered that the weakening of the steel is not caused by an increased porosity, but a relaxation of the crystalline bonds when heated.

The stress-strain curves for the heated steel can as a rough and simple approximation be considered to appear by affinities in the strain axis.

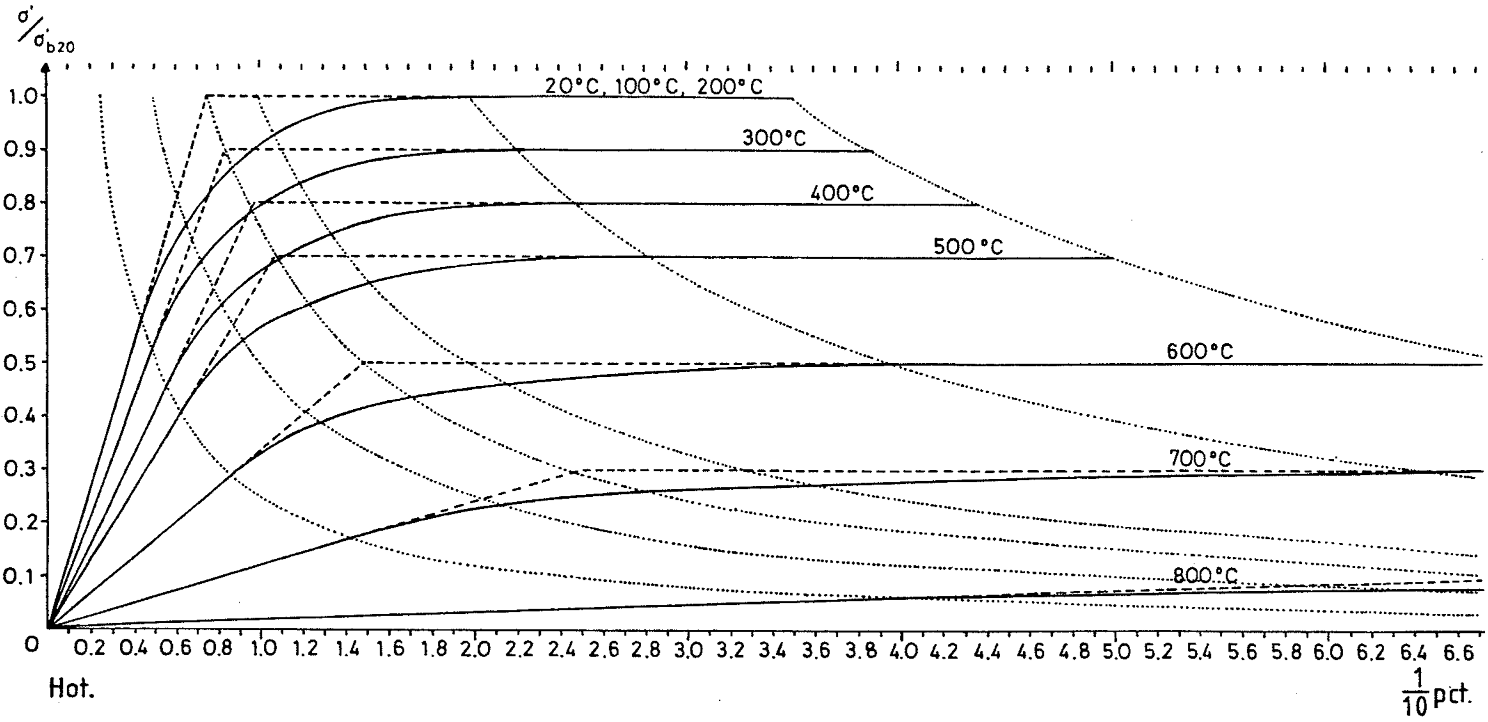
The reduction ξ_s of the yield stress σ_s is thus the same as the reduction of the modulus of elasticity, which is a reasonable approximation for most steels, i.e.

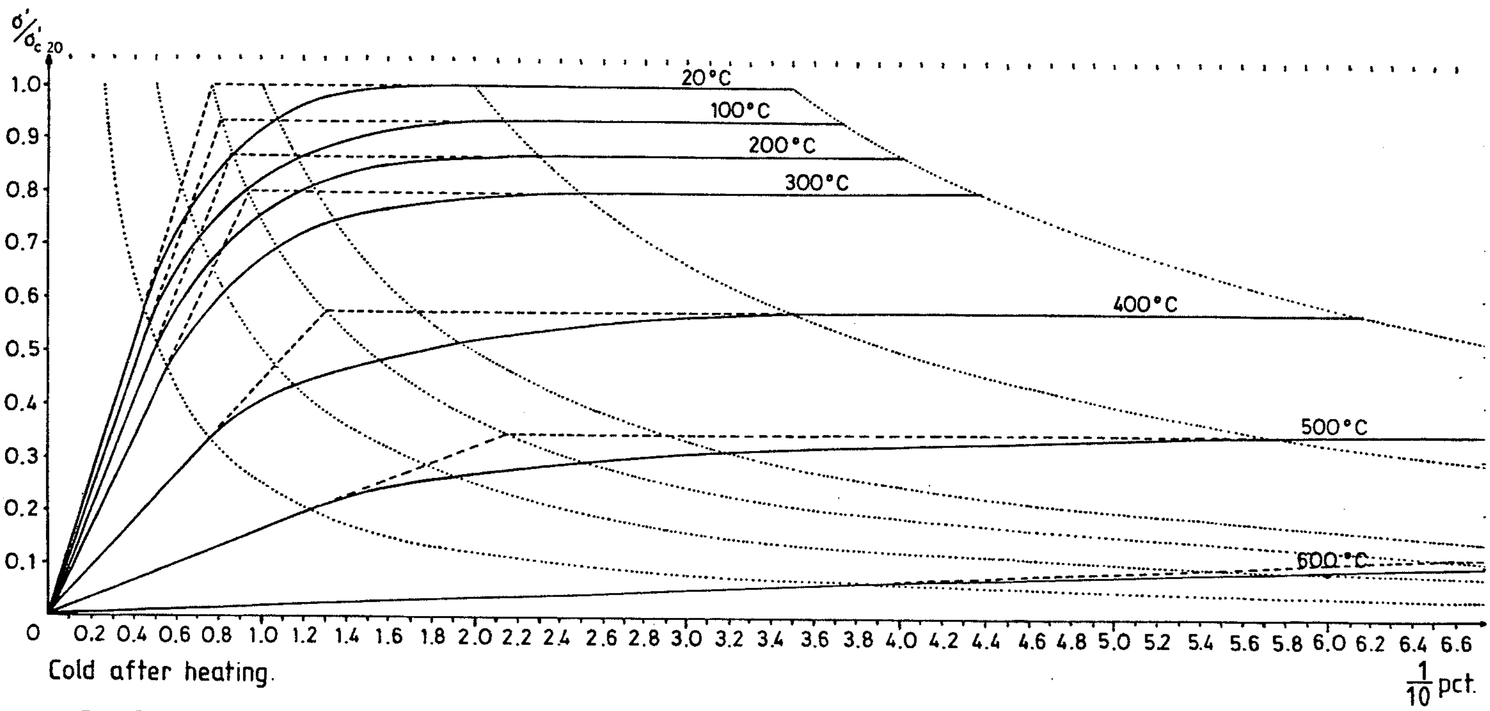
$$\sigma_s = \xi_s\sigma_{s20}$$

$$E_s = \xi_s E_{s20}$$

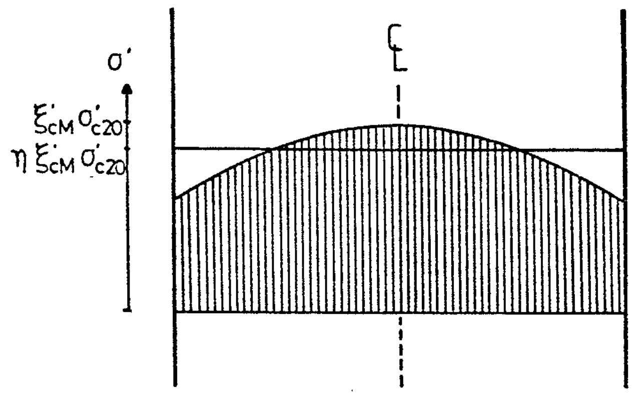


Residual Poissonš ratio of concrete with Danish sea gravel, w/c = 0.87, ν_{c20}= o.16.





Idealized stress-strain curves for concrete exposed to high temperatures.



Definition of the stress distribution factor.

STRESS DISTRIBUTION

One of the main hindrances for developing calcula-ational procedures for determination of the load-carrying capacities and other mechanical properties of concrete structures has been the fact that the maximum temperature and the material properties vary considerably throughout a fire exposed cross-section.

The problems can be handled by application of finite element analyses using an appropriate computer and time for generating the in-data.

However, many of these calculations can be executed much more easily by introducing a new basic concept: the stress distribution factor.

Consider a cross-section exposed to fire at two parallel surfaces.

The isotherms will all be parallel to the surfaces at any time, and the reduction of the compressive strength of the concrete ξ_c' is then a function of the depth from the surface.

The maximum temperature having occurred in the middle of the cross-section until the actual time is denoted T_M, and the corresponding weakening of the concrete is ξ_{cM}'.

If the thickness of the cross-section is C, the stress distribution factor η is determined by

$$\eta = \frac{1}{\xi_{cM}' C} \int_o^C \xi_c' \; (T(z))dz$$

The compressive strength in the middle of the cross-section is

$$\sigma_{cM}' = \xi_{cM}' \sigma_{c20}'$$

The mean value of the compressive strength through the cross-section is then

$$\sigma_c' = \eta \xi_{cM}' \sigma_{c20}'$$

From the stress-strain curves it is seen that the ultimate stress is reached at a strain of 0.35 pct. for most of the temperature levels.

It is therefore a reasonable approximation to assume the cross-section being able to act by its ultimate stresses at every point.

If $\xi_{cM}' < 1$ the ultimate strain in the middle will be

$$\varepsilon_{cM}' = \frac{\varepsilon_{c20}}{\xi_{cM}'} > \varepsilon_{c20}$$

and the approximation then is even more valid.

For applications, where the strain may vary along the centre-line, but has a constant value across the section, the material is considered to be uniform through the section.

The stress-strain curve of the material is assessed to be the one of the weakened concrete having a compressive strength equal to the mean value through the cross-section, but with an ultimate strain not exceeding ε'_{cM}.

The ultimate resistance per unit length of the cross-section is thus

$$C\eta\xi'_{cM}\sigma'_{c20}$$

and the stiffness per unit length is

$$C(\eta\xi'_{cM})^2E_{c20}$$

Concerning the stiffness this represents a minor approximation that can be expressed as

$$(\eta\xi'_{cM})^2 = \frac{1}{C}\int_o^C(\xi'_c)^2(T(z))dz$$

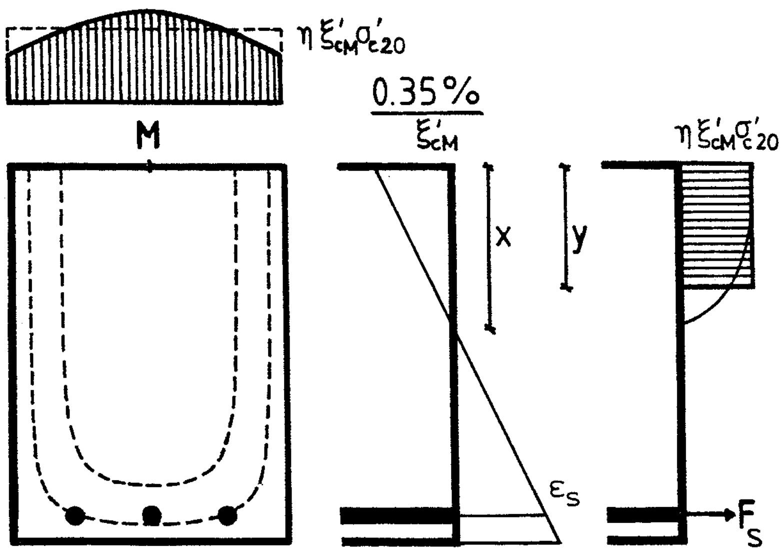
giving rise to an uncertainty of about 10 pct. for the usual values of η of about 0.8.

The approximation is always conservative, i.e. it leads to smaller values of the modulus of elasticity.

The difference increases the more the value of ξ'_c varies through the cross-section, which is often indicated by a smaller value of η.

η seldomly is less than 0.6 and the difference mostly is smaller than 20 pct.

BEAMS SUBJECTED TO BENDING



Beam subjected to bending.

Rectangular beams subjected to bending often have compression zones partly protected against heat exposure from the fire.

Such beams can for example be found directly supporting the ceiling of the room containing the burning objects.

The isotherms of the compression zone are almost parallel to the sides of the cross-section, and the maximum compressive force per unit length of the compression zone at the ultimate limit state is

$$C\eta\xi'_{cM}\sigma'_{c20}$$

Changes in strain are assumed to be proportional to the distance from the neutral axis, and the maximum compressive strain is

$$\varepsilon'_{cM} = \frac{\varepsilon_{c20}}{\xi'_{cM}} = \frac{0.35\%}{\xi'_{cM}}$$

In the cold condition alternative equivalent distributions of the compressive stresses such as a parabola or a rectangle are applied.

These idealizations are even more valid after a heat exposure, since the heat increases the plasticity of the concrete, and the shapes of the stress-strain curves after the heat exposure fit just as well the idealisations as before.

In this presentation a stress block is used with height $\eta\xi'_{cM}\sigma'_{c20}$ and depth

$$y = \frac{3}{4}x$$

where x is the depth of the neutral axis.

The force of the reinforcement is found by summation of the forces in each reinforcing bar after the calculation of the temperature and the assessment of the weakening of each bar.

$$F_s = \Sigma_i A_{si}\xi_{si}\sigma_{s20i}$$

where A_{si} is the area of steel bar number i, ξ_{si} is the reduction of the yield stress for this bar and σ_{s20i} is the yield stress for the unheated steel.

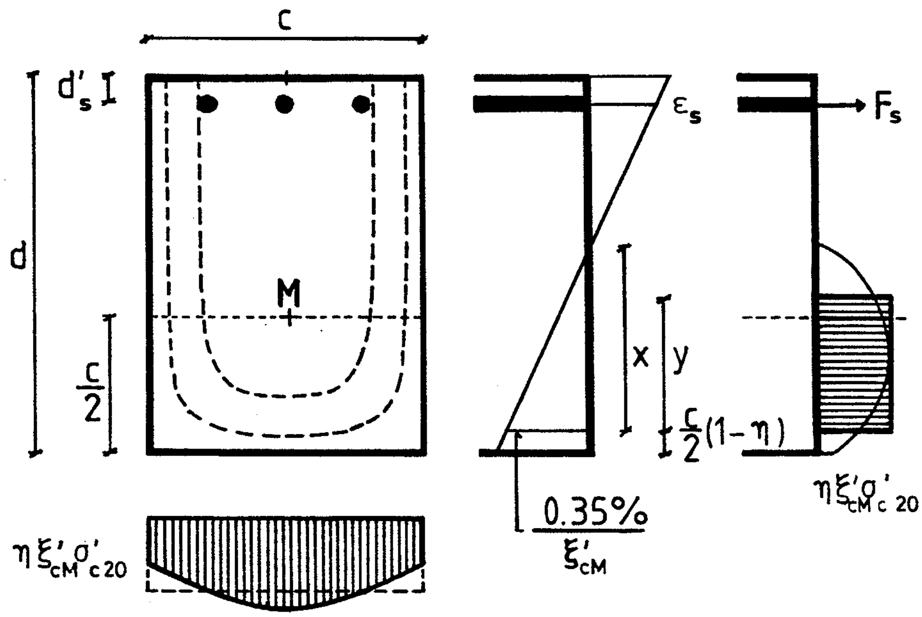
Then we have

$$y = \frac{F_s}{\eta\xi'_{cM}\sigma'_{c20}C}$$

and if d_s is the depth, where F_s acts, the moment capacity is

$$M = F_s(d_s - \frac{1}{2}y)$$

Reversed moment



Beam subjected to bending with reversed moment.

If the compression zone is at the bottom of the beam, i.e. the part of the cross-section, which is exposed to fire on three sides, the isotherms are curved and the problem in estimating the ultimate moment capacity is more complex.

Since the concrete in the compression zone is strongly affected by the heat and the reinforcing bars are protected by the concrete, this is a case, where the weakening of the concrete is of special importance for the bearing capacity of the cross-section.

A reasonable assumption for practical calculations will be that the variation of the weakening of the concrete through the depth from the bottom of the cross-section, caused by the heat conducted from the bottom, is equal to the variation of the weakening through the depth from the sides caused by the heat conducted from the sides.

This means, that the zone of parallel isotherms is considered to stop at the distance $\frac{C}{2}$ from the bottom, where C is the thickness of the cross-section, and if the compressive strength at a depth z from one side is $\xi_c'(z)\sigma_{c20}'$ at the level $\frac{C}{2}$ from the bottom, this value decreases until the bottom level in the same way as ξ_c' decreases from ξ_{cM} at the middle of a two sided exposed cross-section to one of the sides.

The mean compressive strength at the depth z from a side and beneath the level $\frac{C}{2}$ from the bottom is thus

$$\eta\xi_c'(z)\sigma_{c20}'$$

since the mean compressive strength at the level $\frac{C}{2}$ from the bottom (and all levels above this) is

$$\eta\xi_{cM}'\sigma_{c20}'$$

the ultimate compressive force of the zone beneath this level will become

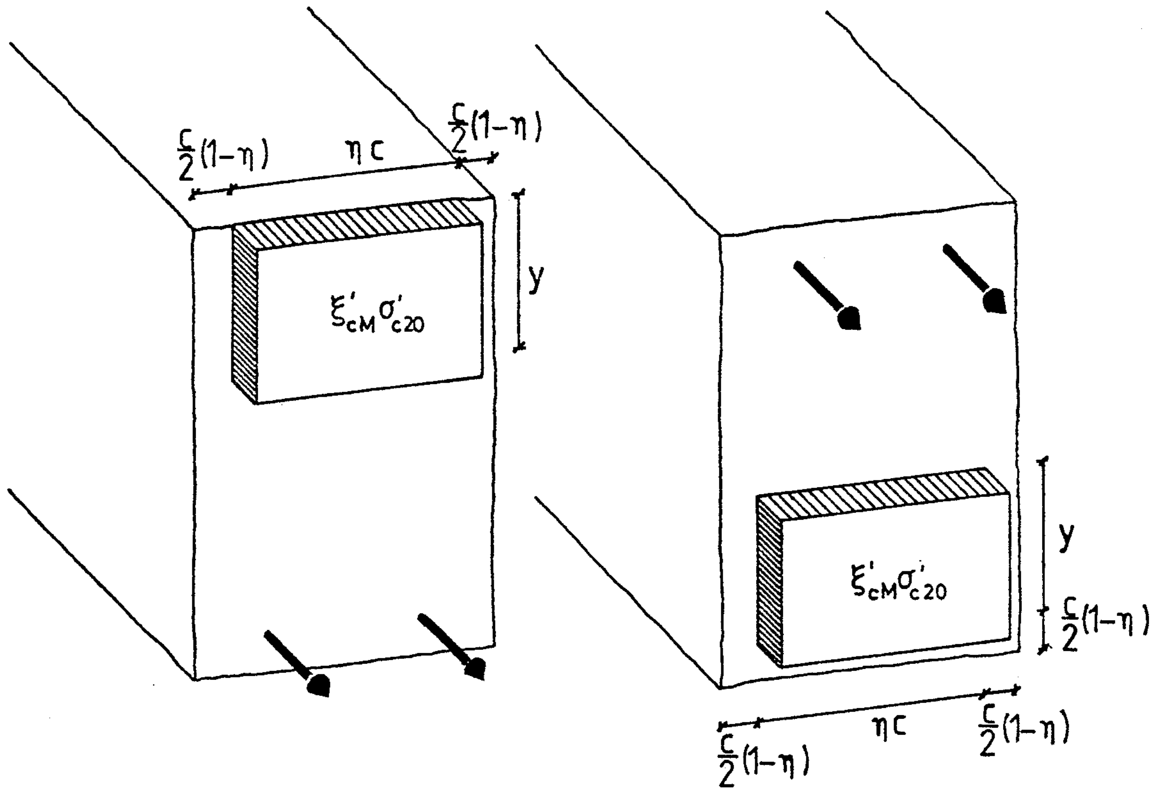
$$\frac{C}{2}C\eta^2\xi_{cM}'\sigma_{c20}'$$

This is achieved, and a proper consideration to the depth of the resultant force is taken, if the ultimate compressive stress $\eta\xi_c'\sigma_{c20}'$ is applied from the level at the distance

$$\frac{C}{2}(1-\eta)$$

from the bottom.

If the ultimate stresses at the two-sided exposed cross-section instead of a mean stress through the depth of the section was considered as a concentrated stress block with the stress $\xi_{cM}'\sigma_{c20}'$, this should



Positions of concentrated stress blocks.

be applied from a depth of $\frac{1}{2}C(1-\eta)$ from each of the two sides.

The concentrated stress block will then be positioned at the same distance from all sides in case of a three sided exposed cross-section.

The total force F_s of the reinforcement is found by summation of the contributions from each reinforcing bar weakened by the heat at the level of the individual bar.

The depth of the block of ultimate stress in the concrete is

$$y = \frac{F_s}{\eta \xi'_{cM} \sigma'_{c20} C}$$

If d and d'_s are the depths of the cross-section and the total reinforcement force respectively, the moment capacity is

$$M' = F_s(d - d_s' - \frac{1}{2}C(1 - \eta) - \frac{1}{2}y)$$

Over -reinforced cross -sections

A precondition for the calculation of F_s as the total yield force or 0.2 pct. proof force is that the strain ε_{smin} is reached in each reinforcing bar, where ε_{smin} is the yield strain or the strain

$$0.2\% + \frac{\sigma_s}{E_s} \approx 0.2\% + \frac{\xi_s \sigma_{s20}}{\xi_s E_{s20}} = 0.2\% + \frac{\sigma_{s20}}{E_{s20}}$$

i.e. ε_{smin} is practically independent of the temperature.

If the strain is less, the cross-section is over-reinforced. The structure may collapse without warning, i.e. at small deflections, and the value of F_s must be redetermined in accordance with the strain distribution of the cross-section.

The risk of over-reinforced cross-sections is usually limited during the fire, while the reinforcing bars are hot and have low ultimate stresses. But when the structure has cooled down, the ultimate stresses of the steel will be regained partly or fully (depending on wether the steel has been cold-worked or not) and the compressive strength of the concrete has further decreased, which may lead to over-reinforced cross -sections. The risk of achieving this after a fire is especially high, if the low ultimate stresses of the reinforcement have been compensated by application of a larger steel area, which can be the result of design methods related to a standard fire without a decay period, such as the one proposed by FIP/CEB [1].

If the compression zone is exposed by fire at the two sides, the strain of the reinforcement is

$$\varepsilon_s = \frac{d_s - \frac{4}{3}y}{\frac{4}{3}y} \cdot \frac{0.35\%}{\xi'_{cM}} \quad (\geq \varepsilon_{smin})$$

If the compression zone is exposed by fire at three sides, the problem is more complicated.

A conservative estimation is achieved if the assumption of a stress block for a material of mean compressive strength in the depth $\frac{1}{2}C(1 - \eta)$ from the bottom is used.

The strain of the reinforcement is then

$$\varepsilon_s = \frac{d - d'_s - \frac{C}{2}(1 - \eta) - \frac{4}{3}y}{\frac{4}{3}y} \cdot \frac{0.35\%}{\xi'_{cM}} (\geq \varepsilon_{smin})$$

If this rough estimation of ε_s leads to a value that is somewhat too small, a more laborious strain analysis can be made, based on the stress-strain curves of the concrete heated to various temperature levels.

However, estimating the uncertainties in assessing the stress-strain curves for the heated concrete, it seems for many applications appropriate to use a simple and slightly conservative criterion to decide whether the cross-section is over-reinforced or not.

Another reason why a slightly conservative treatment of a rectangular compression zone, exposed to fire on the three sides, must be recommendable, is the bevelling of the edges that often is found due to the spalling effect.

Spalling is caused by the flow of steam from the fire exposed cross-section, and the thermal stress distribution at a convex corner increases the risk of this effect.

SHEAR

A classical way of calculating the shear resistance of a reinforced concrete beam is by means of a lattice theory, where the shear is decomposed into tension in the links and compression in the concrete lamellas between the inclined lines of the shear cracks.

This analogy leads to a system of forces in equilibrium, and if the forces in the lamellas and the links respectively are less than the ultimate forces, the shear can be carried by the beam.

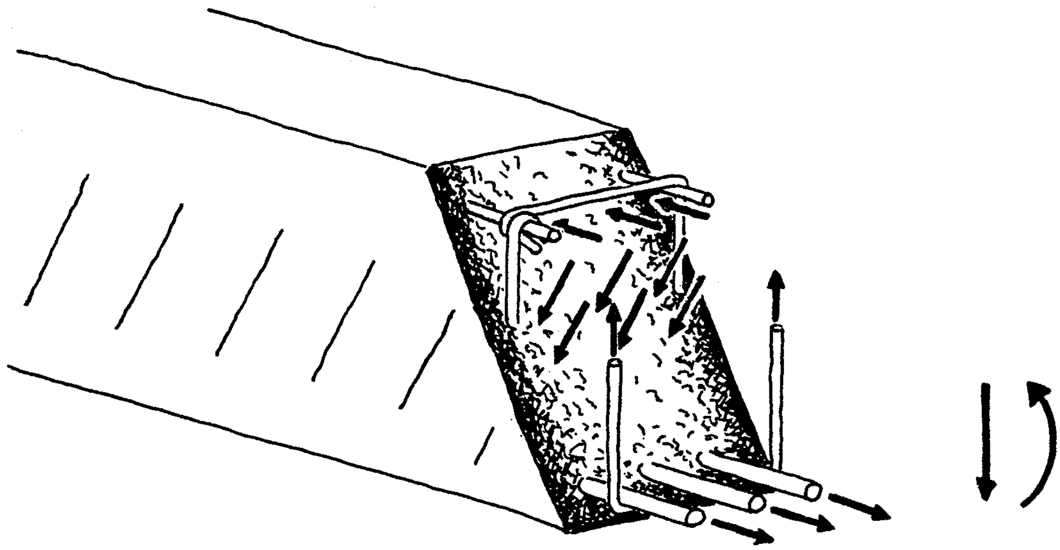
The simplicity of the model is highly advantageous when used for a fire exposed beam, especially if the ultimate shear stresses v_s due to failure of the links and v_c due to compressive fracture of the concrete are known from the common analysis of the loadcarrying capacity at normal temperatures.

Since the links are often placed in the same depth from the side of the beam, the temperature shall be calculated in this depth only, and the reduction of v_s is then equal to the reduction of the tensile strength of the steel ξ_s at this temperature.

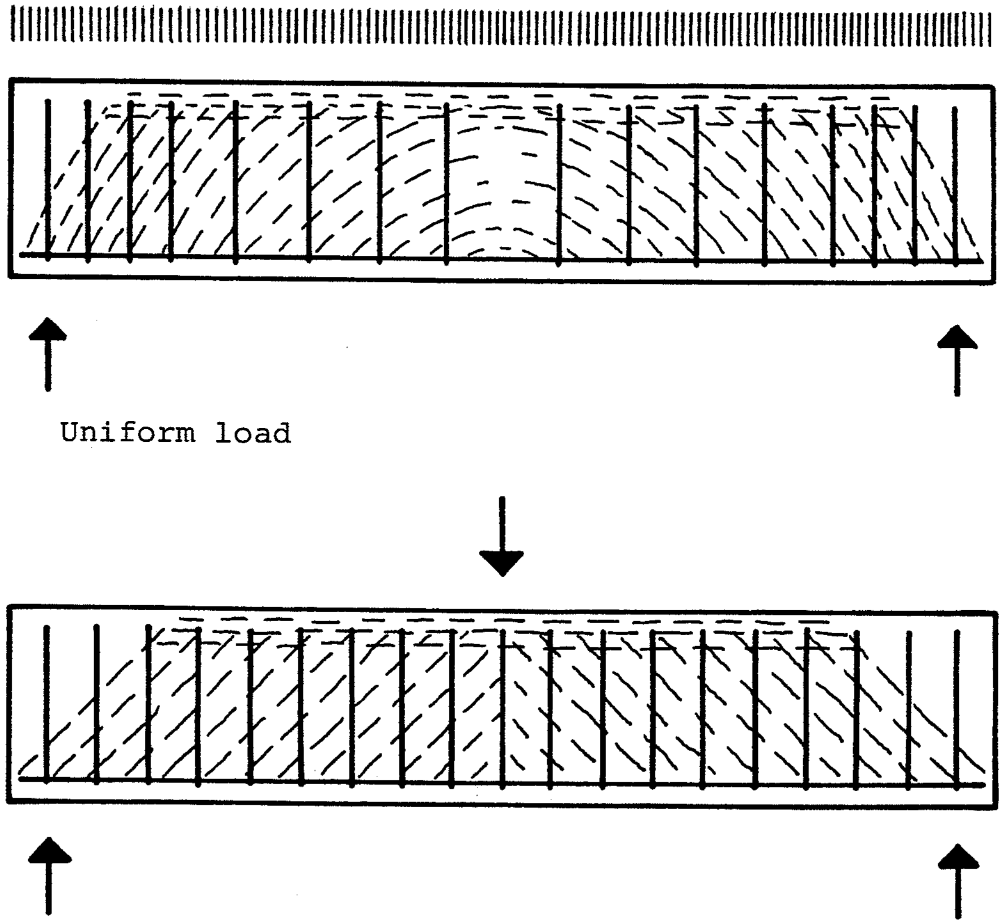
The distribution of the compressive stresses in the concrete lamellas through the cross-section is assumed uniform in the normal temperature analysis, and the mean reduction caused by the fire is then given by $\eta\xi_c$ already known from the investigation of the moment resistance of the beam.

The shear capacity is then given by the mean stresses $\eta\xi_c'v_c$ and $\xi_s v_s$, if the reduced bond between the links and the concrete is found not to cause a greater reduction of the total shear capacity.

In addition the effective depth in shear is reduced in accordance to the increase of the depth of the compression zone.



Stresses in a beam subjected to bending and shear.



Uniform load

Central load

Stress distributions in beams.

SLABS

For a slab the force of the reinforcement is found by a summation similar to the one used for beams.

In estimating the force of the compression zone, the problem is different because the isotherms are now parallel to the neutral axis. This means that the strain and the temperature varies simultaneously, and the stresses for different strain levels must be found by means of different stress-strain curves.

However, drawing the stress distributions for various slabs provided with different thermal insulation, it is seen that the rectangular stress block of a depth equal to 3/4 of the depth x to the neutral axis still represents a fairly good approximation.

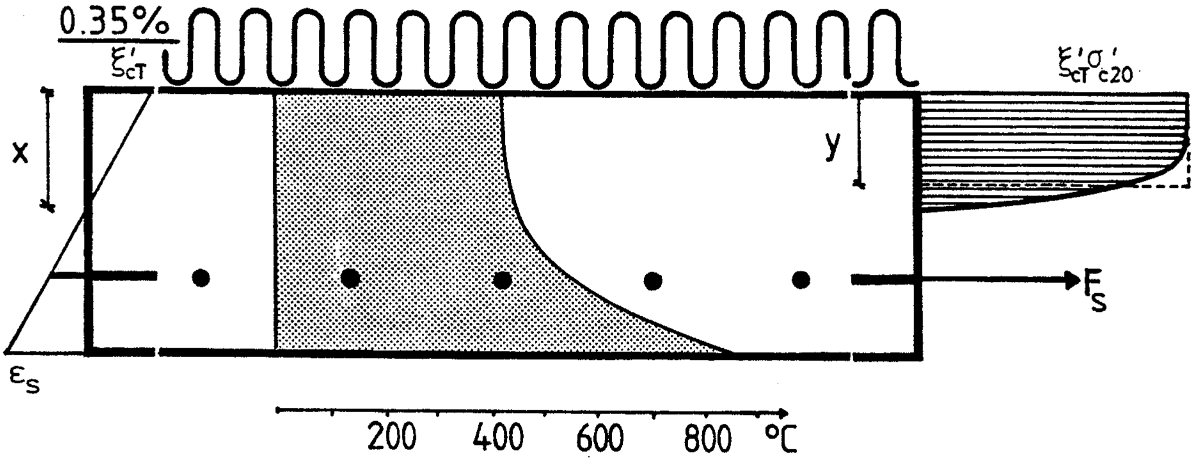
In cases where the compressive strength is zero at the level of the neutral axis it is proposed to use the depth, where the strength becomes positive, instead of x, and closely examine the coherence of the slab.

This case has a special interest where the slab acts as a compression zone of a beam, i.e. the slab is the flange of a T-shaped cross-section.

In those cases, where the reduction of the compressive strength ξ'_{cB} at the bottom of the slab-part of the T-cross-section is not zero, it is proposed to use a maximum depth of the compression-zone of

$$y = \left(\frac{3}{4} + \frac{\xi'_{cB}}{4\xi'_{cT}}\right) d$$

where ξ'_{cT} is the compressive strength-reduction at the top, and d is the total depth of the slab.



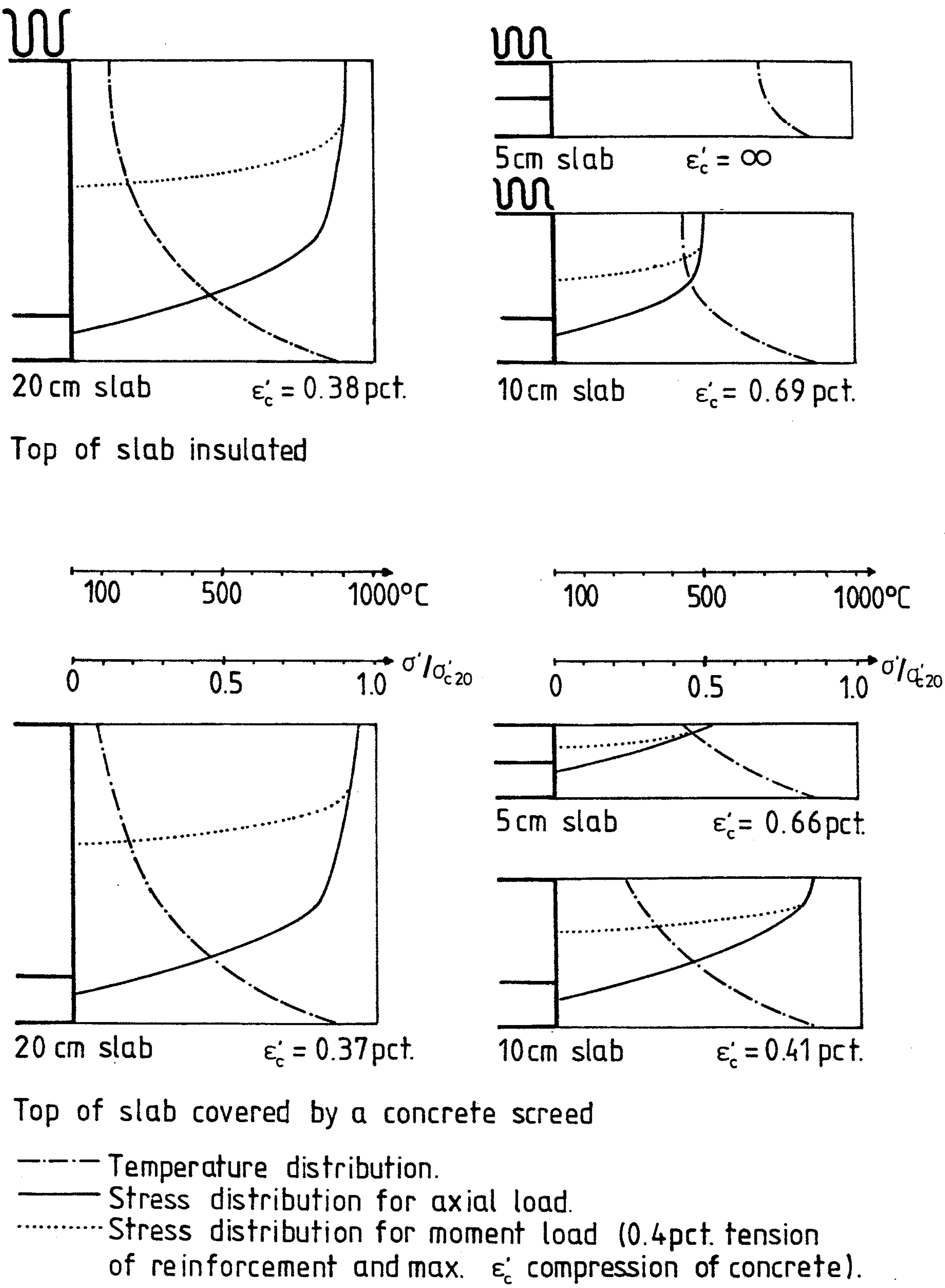
Slab with top-side insulation and subjected to bending.

An uncertainty of up to 20 pct. can arise on the estimation of the compressive resistance of a flange slab, if the expressions are used for a time in the early stages of a fire, where the thermal gradients at the bottom of the slab are large.

The estimation is then conservative, and the uncertainty due to this phenomenon diminishes at later stages of the fire.

In this connection it must be emphasized that the time, where the reinforcement has the highest temperature, is always later than the time of maximum gas-temperature t_{max}, and the time of minimum compressive strength of the concrete is several days after the structure has been cooled down.

This means, that the critical time for a structure will always be later than t_{max}.



Temperature- and stress distributions in various slabs.

COLUMNS AND WALLS

For practical calculations of rectangular concrete columns an assumption can be made similar to the one introduced for beams with a rectangular compression zone exposed on the three sides: that the variation of the weakening through the depth from one side caused by the heat conducted from this side will be equal for all fire exposed sides of the cross-section.

The variation of the compressive strength over the cross-section is equalized by a stress block of height $\xi'_{cM}\sigma'_{c20}$ within a zone of the width

$$\frac{c}{2}(1 - \eta)$$

from all the fire exposed sides of the cross-section.

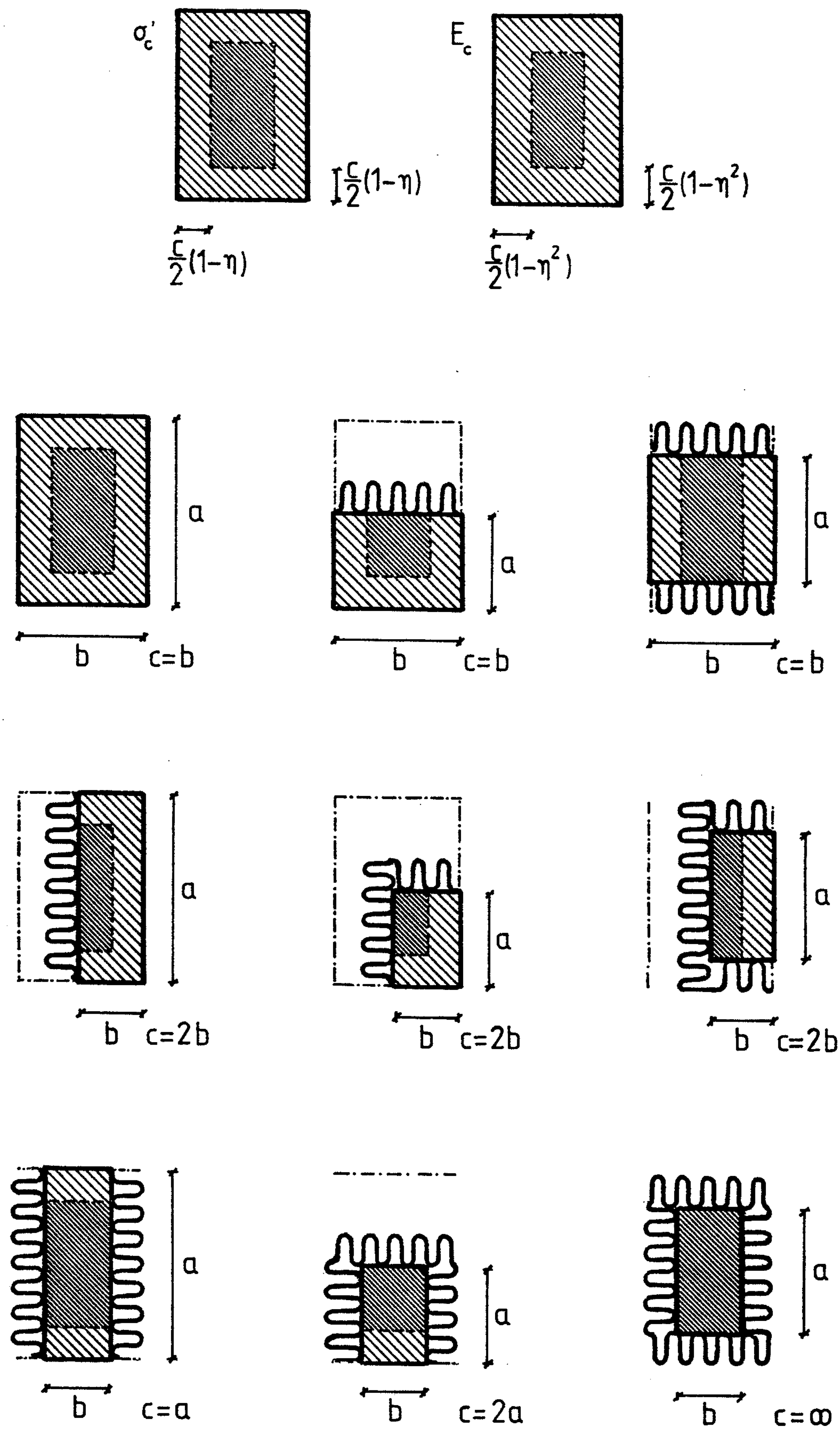
The length c is for this purpose the smallest of the two thicknesses of the column modified for the ability of heat conduction, i.e. a thickness is considered to be the double of the geometrical value if one side is totally insulated, it is considered to be infinite if both sides are insulated totally, and if both sides are exposed to fire the modified thickness is assessed to be equal to the geometrical thickness.

The modified thickness c is also used for the determination of η and ξ'_{Mc}.

The variation of the elasticity over the cross-section is considered by means of a stress block caused by application of an unit strain.

For a wall, exposed to fire on two sides, the mean value of the modulus of elasticity was $\eta^2(\xi'_{cM})^2E_{c20}$.

A corresponding stress block for unity strain would



Modified thicknesses of columns.

be of the value $\xi_{cM}^2 E_{c20}$ over a depth of $\eta^2 c$.

For columns in general the stress block of the value $(\xi'_{cM})^2 E_{c20}$ is surrounded by a zone of the width

$$\frac{c}{2}(1 - \eta^2)$$

to all fire exposed sides of the cross-section, and c is determined as for the distribution of the compressive strength.

The product of the moment of inertia and the modulus of elasticity $E_c I_c$ is calculated for the area of the cross-section occupied by the stress block.

If a column of width d and thickness c (c < d) is exposed to fire on four sides, the product becomes

$$E_c I_c = \frac{1}{12}(d-c(1-\eta^2))(\eta^2 c)^3 (\xi'_{cM})^2 E_{c20}$$

using the Rankine formula for a plain centrally loaded concrete column, the critical compression force F'_{crc} is expressed by

$$\frac{1}{F'_{crc}} = \frac{1}{A_c \xi'_{cM} \sigma'_{c20}} + \frac{\ell^2}{\pi^2 E_c I_c}$$

where ℓ is the effective column length and A_c the effective concrete area for the ultimate compressive stresses.

For a column exposed to fire on four sides A_c is

$$A_c = c\eta(d - c(1 - \eta))$$

Reinforced columns

If the reinforcement of the column is taken into consideration, the ultimate compressive yield- or 0.2 pct.-force F'_s of the reinforcement is summarized

$$F'_s = \Sigma_i A_{si} \xi_{si} \sigma'_{s20i}$$

and the product of the ratio of reinforcement φ and the ratio of moduli of elasticity n is found.

For a four-sided fire exposed column this becomes

$$n\varphi = \frac{E_s}{E_c} \cdot \frac{A_s}{A_c} = \frac{\Sigma_i A_{si} E_{si}}{(d-c(1-\eta^2))\eta^2 c(\xi'_{cM})^2 E_{c20}}$$

In these summations only reinforcement sufficiently embedded in the concrete cross-section should be taken into account. This means that the concrete at the level of the bar and hence the bar itself should not be heated to a maximum temperature of more than the one, at which the concrete cracks, i.e. a limit of about 350°C would be reasonable.

If the stirrups transverse to the longitudinal compressed bars is not positioned immediately at the surfaces og these, but nearer the fire exposed sides of the column, a more detailed analysis is required.

Since many traditional designed concrete columns have the longitudinal reinforcement placed near the surfaces of the cross-section in order to improve the moment of inertia as much as possible, the value of F'_s often becomes very small or nil.

Thus, it is often reasonable only to consider the plain concrete cross-section calculating the critical load of a fire exposed column.

Generally the critical vertical load F'_{cr} for a centrally loaded reinforced concrete column is found by

$$F'_{cr} < \begin{cases} F'_{crc}(1 + n\varphi) \\ F'_{crc} + F'_s \\ CF'_{crc} \end{cases}$$

where C = 2.0 when laps are not used or else C = 1.5.

DESIGN OF A STRUCTURE

On the previous pages design formulas are developed for most of the ordinary structural elements giving their load-carrying capacities at a given time of a fire course.

On purpose nothing has been mentioned so far about what time of the fire course the investigations should refer to. Neither has anything been mentioned about the temperature-time development.

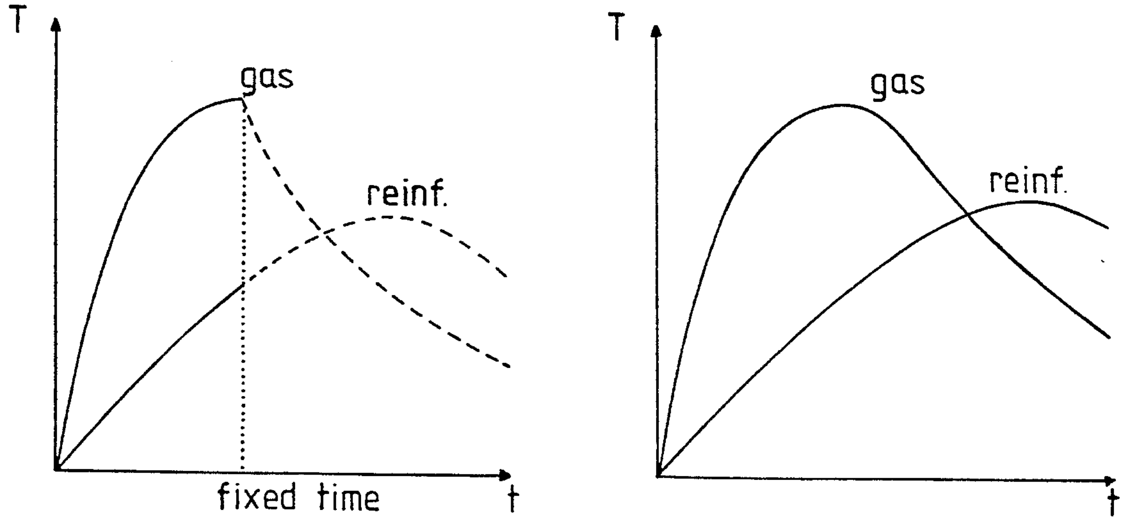
This is because the formulas give the ultimate limit capacities at any time of any fire applied to.

Before the fire starts, they correspond to the usual ultimate limit design formulas. During the fire cause, they show the reduction of the ultimate limit capacities due to the reduced strength properties of the reinforcement and the concrete. And after the fire, they give the residual ultimate capacities, and hence form an effective tool for restoring damaged structures, or for deciding whether the structures should be demolished or not.

Designing a new structure, the requirements regarding the fire resistance must be clarified.

If it is only required that the structure must be able to carry the load at a given time of a fire course of increasing temperature, the investigation is reduced to comprise this time only. Such an investigation does not show, whether the structure is able to carry the load through a complete fire-course.

The reason is, that the time, where the greatest reduction of the load-carrying capacity occurs, always is after the time of the maximum gas temperature,



Temperature-time developments for the gas and a reinforcing bar embedded in concrete.

because it takes time for the heat to be conducted into the cross-section.

If it is required that the structure should be able to carry a certain load through a complete fire course, the investigation must deal with the conditions, where the structure is the weakest.

This means, that the load-carrying capacity should be calculated at the time, when the reinforcement is the weakest, and at the time, where the concrete is the weakest.

The first time is always later than the time of the maximum gas temperature, and mostly this delay is from about 15 minutes to several hours.

The time of the weakest reinforcement is found by an optimization, where the temperatures of the single reinforcing bars are calculated and their strength reductions weighted by their cross-section areas.

The temperature calculations can for example be made by means of the simple methods presented in Hertz [8], and the strength reductions ξ_s of the steel must correspond to the actual steel applied.

In this hot condition the mean strength reduction of the concrete η and ξ'_{cM} should be calculated for the same time as the reduction of the force of the reinforcement.

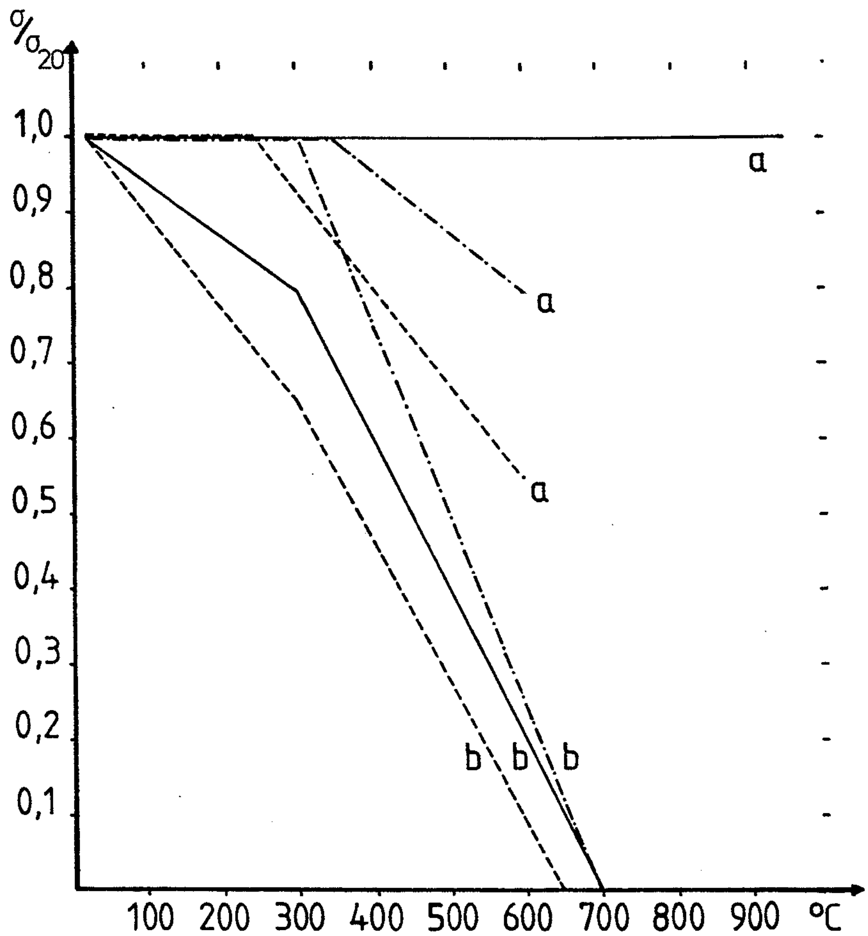
Since the calculation in a hot condition is much less sensitive to variations in time for the concrete than for the reinforcement, tables have been made for the stress distribution factors at the fixed time, where the temperature is maximum in a depth of 30 mm from the surface of a semi infinite specimen (Hertz [9]).

Calculating the stress distribution factor for a hot condition it must be remembered that until a certain depth form the surface, the concrete has gained its maximum temperature before the time considered, and here these temperatures must be used, and in the rest of the cross-section the temperature distribution at the time considered has to be used.

The second part of the investigation deals with the time, where the concrete is the weakest, which is usually about 7 days after the fire.

This calculation therefore gives directly an estimation of the residual ultimate load-carrying capacity of the structure.

The force-contributions from the single reinforcing



St 37 and hot rolled. — a cold condition

Danish tentor. — b hot condition

Cold drawn wire.

Reduction of yield- or 0.2 pct. proof stress for various steels in a hot condition and after exposure to high temperatures.

bars must be summarized with regard to their residual strengths based on a calculation of the maximum temperature of each bar.

The values of η and ξ'_{cM} for the concrete should be calculated from the distribution of maximum temperatures through the cross-section.

Such values have also been calculated for a large variety of cross-sections and fires in Hertz [9].

These values are calculated for an unloaded concrete with Danish sea gravel aggregates, and will represent conservative estimations for most other concretes and loads.

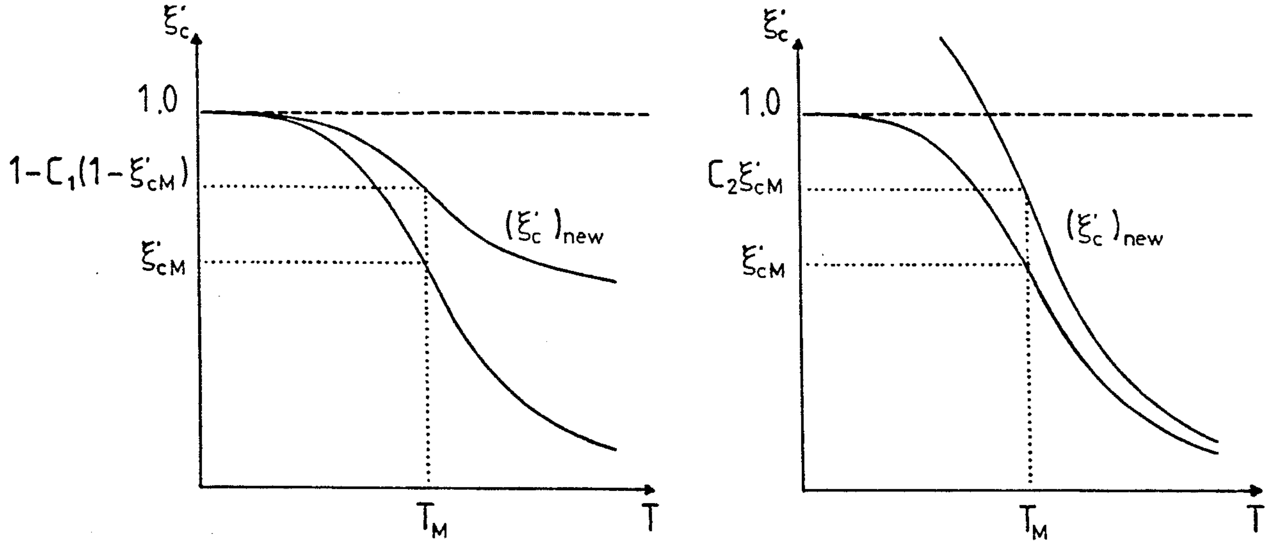
If the reduction of the compressive strength ξ'_c varies in temperature in a way, different from the one for which η and ξ'_{cM} is calculated, a new calculation can always be made, but often new values can be estimated in an easy way by means of affinity considerations using the shapes of the curves of ξ'_c as functions of the temperature.

Many of these curves will be almost identical by multimplication of the differens $1 - \xi'_c$ by a constant C_1, which can be done for $1 - \xi'_{cM}$ and $1 - \eta$ as well.

$$(\xi'_{cM})_{new} = 1 - C_1(1 - \xi'_{cM}) \text{ and}$$

$$(\eta)_{new} = 1 - C_1(1 - \eta)$$

In some cases the reduction can differ for the small temperature levels, and remembering that ξ'_{cM} is the largest value of ξ'_c, and that only the parts of the curves are used representing temperatures larger than the temperature T_M in the middle of the cross -section, these parts may be identical by multipli-



Transformations of ξ'_c.

cation of ξ'_{cM} and η by a constant C_2.

$$(\xi'_{cM})_{new} = C_2\xi'_{cM} \text{ and}$$

$$(\eta)_{new} = C_2\eta$$

where of course the value $C_2\xi'_{cM}$ must not exceed unity.

Finally the two transformations can be combined, so that

$$(\xi'_{cM})_{new} = C_2(1 - C_1(1 - \xi'_{cM})) \text{ and}$$

$$(\eta)_{new} = C_2(1 - C_1(1 - \eta))$$

or alternatively

$$(\xi'_{cM})_{new} = 1 - C_1(1 - C_2\xi'_{cM}) \text{ and}$$

$$(\eta)_{new} = 1 - C_1(1 - C_2\eta)$$

POSTSCRIPT

Although the discussion of the properties of concrete at high temperatures at the beginning of this presentation was very brief, it will be seen that the material is not at all fire resistant, and that the weakening of the concrete has to be taken into account, when calculating the load-carrying capacity of a structure.

In the following chapters formulas have been developed for a such calculation by means of the new concept: the stress distribution factor. Values of this factor are tabulated in the report "Stress Distribution Factors" Hertz [9], CIB W14/81/14 (DK). But the factor can also be calculated knowing the compressive strength of the concrete as a function of the temperature, and calculating the temperature distribution with a sufficient accuracy by means of for example the procedure presented in "Simple Temperature Calculations of Fire Exposed Concrete Constructions" Hertz [8], CIB W14/81/13 (DK), which also can be used for the calculation of the temteratures of the reinforcing bars.

The three reports thus form a basis for the design of any rectangular concrete section exposed to any fire, and especially the residual load-carrying capacity after the fire can be found.

If the high-temperature properties of the concrete are not exactly known, the procedures are supposed to give conservative estimations of the load-carrying capacities of structural elements. On the other hand, if the properties are known, the stress distribution factors may be adjusted accordingly.

However, it must be emphasized that no calculation will be more precise than justified by the knowledge it is based on. A subject for further investigations is therefore the development of testing procedures supplying the designer with precise values of the physical properties necessary for the calculation. These are mainly: the conductivety of heat and the ultimate compressive stress as a function of temperature.

Another subject for investigation is the full-scale testing of structural elements. This will of course always be an alternative to the calculational procedure. It is relevant especially in cases, where the improved accuracy that a full-scale test can give, is more important than the time and cost of a such procedure. But even in these cases, a fast calculational approach is valuable for the decisions concerning how the tests should be carried out most efficiently.

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