The behaviour of concrete products and precast reinforced concrete members in a fire

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PREFACE

This paper describes the behaviour of materials exposed to fire. The performance of precast concrete and precast reinforced concrete when exposed to standard fire test is analysed and the influence of the properties of the aggregates, the cement paste, and the reinforcing steel on fire endurance of the completed assembly is discussed. The author Dr. Ing. H. J. Wierig was formerly engaged in Fire Testing with the University of Braunschweig and is now with the Cement and Concrete Research Institute in Beckum, West Germany.

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(Das Verhalten von Betonwaren und Stahlbetonfertigteilen im Feuer)

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THE BEHAVIOUR OF CONCRETE PRODUCTS AND PRECAST REINFORCED CONCRETE MEMBERS IN A FIRE

1. Introduction

Fire is one of the natural forces to which man has always been exposed. From antiquity down to the present, history tells of many catastrophic fires which destroyed whole villages and cities. At an early date attempts were being made to profit by the experience gained from fire and to adapt the construction of buildings and to plan villages and cities accordingly. As an example we may cite the well-known fire of London in 1666. At the time of reconstruction of the city, very extensive building regulations were enacted[1]. For example, the kinds of building materials to be used were specified, and the width of the new streets was determined. These regulations can be regarded as the forerunners of our present-day regulations.

However, systematic research into the behaviour of building materials and structures exposed to fire was only begun about 100 years ago. At first the investigations were restricted to the burning down of old buildings condemned to demolition[1,2]. Later, systematic fire tests under well-defined conditions were introduced. Today, in most countries, there are well-defined standards and test procedures, and an international standard is under preparation. In Germany fire tests are carried out under DIN 4102 "Resistance of building materials and members to fire and heat". This standard is at present under revision. A draft of the testing regulations contained in the revised version of this standard has just been published.

In the last 100 years a great deal of knowledge has been gained about the behaviour of building materials and members in fires. Nevertheless, the damage caused by fire has continued to increase even in recent years. In Figure 1 the finely hatched columns show the increase in damage due to fire in the Federal Republic of Germany during the years 1954 to 1961[3]. The growth of the gross national products reduced 1000 times is shown for the same period by the coarsely hatched columns[4]. It will be recognized that the damage has been increasing at a somewhat faster rate than the gross national product. Similar trends have been noted in other countries. Figure 1 illustrates very clearly that all economically feasible measures must be taken in order to prevent any further increase of damage due to fire. In this connection preventive structural fire protection acquires a special importance. Structural measures include the following:

the prevention, as far as possible, of the outbreak of fire; measures hindering the spread of fire; and provision of escape routes for the use of the inhabitants and occupants of a building in the event of a catastrophic fire.

For the prevention of the spread of fire damage the load-bearing and separating members of a building must resist the fire for a sufficiently long time, i.e. they must continue to exercise their intended functions. The required fire resistance time depends on the nature and function of the building involved. For example, different regulations are applied to warehouses and theatres in the building codes than to small, single-family dwellings. Designing architects and engineers must have sufficient knowledge of the construction possibilities by which an adequate fire resistance time can be attained in a given member.

In considering the behaviour of structural members in fire, two basic cases must be distinguished. During a fire a construction is only briefly, but very severely exposed. (For this unrepeated catastrophic case, only a simple safety measure is required.) The state of the structure after the fire, i.e. with a view to continued use, is only of secondary consideration. For building compononents which have to withstand comparatively high temperatures while performing their design functions, quite different conditions prevail. This is the case, for example, in furnace construction, and more recently in reactor design. Three different points of view must be taken into account both in the selection of the building materials and in the structural design.

In many respects there is no fundamental difference between the behaviour in fire of precast and poured-in-place concrete construction. The present paper reviews the behaviour of concrete and reinforced concrete in fire and will discuss from individual examples the special properties of concrete products, precast reinforced concrete members, and the possibility of improving the fire resistance time of individual members.

2. The Behaviour of Building Materials Exposed to Elevated Temperatures

2.1 General

With respect to the behaviour of building materials exposed to elevated temperatures, we can separate two broad classes, combustible and non-combustible materials. Wood and many plastics, for example, are among the "combustible" building materials. At higher temperatures, they begin to burn and are destroyed. Not so well-known is the fact that the "non-combustible" materials, including concrete and steel, suffer an alteration of important mechanical properties as an effect of exposure to elevated temper-

atures. Regarding the behaviour of structures in fire, the following properties of the building materials and their changes due to higher temperatures are of importance:

1. Coefficient of thermal expansion;
2. strength, yield point, elongation at fracture and creep behaviour;
3. modulus of elasticity;
4. thermal conductivity and specific heat.

Moreover, changes in composition and structure of some building materials can affect the behaviour of the construction in a fire. For example, many rocks crumble on exposure to extreme heat.

We shall now review the most important changes that occur in concrete and building materials which are used in conjunction with concrete, due to the effect of higher temperatures.

2.2 Mortar and concrete

2.21 Thermal expansion

Mortar and concrete are made up of cement, aggregates and water. The high temperature behaviour can differ considerably, depending on the mix, the materials employed and especially the aggregates. While at room temperature the mechanical properties of the solid concrete are determined primarily by the properties of the cement paste*, the high temperature behaviour, especially the coefficient of thermal expansion, is greatly influenced by the aggregates as well, since they make up the greater part of the volume of the concrete. Therefore, let us first consider the behaviour of the aggregates.

The effect of the aggregates on the behaviour of mortar and concrete at higher temperatures depends primarily on their coefficient of thermal expansion. The texture of a specimen of mortar and concrete can remain intact in the presence of increasing temperatures only if the resulting changes of shape of the separate components are mutually compatible, and the internal stresses can be absorbed by the cement paste. Hence, aggregates with a low coefficient of thermal expansion and a temperature-expansion curve that is as smooth as possible, behave best. Figure 2 shows the thermal expansion of

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* hydrated Portland cement paste.

various aggregates as a function of the temperature[5,6]. It will be noticed that for quartzitic rocks there is a characteristic discontinuity of the thermal expansion between 500 and 600°C. This bend in the temperature-expansion curve is due to the $\alpha \rightleftarrows \beta$ quartz conversion, which under laboratory conditions takes place at 573°C with a heat consumption of approximately 8 cal/g. To be sure, the practical effects of quartz conversion is not as serious a matter in a fire as it may appear to be at first glance. Contrary to experiments with small specimens, structures in the fire are only slowly heated from the outside in to a temperature above 500°C. Structural relaxation, therefore, proceeds only to a limited depth. Of course, if concrete constructions that have been exposed to fire are to be used again, the affected layers must be removed.

With the exception of diabase, non-quartzitic rocks generally behave better than quartzitic material. Among natural rocks, basalt and a number of kinds of limestone are found to have favourable coefficients of thermal expansion, while artificial stones similarly endowed include blast furnace slag fragments and crushed brick. Even for basalt, there is a critical temperature, at approximately 900°C. At this temperature the rock begins to expand markedly, giving off gas.

When mortars or concrete specimens are heated it is not only the aggregates that expand. The cement paste also changes in volume. Figure 3 shows the change of volume of cement paste from Portland cement according to Endell[5], Nekrassow[6], and Philleo[7]. The numerical data of these investigators differ. It is clear, however, that first a thermal expansion occurs, which is subsequently compensated for and exceeded by the shrinkage due to release of water. A further reduction of volume takes place on cooling after heating. Probably the different results are due to different compositions of the cement paste (water-cement ratios) and different rates of heating and different sizes of specimens.

Table I contrasts the various test conditions.

If the cement paste is heated not once, but several times, then, according to ref.[6], a reversible positive expansion occurs from the second heating on (Figure 4).

Since the aggregates expand when exposed to heat, whereas the cement paste begins to shrink again at higher temperatures, the thermal expansion occurring in concrete is less than the aggregate itself. Under otherwise similar conditions the coefficient of thermal expansion decreases with increasing cement content[7]. In Figure 5 the thermal expansions of various

mortars are plotted as functions of temperature after Endell[5]. All stones of the same kind do not necessarily behave the same. Considerable variations are possible among quartzitic material, and especially among limestones.

2.22 Strength

When specimens made from mortar and concrete are exposed to higher temperatures their strength values are altered. Publications[6,8-15] on strength tests of mortar and concrete in a heated state are partly contradictory. This is due on the one hand to varying test conditions (specimen) size, mix, type of aggregate, age of the specimens, storage conditions, heating times, time of testing after heating, water-cement ratio, construction of test furnaces, etc), and secondly to differences in the initial material. The investigators fall into two groups. One group already found a decrease in compressive strength for the comparatively slightly heated stage of about 200°C, which continued at a more or less steady rate at higher temperatures. Other tests, however, showed an increase of compressive strength up to about 300°C and a drop only on further heating. The initial increase of strength on heating of certain concretes can be explained by the fact that heating brings about a "steam hardening", leading to additional hydration of the cement. With many types of aggregates, moreover, a reaction between cement and aggregate is by no means impossible.

The vertically hatched area in Figure 6 shows the scatter range within which most of the results of concrete compressive strength tests at higher temperatures fall. The data of Malhotra[11], according to which the water-cement ratio has no appreciable effect on the relative change of thermal compressive strength, are interesting. Of greater influence, however, is the ratio of cement to aggregates, which presumably can be attributed to the above-mentioned severe internal stresses arising between cement pastes and aggregates.

British investigations[16] showed that after exposure to temperatures up to about 300°C for several weeks no further decrease of strength could be expected (Figure 7).

According to French investigations[17], concrete tensile strengths drop more rapidly than the compressive strength values. The scatter range for the tensile strength is horizontally hatched in Figure 6. For a special fireproof concrete, the compressive strength may be greater after very severe heating than those indicated in Figure 6[6,12,15]. The increase of strength in the temperature above 1000°C, represented in Figure 6 by the

broken-line curve, is due to the fact that in properly composed concrete the lost bond of hydration is replaced by a ceramic bond.

2.23 Modulus of elasticity

When concrete is exposed to elevated temperatures, a decrease in the modulus of elasticity occurs. This strongly affects the deformations in a structure. Also affected is the interplay of external and internal forces in statically indeterminate structures, in prestressed concrete constructions and even in standard, reinforced concrete cross-sections.

Some existing results of modulus of elasticity measurements at higher temperatures differ sharply from each other. Figure 8 shows Woolson's[18] results on diabase and limestone concrete, later confirmed by Busch[9] on sandstone concrete (quartzitic). Woolson investigated concretes with a cube strength of about 160 kg/cm^2. Busch's investigation dealt with cube strengths of 280 kg/cm^2. The compressive stresses were 30 to 50 kg/cm^2. The very strong decrease of the modulus of elasticity even at relatively low temperatures up to 400°C is striking. According to Woolson and Busch, the modulus of elasticity at 400°C is only about 20% its value at room temperature.

In more recent investigations[7,12,19] a slower rate of decrease of the modulus of elasticity was found by Cruz and Philleo. The results of these investigations are also plotted in Figure 8. However, in these tests, also, the modulus of elasticity at 300°C, again fell to 40 to 50% of its value at room temperature, depending on the water-cement ratio.

The difference between the results of Woolson and Busch on the one hand and those of Cruz on the other may be sought in the fact that according to Busch's findings the modulus of elasticity of a concrete that has been exposed to fire increases with increasing load. This means that the ratio of E_{cold}/E_{warm} is more favourable when the tangent moduli are compared, not, as usual, the secant moduli. For practical application, therefore, values may be chosen which lie between those of Busch and Cruz, depending on the nature of the stress.

2.24 Heat conduction and specific heat

The internal heating of a structural member depends largely on the heat conductivity and the specific heat of the building materials employed. These are not constant values, but depend on various factors. If we con-

sider the heat conduction of concrete, we must distinguish between three basic temperature ranges:

Range I below 0^0C

Range II between 0^0 and 100^0C

Range III above 100^0C.

In conjunction with the behaviour of concrete structures in the presence of fire, range I is of interest only in a very few exceptional cases. Normally, the initial temperature of a construction at the start of a fire will be above 0^0C. For the sake of completeness, however, it must be mentioned that a considerable delay in the internal heating of the structural members is due to the latent heat of melting of ice.

In range II the heat conduction (kcal/mh^{0}C) of the concrete is influenced primarily by its bulk density and its moisture content. Figure 9 shows these effects at room temperature[20]. The specific heat of regular air-dry heavy concrete at room temperature can be taken as 0.21 - 0.25 kcal/kg^{0}C.

In Figure 10, with the example of an aerated concrete having a bulk density R = 520 kg/m^3, it is shown that the thermal conductivity at constant bulk density depends not only on the moisture content, but also on the temperature[21]. At higher temperatures the heat conduction characteristics increase. This is due to the fact that at higher temperatures the amount of heat transferred by radiation in the pores increases greatly. It may also be inferred from Figure 10 that the maximum heat conductivity value of aerated concrete is attained not for total saturation with water, but with partial filling of the pores.

In temperature range III, concretes are completely dry. The coefficient of thermal conductivity, therefore, is less than in range II. In Figure 11 the thermal conductivities of a few concretes at higher temperatures are represented as a function of the temperature[22,23]. Here again there is a definite rise in the thermal conductivity with temperature.

The practical importance of the increase of thermal conductivity of concrete with the temperature is limited. The heating of a thick plate subject on one side to temperatures which vary in time is governed by Fourier's equation of heat motion. This is written

$$\frac{\delta T}{\delta t} = a^2 \frac{\delta^2 T}{\delta s^2}$$

where

T = temperature (^0C)

t = time (h)

s = depth in the wall (m)

a^2 = coefficient of thermal diffusion $\left(\frac{m^2}{h}\right)$. (thermal diffusivity)

For heat transfer, therefore, the decisive factor is not the thermal conductivity, but the thermal diffusivity a^2, which is defined as follows:

$$a^2 = \frac{\lambda}{c\gamma}$$

where

a^2 = coefficient of heat diffusion $\left(\frac{m^2}{h}\right)$ (thermal diffusivity)

λ = thermal conductivity $\left(\frac{kcal}{m \cdot h \cdot {}^0C}\right)$

c = specific heat $\left(\frac{kcal}{kg \cdot {}^0C}\right)$

γ = bulk density (kg/m^3).

Since the specific heat c of the concrete also increases within increasing temperature[9,12], the thermal diffusivity which depends on the quotient $\frac{\lambda}{c}$, varies but little.

Much more decisive is the delay in internal heating in a fire owing to the evaporation of the water in the concrete. The latent heat of evaporation of water, of course, is over 500 kcal/kg. This is a very large amount of heat. It follows that the behaviour of a concrete construction in a fire, and especially its internal heating, is decisively influenced by the water content.

A parallel phenomenon occurs in limestone concretes. Here quantities of heat are also consumed in the conversion of $CaCO_3$ into CaO and CO_2 at a rate of nearly 400 kcal/kg $CaCO_3$ at a surface exposed to the fire, thus delaying the heating of the structure. The spontaneous liberation of CO_2 by the limestone sets in at approximately 900^{0}C, but at low CO_2 partial pressures it may start as low as 600^{0}C.

Figure 12 shows the loss of weight of a concrete containing some calcareous aggregate according to ref.(7). The vigorous liberation of water between 100 and 300^{0}C is clearly evident. Above 400^{0}C a slight loss of weight then occurs due to liberation of water from the calcium hydroxide of the cement paste. At about 600^{0}C the conversion of the limestone begins.

This may be summed up as follows:

The thermal conductivity of concrete is determined primarily by the bulk density and the moisture content. The greater the bulk density and moisture content, the greater the conductivity. The complete heating of

structural members is affected decisively by the evaporation of the free and bound water.

2.3 Steel

The fire resistance of all reinforced concrete constructions, including steel concrete and prestressed concrete, is decisively affected by the behaviour of the reinforcement. The change in the mechanical properties of steel on exposure to high temperatures depends on the kind of steel and especially on the carbon content. In the following paragraphs a number of typical examples are cited inasmuch as these are required for an understanding of the behaviour of reinforced concrete constructions.

2.31 Thermal expansion

In Figure 13 the linear thermal expansion of a steel with 0.4% carbon is plotted after ref.(24). It will be recognized that the thermal expansion is continuous up to temperatures of about 700°C. The coefficient of thermal expansion increases slightly. Compared with the thermal expansion of the concretes (Figure 5) the expansion of steel is somewhat less than sand-and-gravel concrete, but somewhat greater than limestone concrete. Generally speaking, however, the thermal expansion of the steel and the concrete is the same even over considerable temperature ranges.

2.32 Yield point at elevated temperatures

Figures 14 and 15 show the yield point (0.2 proof stress) for a number of steels as a function of the temperature. The hot tensile strength curves have been left out, because by and large they are closely related to the yield point curves. Only in the temperature range around 200°C is there at first a slight rise in the tensile strength.

Figures 14 and 15 differ in that in the former the tests were carried out in the hot state, while in Figure 15 they took place after cooling. The values of Figure 14 are important for the behaviour of the material during a fire, those of Figure 15 for appraising the bearing capacity of a structure after a fire. Rolled steels in general recover their original strength on cooling, whereas cold-forged steels lose some of their strength.

It is clear from Figure 14 that the yield point decreases at first

slightly with increasing temperature, but then more rapidly. At 500 to 600 ^{0}C the design stresses calculated in the construction are attained.

Contrary to a widespread misconception, the yield point of high quality prestressed rods is not below that of ordinary reinforcement rods. It is only in the temperature range of around 600^{0}C that all the values more or less coincide.

2.33 Rupture strain

Figure 16 shows the change in rupture strain of a mild steel under the influence of elevated temperatures, after ref. (27). Up to 200^{0}C the rupture strain at first decreases rapidly, but then in the region of higher temperatures it increases just as rapidly. At 600^{0}C the failure strain is about twice as great as at 20^{0}C.

2.34 Modulus of elasticity

Figure 17 shows the change in the modulus of elasticity of a non-siliceous S. M. mild steel as a function of the temperature. The behaviour of other steels is similar. It is clear from Figure 17 that the modulus of the steel drops to about half its value at room temperature between 500 and 600^{0}C. The decrease, therefore, is not as great as in concrete.

2.35 Thermal Conductivity and specific heat

The thermal conductivity of the steels normally used in construction decreases with increasing temperature. Figure 8 shows the heat transfer coefficient of pure iron and of structural steel with 0.8% carbon content. The values for most structural steels lie within the hatched area. It must be realized that at comparatively low temperatures, i.e. at the beginning of a fire, the heat transfer coefficients of steel are 30 to 50 times as great as that of concrete. This fact affects the behaviour of very heavily reinforced structural members in the fire.

The specific heat of iron and steel increases with increasing temperature. At room temperature the specific heat is approximately c = 0.11 $\frac{kcal}{kg \cdot {}^0C}$ and at 800^{0}C it is 0.2 $\frac{kcal}{kg \cdot {}^0C}$

2.4 Other materials

In the erection of concrete structures, especially where pre-

fabricated concrete parts are used, a number of other materials besides concrete and steel may be employed in the finished structure, e.g. as bonding agents, veneers, plasters or for sealing purposes. These materials include, for example, gypsum products, wood or wooden wares, and plastics. In what follows we shall briefly outline the most important properties of such materials with respect to fire.

Gypsum, as is known, hardens by hydration of the semihydrate or of the anhydrate to the dihydrate form. When gypsum is exposed to higher temperatures, some water is given off between 100 and 160°C and complete dehydration to anhydrate occurs at about 200°C. For this, considerable quantities of heat are consumed. Hence the heating of all constructions made of gypsum is greatly delayed at approximately 100°C. Use can be made of this fact at times, when rapid heating of a building element must be avoided.

However, after dehydration the gypsum loses much of its strength. Moreover, with gypsum plasters under very gas-tight floors the steam pressure may become so great that the entire plaster breaks away from the ceiling.

Wood occurs in concrete structures, e.g. in the form of small structural parts such as dowels, or the like. Wooden laths are also embedded in the concrete in the manufacture of certain concrete products with a view to securing projecting sections. Wooden parts surrounded by concrete on several sides do not ignite as easily as completely exposed ones, owing to their poor contact with the atmospheric oxygen. In addition, larger cross-sections always have a higher resistance to fire than delicate, thin structural parts. Caution is always required whenever parts of the whole construction which are essential for fire protection are secured to wooden parts. A critical temperature for the ignition of wood can only be stated approximately, since it varies greatly with the kind of wood, its moisture content, the atmospheric oxygen available, the surface texture and other factors. Under unfavourable conditions, however, ignition may be expected at 250 to 300°C. Some times, even lower ignition temperatures have been noted.

Ceiling fillers or slabs of mineralized wood wool have been found excellent in conjunction with concrete. The resistance of thin steel reinforced concrete ribbed ceilings to fire can be greatly improved by the use of these parts, since even in a fully raging fire they ash slowly, and thus constitute good insulation[29,41].

In the field of plastics, development is in full flood. No general

critical temperature for all plastics can be stated. Some plastics alter their properties even at lower temperatures than wood. Especially in the field of prefabrication, where plastics are popular as binding agents, adhesives or joint seals, therefore, fireproof requirements must be taken into account in the planning stage, because subsequent changes are frequently very difficult to carry out.

3. The Behaviour of Concrete and Steel-Reinforced Concrete Members in Fires

3.1 General

A knowledge of the change of properties of building materials will suggest important ideas concerning the behaviour of structures in fire. The fire test on the whole structural part, however, is decisive in appraising fire resistive quality. In many countries there are standards for conducting fire tests. These regulations are alike in principle, although, they differ from each other in detail.

Basically, it is laid down that the structural part to be investigated must be exposed to damaging fire[30], i.e. a fully developed conflagration. In order to get comparable test results it is customary to represent the development of an idealized fire by temperature-time curves. Figure 19 shows the temperature-time curves used in various countries. It will be noted that all the curves are very similar to each other, so that the results on constructions investigated by the standards of various countries on the whole are comparable with respect to temperature load. In order to get the best possible agreement, a uniform curve has been drawn by ISO, which fits the previous curves well. However, even under earlier conditions, differences in the results from fire tests in different countries have been due less to different temperature loads than to differences in test material and other test conditions.

In attempting to analyse the curves represented in Figure 19 it must first be realized that in the first half hour, at the start of the fire, the temperatures in the fire compartment rise very rapidly, and then increase more slowly. The rapid changes of temperature at the beginning of a fire in general result in large temperature differences in the structural member. At an early stage, therefore, considerable deformations or stresses occur. As the fire continues, there is a continuously progressive, but more even heating of the construction, which, unless a temperature equilibrium sets in before hand, can result in failure. The uniform temperature-time curves correspond approximately to the course followed by a fire of medium severity.

This is important, because for the most part the temperatures lie below the 1200°C limit. At this temperature, however, decomposition or destruction of a number of mineral building materials occurs. Several kinds of natural stones, for example, decompose. Many light-weight concretes, which show excellent behaviour in fire up to about 1200°C owing to their good thermal insulation properties, are destroyed when the temperatures go higher than this. Of course, such conditions would occur only in rare and exceptional cases.

In the course of a fire the structural parts ought to perform their functions. Essentially, these are as follows:

a) Bearing parts, such as beams, columns, ceilings, etc. must retain their bearing strength and stability.

b) Space separating parts must continue to be effective in separating space, i.e. they should not afford a passage for the fire.

Basically, two cases must be considered in the testing of structural members for their resistance to fire. On the one hand, the structures may be tested to failure, i.e. until they no longer satisfy the above functions a or b, as the case may be, or a time test may be made, i.e. it is determined whether the functions will continue to be performed during a predetermined time.

In order to be able to estimate whether and how long the bearing capacity of a structure will remain intact in a fire, the resistance of bearing structural parts to fire is tested under load. There is little difference in the regulations on this point from country to country. In general, however, the simple calculated permissible load is applied in the fire test. Since fire is a catastrophic occurrence, a safety factor of one is regarded as adequate as far as bearing capacity is concerned.

The stability is threatened by severe deformations. For example, in the case of walls exposed to the fire, on one side, the resulting deflection may produce such great eccentricity that the wall will collapse.

Space separating structural members must not allow the fire to pass. That is to say, no hot gases should seep through, and the temperatures on the side away from the fire must not rise above a certain value. The critical rise in temperature in most countries today is put at an average of about 140°C above the initial temperature, and at individual points up to 180°C is allowed. The reason for this requirement is to prevent easily ignitable material, which may by chance be stored on the unexposed side of the space separating members, from igniting, for otherwise the fire will

jump the barrier.

In a number of countries some relaxation of these rules is allowed for certain members, e.g. doors, parapets, or walls of glass blocks. Special demands are made of chimneys and roofing materials.

The duration of the fire test depends on the type of structure in which the test member is to be used. Requirements differ greatly in the various countries. In Germany for buildings threatened with fire, "fire-resistant", and in exceptional cases "highly fire-resistant" construction is required. "Fire-resistant" members, according to DIN 4102 must continue to perform their functions in the fire tests for 1 1/2 hours, and "highly fire-resistant" must do so for 3 hours. In other countries, for certain buildings such as warehouses, which are particularly exposed to fire hazards, still longer resistance times are prescribed.

Increasing the demands for duration of resistance against fire beyond a certain time does not always require substantial changes in the construction. This is because for many structural members there is a certain time at which, during the standardized fire test, heat equilibrium is more or less attained. In this state the heat losses on the side away from the fire are approximately equal to the input of heat on the side towards the fire. If the structure has not collapsed by this time its resistance of fire can be greatly increased by reaching this threshold.

To carry out the fire test, the structural member to be tested is installed in a fire test chamber, and, if it is a bearing member, is subjected to a load. Figure 20 shows a fire test chamber with a loading apparatus for walls. The wall is prepared for testing. Figure 21 shows, semi-schematically, a fire test chamber for ceiling constructions. Today the testing institutions of many countries have very modern fire test chambers with partially or fully automatic control.

The test member is installed in the fire test chamber in a manner that simulates as closely as possible the incorporation of the member in practice.

3.2 Walls of light weight and heavy concrete

3.21 Heating

Concrete walls can be manufactured either monolithically from heavy or light weight concrete, or may consist of slabs of considerable size or of blocks. Basically, we must distinguish between bearing and non-bearing

walls. The former have both bearing and space separating functions, whereas the latter operate only as space dividers. In general, thick walls of the kind required for outside walls in tall building constructions, due to thermal insulation considerations are not endangered by fire. In the case of thin walls, however, there is a possibility that the permissible rise in temperature on the side away from the fire may be exceeded.

Figure 22 shows the necessary thickness for massive walls of heavy and light weight concrete as a function of the test time, so that no increase of temperature greater than 140°C above the initial temperature will occur on the unexposed side. In the case of walls with cavities, the deciding factor is the thickness of solid material. The fire resistance time F increases exponentially with the wall thickness d, approximately according to the function $F = d^{1.7}$(31).

The critical increase of temperature on the side away from the fire up to test times of 3 hours is only important, generally speaking, in the case of heavy concretes and wall thicknesses of $d < 15$ cm. A few kinds of hollow blocks made of heavy concrete constitute an exception to this. As in the case of hollow blocks of other materials, in unfavourable cases it can happen that the shells on the side of the fire will burst after a certain time and that cracks will form through the entire block as the test continues. The permissible rise in temperature for light-weight concretes is seldom exceeded, even in the case of thin walls.

For masonry brick walls or slab walls, careful design and execution of the joints is extremely important. Grouting or smoothing of the joints may help, so as to ensure that no hot gases can penetrate through any gaps in the joints. Especially where thin plates are employed, the joints have to be profiled as far as possible, so that adjoining plates fit together in tongue and groove fashion.

The thermal insulation of the structural members during a fire is greatly affected by the moisture content. First of all, considerable quantities of heat are needed for the evaporation of the excess water, which quantities, of course are liberated again during condensation. Thus structural members with high moisture content rise rapidly to 100°C, but only slowly go above 100°C. In the case of concrete separating members, such as walls and ceilings, lags in the temperature rise are observed on the side away from the fire generally for temperatures between 80° and 100°C. Moisture often penetrates in the form of water of condensation during a fire at joints and hairline cracks. The temperature of these moist places is somewhat higher than that of their dry surroundings. In the case

of a heavy concrete wall with a bulk density of about 2400 kg/m^3, a difference of 1% by weight in moisture content results in temperature differences of about 25^{0}C on the unexposed surface, after all moisture has evaporated.

In Figure 23 the temperature curve in heavy concrete walls is represented as a function of the wall thickness, and in Figure 24 the same curve is shown for a 15 cm-thick wall of aerated concrete. The two diagrams are indicative only, since the scatters and temperature measurements inside the walls are very great owing to differences in moisture content, aggregates and bulk densities. The delay in heating at 100^{0}C is definite in the case of the aerated concrete wall, as is also a change in thermal conductivity above and below 100^{0}C.

Numerous compilations about fire tests on walls are available in the literature. Table II contains a selection of a few typical walls. Figure 25 shows a wall of slag concrete blocks after a 3-hour fire test.

3.22 Deformations and stability

When exposed to fire on one side, walls tend to bulge more or less extensively towards the fire side. After the fire, the bulging generally recedes, and the bulge may even appear on the other side. The magnitude of the bulging depends on the material, the wall thickness, and above all on the height of the wall. The mean bulging of unfixed walls can be estimated approximately by the following equation

$$f \simeq \frac{C \cdot h^2 \cdot (T1 - T2)}{8d}$$

where

C - material constant

f - deflection

h - height of wall

T_1 - temperature on the side towards the fire

T_2 - temperature on the side away from the fire

From this equation the important fact is realized that under otherwise equal conditions the bulging increases with the square of the height of the wall. In Figure 26 the conditions are represented for two walls which are similar except for their height. One wall is twice as high as the other. The bulging on one side, however, is four times as great. This must be pointed out, because fire tests on walls are usually made only on speci-

mens 2 m to 3 m high, i.e. the usual height of a storey. In the case of taller walls, which indeed are rare, but which do exist nonetheless, the effect of the greater bulging in fire should be taken into account, because stability may no longer be guaranteed owing to large eccentricity.

Figure 27 shows the bulges in two walls during fire tests.

Besides lateral bending, gaps may occur in the joints at the margins owing to bulging of the walls. Therefore, good connection between the wall and the adjacent structural members must be assured in the design.

3.3 Floor structures

3.31 Massive, steel reinforced concrete slabs

Whereas walls are predominantly compressively stressed, steel-reinforced concrete floors as a rule are subject to bending stress. Generally speaking, in floors of tall buildings there is tension on the underside of the slab and compression on the upper side. In a fire, a ceiling above the burning area is rapidly heated on the underside. Owing to the relatively poor heat conduction of concrete the steel reinforcement situated close to the underside in the tensile zone is heated more rapidly to the critical temperatures of steel than is the concrete on the upper side of the floor structure heated to its critical temperature. Therefore the heating of the steel reinforcement plays a decisive role in the resistance of steel reinforced concrete floors to fire.

The time during which a steel reinforced slab resists the fire is determined substantially by the following factors:

a) existing steel stress
b) steel covering
c) nature and thickness of lining underneath
d) quality of concrete
e) thickness of the floor
f) moisture content of the concrete
g) kind of aggregate
h) positioning of the slab (restraint)

Figures 14 and 29 show clearly that reducing the steel stress, or in other words over-reinforcing, increases the length of time of the resistance.

By increasing the covering of the steel, the heating rate of the steel reinforcement is reduced and thus the resistance of the steel re-

inforced concrete slab to fire is increased. For a slight increase in the steel covering, for example from 1 to 2 cm, the increase in the time of fire resistance, to be sure, is not very great. The reason is evident from Figure 28. It can be recognized that in the side of a wall or ceiling nearest the fire there is a very steep temperature gradient, which becomes flatter towards the inside. That is to say, the temperature difference between the underside of the floor exposed to the fire and the depth of 1 cm is greater than between 1 cm and 2 cm depth from the surface. A substantial increase in the time before fracture is obtained only with still greater steel coverings. Plasters in earlier tests showed greater protective effect than corresponding increases in steel covering. An essential condition, however, is that the plaster adheres well. The explanation may be sought in the fact that plaster has a lower bulk density than concrete and thus shows greater thermal insulating properties. As a consequence the temperature gradient in the vicinity of the concrete ceiling surface on the side of the fire is steeper than that in Figure 28. Moreover, heat conduction is hindered by the transition between different layers. On the debit side it may be said that in practice plasters are frequently applied in thinner coats than contemplated in the design and that green plasters with high moisture content are quickly stripped off in a fire. This is especially true if the underside of the concrete is smooth. Since the test described in ref. (25) was made, concrete technology has advanced. Today, in general more compact concretes are produced. It is therefore easy to understand that in more recent fire tests the favourable effects of plasters are not always evident, since the plasters have been detached from the concrete at an early stage in the fire. Priority should be given to this problem in the future, since in prefabricated structures traditional plasters may be used less than before, for other reasons as well.

For a number of years special coatings, e.g. of asbestos, vermiculite and perlite bases, have been used to increase fire protection. With these coatings, even comparatively thin ones, the heat insulation can be so greatly improved that the fire resistance time is increased to many times that of the uncoated floors. Figure 29 gives two examples showing the effect of coatings on the heating rate of the steel reinforcement rods and hence on the fire resistance time of reinforced concrete floors. The curves may be designated as follows:

Curve 1 - uncoated steel reinforced concrete floors (covering depth 1 cm) after ref.[25];

Curve 2 - the same construction, but with a lime-cement plaster 1.5 cm thick;

Curve 3 - the same construction, but with a cement-lime-vermiculite coating 1.5 cm thick.

The effect of concrete quality on the fire resistance time has not yet been fully clarified. In earlier tests it was established that for poor and medium quality concrete up to approximately B 300 the ability of a steel reinforced concrete floor to resist fire increased somewhat with increasing concrete grades. These results, in more recent tests, which were not always confirmed are somewhat surprising, since the bulk density and the thermal conductivity of the concrete in general increases with increasing grade. Perhaps the explanation is that with the increase in concrete grade the modulus of elasticity also increases. Given equal thickness and reinforcement the floor, however, a higher modulus of elasticity of the concrete means a lower steel stress. In going from a B 160 to a B 300 the modulus of elasticity increases by approximately 50%. This displaces the neutral axis of the cross-section upwards, and, for a concrete floor 12 cm thick with an average reinforcement component, causes a drop in steel stressing of about 6 to 10% (Figure 30). With an unplastered ceiling this corresponds to an extension of the fire resistance time by about 10 minutes. However, these findings for comparatively low grades of concrete must not be applied to higher grades, because here the modulus of elasticity does not increase greatly, and secondly very compact concretes do not behave at all favourably in fires, and under the most unfavourable conditions tend to spall.

The considerable effect of the thickness of the floor on the fire-resistance time is represented in Figure 31. This relates to concrete slabs reinforced with comparable steel rods. Aside from the fact that comparatively large cross-sections behave more favourably in fire owing to their greater heat capacity, it may also be assumed here that despite mathematically equal calculated steel stress, the actual steel stress in a thicker floor is less. Another favourable result is that the changes of form of a thick floor are comparatively less than those of the thinner one. Moreover, it should be borne in mind that under otherwise equal conditions, the moisture content of a thicker member is generally greater and hence it takes longer to heat through. As will be shown later, the fire resistance times in other structural members, also, are increased by increasing the cross-sections. The influence of floor thickness on fire-resistance time is

of special importance in practice, because in Germany almost all relatively extensive series of fire test have been carried out on steel reinforced concrete floors of thicknesses less than 10 cm. The values obtained formed the basis for the pertinent regulations. However, for the thicker floors now being used predominantly in practice, these values are much too unfavourable. This gain may be particularly important especially in prefabricated construction, where for a great variety of reasons plastering on the underside of steel reinforced concrete floors is not possible. A fire-resistance time of 90 min ("fire resistant") is readily attainable with floor thicknesses of about 15 cm, increasing the steel covering from 1 to 2 - 3 cm.

The static system also influences the bearing capacity of a floor in a fire. Continuous slabs are more favourable than single bay slabs. In the case of continuous slabs, before failure of the strongly heated horizontal reinforcement, at first the still relatively cool upper column reinforcements act with increased stresses as cantilevering reinforcement. As a consequence there is a relieving of the stresses in the bay reinforcements. Collapse does not occur until the yield point is exceeded in lower and upper reinforcements. From a fireproofing point of view, therefore, a continuous upper reinforcement throughout the bay is especially favourable.

The behaviour of single-bay slabs fixed at all sides has not yet been fully clarified[34,67].

Table III gives a summary of the increases of fire-resistance time as a result of the above-mentioned measures.

Besides these possibilities, the resistance of massive steel-reinforced concrete slabs can also be affected by the choice of aggregates. To be sure, the available data do not provide a uniform picture, since obviously other factors, for example, moisture content at the time of the test, the restraint of the slabs in the experiment, etc., have a very strong influence on the test results. According to Davey and Ashton[34], concrete floors of similar nature but made with different aggregates had the following fire resistance times:

Flint	31 minutes
Dolerith	41 minutes
Basalt	65 minutes
Crushed brick	2 hours

3.32 Prefabricated floors

3.321 Fire-resistance time

In recent years prefabricated floors have been very widely used. Among these a few basic types may be singled out from the very large number of systems found on the market.

1. Beam floors with intermediate structural members.
2. Ribbed floors with intermediate bearing members or compressed slabs of poured-in-place concrete.
3. Floors consisting of beams or planks laid side by side.
4. Prefabricated slab floors.

The behaviour of prefabricated floors in fire in principle is similar to that of solid slabs. Since the steel reinforcements are frequently better protected than in solid slabs, in many cases a longer fire-resistance time may be expected than solid slabs of equal thickness.

Accordingly the measures indicated under point no. 3.31 for solid slabs for improving the fire-resistance time hold also for prefabricated floors. Investigations have shown that almost all non-prestressed reinforced concrete floors which were given a coating of lime-cement 1 1/2 cm thick on the underside survive the required test time of 1 1/2 hours in the fire test according to German regulations.

In Table IV, examples of typical prefabricated floors are given with the corresponding test results. For comparison, the results of a few solid floor slabs are also listed.

3.322 Deformations

Besides bearing capacity, changes of shape in the course of a fire are also important in the behaviour of prefabricated floors. Directly after the start of fire exposure, floors exposed to the fire begin to change their shape. As a rule they bend toward the fire, i.e. they begin to sag downward. Figure 32 shows the deflections of some of the prefabricated floors listed in Table IV. The value of the deflection depends not only on the span and thickness of the floor, but also on the construction and the material. For example, hollow beams of heavy concrete or solid lightweight concrete slabs bend somewhat more than solid heavy concrete slabs under similar conditions.

In the case of prefabricated floors of beams or planks laid side by

side, we wanted to know how they would behave in the presence of fire under partial loading. If the deformation of loaded parts of a floor compared with that of the unloaded parts, there is a danger of shearing at the separation joints and the fire breaking through. A number of floors were therefore investigated in fires, only half of which were loaded on one side with the calculated permissible loads[43]. All the floors (floors without transverse reinforcement and with relatively large deflection were deliberately chosen for the test) resisted the fire under these conditions and the fire did not break through. In addition, small floor units of aerated concrete were tested in unloaded and loaded conditions. As is evident from Figure 33, the results showed that the deflections were almost the same at the beginning of the fire test. This means that the initial deflection depended entirely on the temperature difference between upper and lower sides. Only after exposure to the fire for over 20 minutes did the deflection of the loaded floor begin to increase more strongly than that of the unloaded. At this time a gradual reduction of the modulus of elasticity began. Even at this time, however, the deflection component due to temperature differences remained relatively large compared with that due to the loading.

In certain exceptional cases, floors are deflected at the start of a fire not towards the fire, but away from it. This happens, for example, if the ribs of a ribbed floor are heavily insulated in order to protect the steel from heating, and the upper compressed slab heats up more rapidly than the ribs. On the other hand, a permanent upward deflection after the termination of a fire is comparatively frequent. This occurs, for example, with some aerated concrete floors. As a result of fire exposure the concrete after cooling undergoes a slight, permanent increase of volume, while the steel reinforcements revert to their original length. The result is a "prestressing" effect which causes an upward curvature.

3.323 Downward propagation of fire

Generally speaking, fire tests are carried out under the obvious assumption that fire spreads upward from below, i.e. floors above the fire are mainly endangered. Sometimes, however, we are interested in the behaviour of floors where the fire is propagated downward. For this purpose, tests were made with a partial prefabricated floor of steel reinforced hollow planks laid side by side[44]. The same floor had already been tested in the usual way (cf. Table IV, No. 4). The tests showed that the floor

remained intact under fire exposure from above during the test time of 1 1/2 hours. The permanent deflection of the floor after the fire was greater than in the regular tests. Figure 34 shows the upper side of a concrete plank of the test floor after the fire.

3.324 Prefabricated slab floors

In the prefabricated, large-area steel reinforced concrete floors, which have very recently come into use in apartment houses and in industrial construction, it is often not possible to apply a plaster to the underside of the floor during the prefabrication process. Subsequent plastering may nullify the economy of this method of construction. Under these circumstances the fire-resistance time can be increased by the following measures:

1. By decreasing the reinforcement stress
2. By increasing the reinforcement covering
3. By the use of suitable aggregates
4. By incorporation of an ancillary layer of lightweight concrete with low thermal conductivity
5. By incorporation of slabs of heat-insulating materials either in the concreting stage, or subsequently, over the finished and installed ceiling.

Simply increasing the reinforcement covering is of limited value. An unplastered high-grade concrete floor sometimes fails because one or more pieces of the concrete surface spall off and at these points the steel reinforcements rapidly become heated. Structural measures should therefore be taken, e.g. by incorporation of a light wire mesh, to ensure that spalled portions cannot fall off. At the very high temperatures occurring in a fire, heat transfer takes place primarily by radiation, and therefore particles of concrete that have loosened but do not fall still provide adequate protection for the steel reinforcements.

Ancillary layers are made mainly of lightweight concrete or from slabs of other heat-insulating materials. The required fire protection has also been attained in prefabricated concrete slabs simply with the aid of gypsum boards or cement-bound wood-wool lightweight boards which are merely concreted over (cf. Table IV, no. 2).

3.4 Prefabricated steel reinforced concrete steps

Steps constitute a special case of the "floor structure". Opinions differ as to whether the behaviour of steps in a fire must meet the same requirements as floors, i.e. whether they must remain effective as space separators and at the same time retain their bearing capacity, or whether only the latter is important. At the present time German regulations require both specifications to be met. However, an amendment may be expected.

Prefabricated steps are generally produced in the following structural forms:

1. Large plates with steps mounted on them
2. Beams placed side by side
3. Side walls with steps placed between them
4. Beams with steps mounted on them

Basically the same considerations apply from a fire standpoint for steps as for floors. Figure 35 shows examples of two designs which can be regarded as "fire-resistant" according to the existing German regulations, i.e. they have a fire resistance-time of more than 1 1/2 hours[45,46].

3.5 Steel reinforced concrete beams

Test results on the behaviour of steel reinforced concrete beams in fire are available from various countries[25,34,47]. In some instances T-beams, and in others beams of rectangular cross-section, have been investigated. From these tests it may be assumed that the behaviour of steel reinforced concrete beams is determined by substantially the same quantities as that of slabs. However, no direct comparison between steel reinforced slabs and beams is possible, since in the case of beams the ratio of the surface attached by the fire to the cross-section is less favourable than it is for slabs. On the other hand, in the case of beams the greater structural height is a positive factor.

The effect of the steel covering has been investigated in considerable detail. In the case of beams, it was again found that the fire-resistance time did indeed increase with increasing cover, but not proportionally to the thickness of the covering. Figure 36 shows the effect of steel reinforcement covering on the fire-resistance time. In all the tests, which owing to the different sizes of beams and different bearing arrangements are not directly comparable, it was clearly evident that the increase

of fire-resistance time is less than the increase of steel reinforcement cover.

The age of the member influence the fire-resistance time of T-beams more definitely than plates[25]. In Figure 37 the effect of age is represented. Presumably the increase of fire-resistance time with increasing age can be attributed to a simultaneous increase of the quality of the concrete and the modulus of elasticity.

As in the case of slabs, an increase of fire-resistance time can also be obtained by a suitable choice of aggregates[34]. Three beams were produced which differ only with respect to the kind of aggregates employed. Figure 38 shows the cross-section of the beams; the test results are contained in Table V.

In considering the earlier test results on beams it should be mentioned that the beams were made of relatively low-strength concretes. Nowadays higher grade concretes are generally employed, especially in the manufacture of prefabricated steel reinforced beams. With structural members of high quality concretes there is a danger of the explosive spalling of pieces of concrete from the surface. If parts of the beam reinforcements are thus exposed, rapid failure occurs. It is assumed that this spalling is caused by water vapour over pressure due to evaporation of the water present in the concrete. This view is supported by the fact that such spalling occurs predominantly in green and very high quality, i.e. very compact concretes, shortly after the start of exposure to the fire. Probably other influences also contribute to the tendency of concrete to spall in fire. For example, an influence of the aggregates cannot be ruled out. Probably this has more to do with intrinsic porosity than with the kind of material involved. For example, spalling was observed in very moist mortars made of lightweight aggregates. This may be due to the fact that on rapid heating moisture stored in the aggregate cannot escape rapidly enough through the more compact cement paste.

It is probable also that the tendency to spall is influenced by the design of the cross-section and by the mechanical stress during the fire. A preference towards spalling is noted, for example, in the slender stems of I-beams and on the under sides of the cross slabs of T-beams. In these members strong heating produces very high compressive stresses owing to the inhibited expansion, and these stresses cannot be absorbed even by concretes of very high strength. More precise investigations should be carried out and published as quickly as possible into the causes of spalling, which

certainly does not occur on all occasions.

3.6 Columns

3.61 Steel reinforced concrete columns

Whereas in steel reinforced concrete beams and floors the reinforcements are generally subjected to tensile stress, in the case of reinforced concrete columns the steel is stressed in compression. The compressive strength of steel, i.e. the compressive yield point, is also reduced under the influence of high temperatures, so that in the presence of fire there is a greater danger of buckling.

In earlier years, extensive series of fire tests were carried out on steel reinforced concrete columns[48-50]. The fundamental tests of Ingberg and his co-workers are still indicative. At that time concrete quality grades were still comparatively low, the column cross-sections were large and the reinforcement ratios were low. Very high fire-resistance times were attained with plastered columns where the plaster was furnished with a light wire-mesh inlay, which prevented the plaster from dropping off in the course of the fire.

More recently there has been a strong tendency in construction towards reducing the dimensions of individual structural members. In particular, architects have been calling for ever slenderer columns in façade. This has meant the employment of higher concrete grades and larger proportions of reinforcement.

With the advance of prefabricated construction a desire has arisen to use uncoated columns with high fire-resistance time, since subsequent coating tends to nullify the economic advantages of prefabricated construction. Therefore, new series of fire tests have been conducted recently at various places with slender uncoated, and in some cases heavily reinforced columns[34,51-55].

In addition to depending on the cross-section size, the resistance of a steel reinforced concrete column is influenced primarily by effectiveness of the covering over the steel. Owing to the severe compressive stresses in the boundary zone, this covering is more severely stressed at the beginning of the fire in columns than in other structural members. If the concrete or coating falls off prematurely, then even thick columns will not attain long fire-resistance times. The most important measures for raising the capacity for resistance of steel concrete columns to fire lies in holding the

covering, whether of concrete or plaster, in place while it is being attacked by the fire. This is best done by inlaying a light wire mesh between the reinforcement and the surface of the concrete. When this is done, constant heating of the column, and hence a more even course of the fire test, is obtained. The following observation may be made in here. With the use of certain aggregate materials, e.g. limestone or blast furnace slag fragments, the concrete covering is less liable to fall off. In such cases, the wire mesh inlay can sometimes be dispensed with.

The bearing capacity of a compressively stressed column is described approximately by the following addition law:

$$P_{collapse} = K_b \cdot F_b + \sigma_{st} \cdot F_e$$

where:

K_b = cube strength of the concrete
F_b = cross section of the concrete
σ_{st} = tensile yield limit of the steel
F_e = steel cross-section

In principle these conditions also apply in the course of heating, taking into account, obviously, the altered properties of the material. From the equation the following conclusions may be drawn:

With increasing concrete cross-section, given to equal reinforcement ratio, the fire-resistance time has increased, because complete heating of a larger cross-section up to the critical temperatures must take longer.

However, increasing the reinforcement percentage, i.e. increasing F_e, under otherwise similar conditions has not so great an opposite effect, since the steel reinforcements of a heavily reinforced column heat up just about as quickly to the critical temperature as they do in a lightly reinforced column.

The time until failure is also influenced by the size of the applied load. No data are available on the influence of the degree of restraint and the slenderness on the fire-resistance time. It can, however, be assumed that the test results would be greatly influenced.

Figure 9 shows the results of recent fire tests with prefabricated columns carried out under the auspices of the Federal Association of the Concrete Block Industry(53). In this, columns 3.60 m long with the following cross-sections were tested:

$F = 15 \text{ cm} \times 20 \text{ cm} = 300 \text{ cm}^2$
$F = 15 \text{ cm} \times 24 \text{ cm} = 360 \text{ cm}^2$

$F = 20\ cm \times 20\ cm = 400\ cm^2$

$F = 24\ cm \times 30\ cm = 720\ cm^2$

For some of the columns limestone was used as the aggregate, and for others a quartzitic material. Figure 40 shows the column cross-sections. Figure 41 shows a column after the fire test and the quenching water test. The inlaid wire mesh is clearly shown.

Figure 39 shows a definite increase of the fire-resistance time with increasing concrete cross-section. Striking, however, is the excellent behaviour of thin columns, especially when limestone is used as the aggregate. British fire tests showed a similar result on somewhat shorter columns, and some of these are plotted in Figure 39 as well[34]. In Figure 42 the results of the fire tests of the columns represented in Figure 40 are compared with the results of more recent French fire tests[52]. Since in the latter the test arrangement (column length 2.30 m) differed, the results are not directly comparable. Nevertheless it can again be recognized that the fire-resistance times in the case of a French test, as in the German results, are more or less proportional to the concrete cross-section. In the French tests the reinforcement ratio μ was substantially decreased with increasing concrete cross-section, so that the applied load on all columns was equal. Despite the radically altered reinforcement component, the tendency is the same as in the test with constant reinforcement component.

Another successful means of increasing the fire-resistance time of columns was found to be the displacement of a substantial part of the reinforcement towards the interior of the column. With heavily loaded spiral columns, which have a somewhat greater resistance than rectangular columns, fire resistance times of more than 1 1/2 hour were obtained[55].

To sum up, the behaviour of steel reinforced concrete columns in fire may be represented as follows:

1. The most important preventive means is to see that the covering of the steel does not drop off. A wire mesh inlay will accomplish this.
2. The choice of aggregates has a very great influence. Limestone concrete columns behave better than concretes with quartzitic aggregates. Basalt and blast furnace slags fall somewhere between the above-mentioned groups.
3. Under the calculated permissible load columns of larger cross-sectional dimensions fail later than slender columns. The

influence of the reinforcement ratio is slight compared with that of the concrete cross-section.

4. Given suitable design and a good use of proper aggregates, high fire-resistance times can be obtained even for very slender columns.

3.62 Unreinforced columns

From an earlier time we have the results of fire tests on unreinforced columns[50]. Figure 43 shows that very thick columns may fail faster in a fire than columns of moderate dimensions. This observation has sometimes been made in real fires as well. It should be borne in mind, of course, that the concrete grade of the tested column was very low.

The reason for this comparatively poor behaviour of the thick columns may be that the share of the heated surface in the total cross-section of the column is comparatively small. The interior of the column, owing to lower temperatures does not then share the longitudinal elongation of the surface, and heavy loads must be borne by the thin surface. In the case of low-grade concretes, the bearing capacity in the boundary zone is then quickly exceeded.

4. The Behaviour of Prestressed Concrete Structures in a Fire

4.1 General

Non-prestressed steel-reinforced concrete and prestressed concrete have much in common from the standpoint of fire. The decisive factors as far as the fire-resistance time of a structural member is concerned are its cross-section and the heating of the steel reinforcements. Any means by which this heating is delayed have a positive fact on the resistance of the construction to fire. The heating of the steel reinforcements is affected essentially by the size of the cross-section and the shape of the member, as well as the covering of the steel.

Generally speaking, the dimensions of prestressed concrete constructions are smaller than ordinary steel-reinforced member of equal bearing capacity. This entails a smaller heat capacity and a more rapid complete heating of the members, and has the effect of reducing the fire-resistance time. On the other hand, the example of thin, heavily steel-reinforced concrete columns dealt with above, where similar conditions prevail, shows that suitable means are available in order to give even

slender cross-sections a relatively high resistance to fire.

The main differences between prestressed and ordinary steel-reinforced concrete from the point of view of fire technology are as follows:

1. The reinforcement rods are of higher quality (cf. Figure 14). The drop in high temperature yield point of the prestressed reinforcement rods is greater than that of ordinary structural steels for non-prestressed reinforcements.
2. The steel stresses are not proportional to the external moment. Therefore a reduction of the external load is not so favourable as in non-prestressed concrete.
3. Different dimensioning procedures are employed. In many instances the actual steel stresses in non-prestressed concrete are lower than the calculated stresses. This results in an extra safety factor with respect to fire.

4.2 Prestressed concrete floors

Prestressed concrete floors are usually prefabricated parts. The shapes employed are similar to those of non-prestressed prefabricated floors. The different types may be used either with or without additional poured-in-place concrete. The bearing parts of prestressed concrete floors are generally produced in the prestressing bed with immediate bond.

The critical temperature of the reinforcement rods in the case of prestressed concrete floors is somewhat lower than for steel-reinforced concrete floors. The reason is that the ratio of the working stresses of the steel in relation to the decrease in high temperature yield point is less favourable than for ordinary steel-reinforced floors. Moreover, with progressive heating the prestressing, owing to changes in the modulus of elasticity of steel and concrete, and owing to the creep of the heated steel, is gradually relaxed and finally disappears altogether. When this happens the reinforcement then acts simply as a non-prestressed reinforcement. The neutral axis changes its position. This results in severe overloading of the compression zone of the concrete, and ultimately results in failure. Characteristic of the behaviour of prestressed concrete floors in a fire is a very marked more rapid increase in its deflection before fracture occurs. This happens in non-prestressed floors, as well, of course, but is not so readily recognizable.

In order to attain a maximum fire-resistance time, it is necessary,

for the above reasons, to delay the heating of the prestressed reinforcement rods in the fire as long as possible. An effective way of doing this has been found, for example, to be the application of an ancillary layer of lightweight concrete below the supporting floor beams. This increases both the heat insulation and the adherence of the plaster, which is of great importance in the attainment of a high fire-resistance time value.

In Table VI we represented two typical prestressed concrete floors which satisfied the required fire-resistance time of 1 1/2 hours in Germany ("fire-resistant" construction)[37]. Many other floor tests are described in[52,56].

A considerable improvement in the fire-resistance times of prestressed concrete floors can be attained by means of special plasters (cf. Section 3.31) and also by suspended membrane ceilings. Either of these means can increase the fire-resistance time up to 4 hours and more in certain designs.

A slight increase in the fire-resistance time can also be obtained by "over-dimensioning".

As already stated above, in prestressed concrete designs the external moment is not proportional to the reinforcement stresses. Nevertheless, the fire-resistance time of a floor under small load is somewhat increased, because during relaxation of the prestress the instant at which tension appears in the compressive tensile zone occurs somewhat later. There have been exceptional instances, of course, where the failure of unloaded floors occurred earlier than in similar floors subjected to a load. Presumably this is because in the case of the unloaded floor the compressive stress due to prestressing and thermal expansion on the side exposed to the fire was too great, and hence premature splitting-off of the reinforcement covering occurred.

To sum up, the following can be stated about the behaviour of prestressed concrete prefabricated floors:

1. The decisive factors governing the fire-resistance time are the dimensions of the cross-section and especially the thermal insulation of the reinforcement covering.
2. The critical steel temperature is somewhat below that for non-prestressed reinforced floors.
3. Suitable coats of plaster or suspended membrane ceilings can result in very long fire-resistance times of 4 hours and more.

4. Most fireproofing measures and modifications have effects similar to those exerted on non-prestressed reinforced floor construction.

4.3 Prestressed concrete beams

In recent years there have been numerous systematic experiments in Great Britain[57], the Netherlands[47], the U.S.A.[58,59,67] and in other countries[60-63,69] on the behaviour of prestressed concrete beams in fires. In these tests the following influences, among others, on the behaviour of prestressed concrete beams were investigated.

1. Form and dimensions of beam cross-section
2. Covering of reinforcement rods with concrete
3. Nature of the bond
4. Kind of aggregate
5. Load
6. End restraint
7. Additional heat insulation of surface
8. Causes of spalling

Although agreement in all points is not found in the various test reports, they nevertheless furnish very valuable information.

Since in the case of a beam the heat attacks from several sides, it is heated through more rapidly than a slab which is exposed to the fire on one side only. The smaller the beam cross-section, the less time it takes to heat it through. For this reason, as in the case of steel-reinforced concrete columns, the size of the cross-section has been found to be an essential factor governing the fire-resistance time.

More difficult to recognize is the influence of the shape of the cross-section. Rectangular and thick I-cross-sections behave very similarly. With flat beam cross-sections, and the TT-cross-sections which are common in America, it is less easy to eliminate the size of the cross-section as a parameter for the fire-resistance time. Basically, however, for these forms also it is true that the fire-resistance time increases with increasing cross-section area. Table VII gives a survey of the cross-section forms of steel-reinforced concrete beams investigated.

The rate of heating of the steel reinforcements depends not only on the beam cross-section, but primarily also on the covering. The higher the desired fire resistance, the greater must be the amount of concrete covering

of steel reinforcements. In Figure 44 we have the results of several tests on steel-reinforced concrete beams with instant bond. The influence of the beam cross-section and the reinforcement covering is clear. The minimum steel coverings listed in Table VIII are obtained according to ref. (47) for the various fire-resistance times, taking into account the beam cross-section. The values given are based on the assumption that the covering will not fall off and that the concrete is of a technological quality such that no spalling will occur. From a comparison of Figure 44 and Table VIII it can be inferred that the values of Table VIII already provide a certain safety factor.

The values listed in Table VIII apply to compact cross-sections. With very high, slender beams the covering must sometimes be still further increased. It is very important, especially in the case of larger beams, that the concrete covering cannot fall off during the fire. For this reason longitudinal bar and lateral tie reinforcements must be applied.

In comparing beams with instant and delayed bond, similar results, on the whole, were obtained(47). With small coverings, beams prestressed after pouring often showed somewhat better behaviour. This is because the high cement injection mortar in the prestressing channels has poor heat conductivity (no aggregates) and a substantially higher specific water content than the concrete. This water content delays the heating (cf. Section 2.24). To be sure, the high content can also have a deleterious effect if the vapour pressure becomes too great and causes concrete to spall. Escape routes for the water vapour should therefore be provided.

The investigation of a lightweight concrete of bloating clay with about 2/3 the bulk density of the heavy concrete investigated for comparison, resulted in about 20% longer fire-resistance times(59). The author is unaware of many systematic investigations of the influence of other aggregates such as limestone or blast-furnace slag, which was very effective in the case of columns.

Reducing the external loading has a positive effect. However, for the reasons explained in Section 4.1 this influence is not nearly so great as in unreinforced concrete(47).

As in the case of non-prestressed reinforced structural members, we do not have adequate information, either, about the influence of end restraint on the fire-resistance time of prestressed concrete beams although in practice such restraint is almost always present. In British tests(57), where the longitudinal expansion was prevented, an increase of fire-resist-

ance time was found for small beam dimensions, but a decrease for large ones. According to American investigations[58] the effect of fixing depends decisively on the magnitude of the "restraining force". This influence is represented systematically in Figure 45. According to this curve the shortest fire-resistance times are obtained for total end restraint and simple supports. Between these two extremes the bearing capacity under fire stress can be prolonged in such a way that, for example, in the case of space separating structural members of prestressed concrete the increase in temperature on the side away from the fire, not the collapse of the member, becomes the determining factor for the resistance time.

Obviously prestressed concrete beams can be very well protected against attack by fire by means of an additionally applied heat insulation layer, e.g. of vermiculite concrete. According to Ashton and Bate, encasing a beam in vermiculite concrete 25 mm thick increases the fire-resistance time by more than 2 1/2 hours. American tests show[57] that the fire-resistance time of a TT-beam is doubled by spraying on a 12 mm coating of vermiculite and tripled by spraying a 25 mm thick coating.

5. The Behaviour of Other Concrete Parts and Construction in a Fire

5.1 Chimneys of lightweight concrete pipe sections

Unlike the structural members hitherto described, chimneys are under stress from high temperatures not only in the catastrophic case, but also in the course of normal operation. To be sure, the temperatures of the combustion gases, which are brought through a flue, are considerably lower than those that occur in a severe fire. In domestic heating plants temperatures between 200 and 400°C can be expected.

In addition to these "normal" temperature stresses, however, still higher temperatures can occur in chimneys. This is the case, for example, in chimney fires and so-called "burning-out". Certain fuels tend to deposit considerable quantities of shining soot in the chimney. This soot under suitable conditions can be ignited. This can happen either involuntarily, or can be brought about deliberately by the chimney sweep or the fire department and kept under control. When chimneys are being burned-out, temperatures of 1000°C and more occur[64]. Different views are entertained on the effective duration of these high temperatures[65]. They probably depend very strongly on the specific conditions, especially of the fuels

employed. In any case, conditions during burning-out are very similar to those arising in a fire.

A number of requirements must therefore be laid down for chimneys from the point of view of fire protection:

1. In normal operation the outside should not get too warm, so that undesirable phenomena such as the discolouration of tapestries, etc., will not occur.
2. They must be sufficiently gas tight to prevent any combustion gases from leaking into adjoining rooms.
3. In a chimney fire, the temperatures of the outside of the chimney must not exceed the limits laid down for space separating structures (cf. Section 3.1), so that no ignition by transfer will occur in adjoining rooms. Hence any cracks that may occur must not exceed a certain width.
4. Chimneys must have adequate mechanical strength, because they are subject to mechanical stress both during fires and when being swept.
5. They must be sufficiently stable.

The above-mentioned requirements are determined by tests which differ in execution from the normal fire tests described in Section 3.1. Figure 46 shows the temperature-time curves for heating and burning-out tests according to DIN 18160, sheet 6 "heating plants-testing principles for domestic chimneys". First a specimen is subjected to a heating test, and then, after cooling to a burn-out test.

In recent years chimney pipe sections of lightweight concrete have proved very effective. Systematic tests have been carried out by Seekamp and Möhler[65]. Many chimneys made of such pipe sections have also been tested on behalf of individual companies.

As an aggregate for lightweight concrete chimney pipe sections, preferably blast furnace pumice stone slag, crushed brick, bloating clay and broken, porous lava slags are employed. In order to meet the contradictory requirements of good heat insulation and adequate gas tightness, a narrowly restricted range of bulk densites, depending on the material, must be observed.

For small cross-sections generally one-piece units are employed which may be of either solid wall or cellular design. For larger cross-sections, multiple-shell units have been found more effective. Where chimneys have to be burned-out fairly often it has been found useful

to incorporate a thin reinforcement of annealed wire in the chimney units. Figure 47 shows examples of several types of chimney pipe units.

5.2 Glass block walls

Glass block walls occupy a special place in the field of wall construction. In a fire these must remain space sealing. No requirements are stated for the rise of temperature on the side away from the fire. Figure 48 shows a glass block wall in the course of a fire. This wall has satisfied the requirements laid down in the German regulation, whereby glass bricks must withstand standard fire for one hour and then withstand the quench water tests. The deformation of the wall, which after cooling is restored to its original shape, is clearly evident in this picture.

5.3 Roofing

Concrete roofing materials consist predominantly of:
lightweight concrete slabs,
asbestos-cement slabs, or
concrete roofing tiles.

For the case of lightweight concrete slabs, what has been said in Section 3.3 again applies. The latter two products constitute roof covering materials in the narrower sense. Asbestos cement slabs and concrete roofing tiles are among the so-called "hard roof coverings". Both types are sufficiently resistant to air-borne fire and radiant heat, i.e. they protect a building from the effects of neighbouring structures on fire. Figure 49 shows a timber frame building which is veneered on the gable side with asbestos cement slabs. The building is completely undamaged, although a neighbouring building has been burned to the ground.

6. Mathematical Determination of the Fire Resistance Time

Attempts were made at an early date to determine the fire resistance of construction mathematically in advance. For example, Busch[9] attempted to calculate the time taken for structural members to heat through in a fire with the aid of the familiar general Fourier differential equation for heat transfer, and from this to determine the fire-resistance time.

The values thus obtained and the heating through curves are very interesting, to be sure, and give many indications and suggestions, but they describe the actual conditions in many cases unsatisfactorily. This

is mainly because the material constants change radically with the temperature and generally they are difficult to take into account. Also the conditions under which a test is run can vary greatly, so that for example the moisture content may have a decisive effect on the behaviour during a fire.

Recently attempts have been made to deal with the behaviour of constructions analytically not on the basis of mathematical theorems, but to draw general conclusions from the results of fire tests on various structural members. In the Netherlands, for example, the rate of heating of steel reinforcements has been suggested[47] as a basic parameter for the behaviour of reinforced concrete structures. It can be assumed that with detailed analysis of the test material at present on hand, a number of other essential influence factors can be eliminated. This would apply, for example, to the size and shape of cross-sections, the static system, of the kind of aggregates, etc.

On the basis of these magnitudes, a sufficiently accurate advance estimate of the fire-resistance time should be attainable. At the same time, it should always be borne in mind that the required fire-resistance times, like the variation of the fire with respect to time as assumed in the standards (temperature-time curves), are only conventions. For the behaviour of a structure in a real fire, it is of minor importance whether a given structural member withstands the standard fire, e.g. for 1 hour and 50 minutes or for 2 hours and 10 minutes. The order of magnitude is what matters. In any event it makes little sense to the author if a large number of constructions very similar to each other have to be tested in extensive experiments in order to satisfy a standard. This is precisely what has happened in recent years in the field of concrete products and prefabricated concrete parts.

7. The Behaviour of Concrete Products and Prefabricated Concrete Parts in Real Fires

In the foregoing sections we have dealt mainly with the behaviour of the various concrete constructions in the fire test under standardized conditions. Only through experiments under strictly defined conditions was it possible to investigate systematically the influences on the resistance of structural members. However, it is the behaviour of construct-

ions in a real fire which in the last analysis really counts. To what extent do the results of fire tests accord with experience from real fires?

From the very outset it cannot be expected that detailed findings, for example, the propagation of a temperature through a construction, will be in exact agreement in fire tests and real fires. A comparison must rather be made primarily in terms of the general behaviour. Considered this way there is far-reaching agreement between tests and actual fire results. To be sure, the elimination of influence factors is more difficult with respect to real fires than for fire tests. On the other hand, in fire disasters, a number of characteristics become evident which in tests could not be so clearly observed.

Generally it may be stated that almost all measures which increase the bearing capacity of a construction under normal temperature conditions are even more effective against stresses due to fire. This can be realized particularly after fires in older buildings from the early days of concrete construction, in which the present-day building code had not yet been applied.

Like the fire tests, practical experience also shows that the covering of the reinforcements is of particular importance in all steel-reinforced concrete structural members. By investigations on burned buildings it has been established(66), that the thickness of the reinforcement covering is less important than that it be prevented from spalling during the course of a fire. To ensure this, in beams and columns sufficiently compact longitudinal bar and lateral tire reinforcements are required. Figure 50 shows that if the stirrup spacing is too wide the reinforcement covering falls off, whereas if the tie spacing is more compact it stays on. Also, the diameter of the ties, especially in the case of columns, should not be too small.

The falling-off of the covering is also favoured by a narrowly spaced supporting reinforcement. The great differences of stress occurring in the concrete in a fire are especially critical at points where the concrete cross-section is reduced, i.e. at and between the steel reinforcements. The observance of the minimum distances between reinforcement rods laid down in steel-reinforced concrete standards is therefore of special importance from a fireproofing point of view.

It has been found important, especially in older buildings, to have adequate thrust reinforcement and adequate anchoring of the reinforcements, since otherwise the considerable deformation due to the high temperature

stresses cannot be absorbed.

Furthermore, the bearing capacity of concrete constructions in fires in a large number of cases has been found in practice to be greater than in the fire test. This applies particularly to constructions which were tested as statistically determinate bearing structures, but which in practice are statically indeterminate. This is the case for numerous prefabricated concrete constructions, especially prefabricated floors. The latter are almost always tested as monoaxially restrained. In practice, however, there is a certain lateral support as well. In fire tests, where in general free lateral movement is permitted, it has been found that slight lateral support or restraint produces a considerable increase in the fire-resistance time.

One effect which is not covered at all in fire tests is the size of the building. In large buildings it has been observed after fires that such large mutual displacements of entire sections of a structure took place as a result of great changes of shape, that the individual structural elements were no longer able to absorb these displacements[68]. For this reason, in all buildings exposed to the danger of fire there must be an adequate disposition of expansion joints.

To sum up it may be stated that for concrete, steel-reinforced concrete and prestressed concrete constructions there is good agreement between behaviour in a fire and behaviour in a fire test.

Failure to take into account generally benign construction principles which are of great importance in the case of a fire, has an unfavourable result. More favourable in many cases, on the other hand, are certain concealed static reserves which do not show up in the test.

8. Summary

Concrete and reinforced concrete construction has existed for over a century. Throughout this period it has been found over and over again during fires that concrete is a building material that is outstandingly resistant to fire and is, in this respect, rivalled by hardly any other material. It has the advantage over combustile materials of being incombustible, and it compares favourably with metals by virtue of its relatively low thermal conductivity, its large mass and its high specific heat.

Despite these favourable conditions, concrete and reinforced concrete structures are not able to resist fire indefinitely either. During a fire

the structural members affected are gradually heated through and through. Depending on the dimensions of a member, this heating process proceeds at a higher or lower rate. With progressive heating, changes occur in the material properties of the concrete and reinforcing steel. In addition, the distribution of forces within the cross-section and sometimes also the stability conditions are affected. Since the penetration of the heat occurs more slowly in members having larger cross-sectional dimensions, these dimensions are, generally speaking, the most important factor affecting the behaviour of concrete members in fire. Furthermore, in the case of reinforced structures, the fire resistance is very largely dependent upon the protection of the reinforcement against excessive heating. The thermal insulation afforded by the concrete cover to the steel can be improved by plastering or by special selection of the aggregates employed.

To summarise, it can be said that it is nowadays possible, by means of suitable technological and constructional precautions, to build concrete, reinforced concrete and prestressed concrete structures for any desired fire-resistance period within the limits encountered in actual practice. By taking advantage of the knowledge gained in fire tests and in fire damage to buildings it is possible also to obtain considerable fire-resistance periods in precast concrete construction with the slender members favoured by that method of construction.

The great spread of concrete and reinforced concrete construction over the past hundred years has occurred not least because - in addition to other advantages - these materials are superior to other building materials in the event of fire. It can safely be assumed that in the future, too, this superiority will be retained with respect to new building materials and methods.

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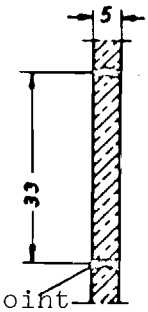
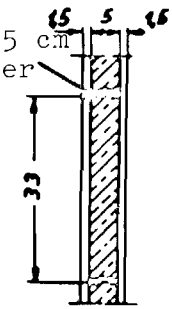
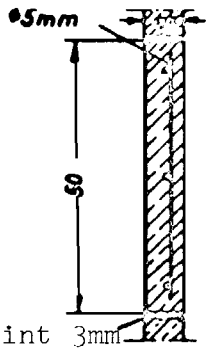
Table I

Test conditions for the measurement of thermal expansion of cement paste

Author	Storage before test	Dimensions of specimen	Water-cement ratio	Age at time of test
Endell	7 days in water then air	Prism 18 x 18 x 100 mm	0.26	28/50 days
Nekrassow	wet*	cylinder Ø 16 mm; L = 100 mm	—	28 days
Philleo	water	prism 51 x 70 x 152 mm	0.40	28 days

* Testing at 110°C, prior to the drying.

Table II

No.	Ref.	Wall cross-section dimensions, cm	Material Bulk density R (air dry) Compress.str. σ Dr Moist.cont. F	Temp. inc. on side away from fire Test time (h)	△T in °C Mean	Max
1	2	3	4	5	6	7
1	36	 1 cm joint lime cement mortar	Pumice cement boards 100 x 33 x 5 cm R~650 kg/m³ σ Dr not tested F = 12.7% by wt.	1/2 1 1 1/2 1 1/2	38 44 60 test finished	54 54 74
2	36	 1.5 cm plaster	Pumice cem. boards plastered R~650 kg/cm² σ Dr not tested F = 12.7% by wt.	1/2 1 1 1/2 1 1/2	21 29 45 test finished	24 34 54
3	22	 joint 3mm cemented	aerated conc. slabs 120 x 50 x 7,5cm R = 730 kg/m³ σ Dr = 35 kg/cm² F = 26% by wt.	1/2 1 1 1/2 1 1/2	34 54 57 test finished	40 *) *)

*) inadequate formation of joint

Table II Continued

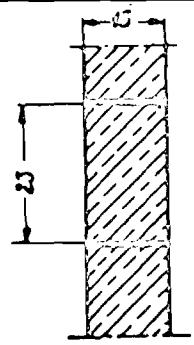
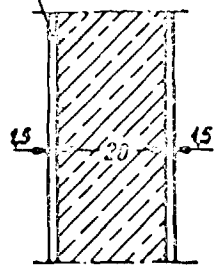
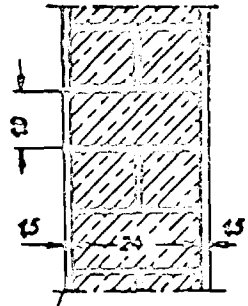
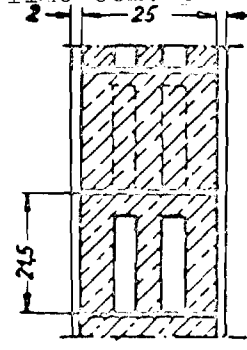
4	22		aerated conc. blocks 50 x 25 x 7.5 cm R = 550 kg/m³ σ Dr = 25 kg/cm² F = 4.2% by wt.	1/2 1 1 1/2 2 3	- 6 35 48 test fin-ished	- 10 44 53
5	35	lime cement plaster 	no fines conc. crushed clay brick 7/15 mm R = 1330 kg/m³ σ Dr = 46 kg/cm² F not tested	1/2 1 1 1/2 1 1/2	- 8 36 test fin-ished	2 20 45
6	35	 lime cement mortar	solid slag conc. blocks 24 x 12 x 10 cm R = 1310 kg/m³ σ Dr = 66 kg/cm² F not deter.	1/2 1 1 1/2 2 3 3	- - 6 21 42 test finished	- - 9 36 70
7	35	lime cem. plaster 	hollow slag conc. blocks 50 x 25 x 21.5 cm R = 960 kg/m³ σ Dr = 45 kg/cm² F not deter.	1/2 1 1 1/2 2 3 3	- 1 15 39 50 test fin-ished	- 1 16 47 54

Table II Continued

8	34	2 layers of plaster 22,5	heavy conc., 2 compartment hollow blocks R-not known σ Dr - not known F stone = 3.5% by wt F plaster=6.1% by wt	1/2 1 1^h:50 1^h:50'	- 26 58 Cracks in walls	- - -
9	33	10 Z=cement W/Z=water/cement Z ~ 290 kg/m³ W/Z ~ 0,63	heavy concrete limestone aggregate R = 2350 kg/m³ σ Dr = 417 kg/cm² F not tested	1/2 1 1 1/2 2 2	~60 ~110 ~160 ~230 test finished	- - - -
10	33	~15 Z ~ 290 kg/m³ W/Z ~ 0,63	heavy concrete quartizitic sand and gravel R = 2240 kg/m³ σ Dr = 310 kg/cm² F not tested	1/2 1 1 1/2 2 2 1/2	~15 ~80 ~85 ~100 test finished	- - - -
11	33	~20 Z ~ 290 kg/m³ W/Z ~ 0,63	heavy concrete quartzitic sand and gravel R = 2200 kg/m³ σ Dr = 310 kg/cm² F not tested	1/2 1 2 3 ~5	- ~44 ~71 ~80 test finished	- - - -

Table III

Increase of fire-resistance time of massive concrete slabs owing to design modifications

Modification	Additional time before collapse in %
Approximately 30% reduction of steel stress	approx. 40%
Increase of steel covering from 1 to 2 cm	10 to 30%
1.5 cm plaster of strongly adhering lime cement mortar	50 to 100%
Special plasters with vermiculite, perlite or sprayed asbestos base applied underneath	200 to 1000%
Increase of concrete grade from B 120 to B 225	5 to 30%
Increase of floor thickness from 10 to 18 cm	approximately 45%

Table IV

No.	Ref.	Floor design Dimensions in cm	Span l (m) bearing bending moment at mid-bay M (mkg/m)	Behaviour in fire test: Test time in h and min	Deflection at mid-bay	Mean temp. °C: At reinforcement	Mean temp. °C: At surface opposite fire
1	2	3	4	5	6	7	8
1	34	Heavy conc. massive slab; ø 3/8"; ~11; ~9; ~9; ~1,2; steel covering Conc. str.: W_{36} = 260 kg/cm²	ℓ ~ 3,65m simply supported M ~ 610mkg	1/2 1h 1h : 15m 1h : 22m	- <15 = collapse	305 475 545	48 90 94
2	38	Heavy conc. massive slab; 12; 1,25; 15; 15; 1,0; steel covering Gypsum board slab with fibre-glass added Conc. str.: W_{28} = 528 kg/cm²	ℓ ~ 2,70m simply supported M ~ mkg	1/2h 1h 1 1/2h	0,35 0,65 1,05 Fire test concluded	70 133 205	7 23 63

Table IV Continued

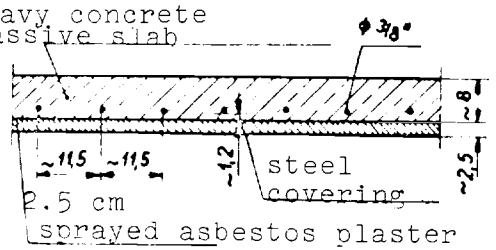
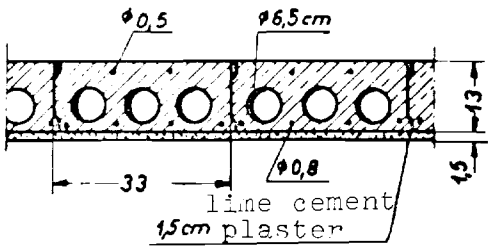
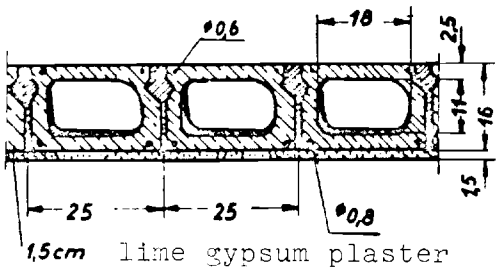
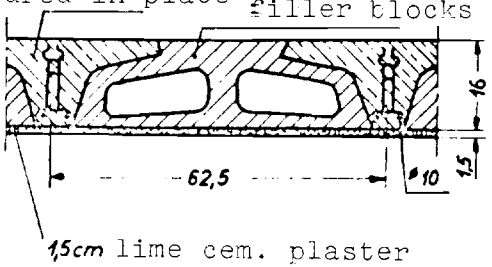
3	34	 Conc. str.: W_{80}: 280 kg/cm²	ℓ = 3,98 m fixed all sides crossed reinforcement M~270 mkg (Peldmoment)	1/2 1 2 4	- - - 1	34 53 69 125	15 29 50 91
				Fire test concluded			
4	37		ℓ = 4,00 m simply supported M~960 mkg	1/2 1 1 1/2	4,05 9,15 13,7	- - -	28 63 82
				Fire test concluded			
5	39		ℓ = 4,00 m simply supported M~1250 mkg	1/2 1 1 1/2	3,1 5,2 6,4	213 354 483	35 73 104
				Fire test concluded			
6	37		ℓ = 4,00 m simply supported M=unknown	1/2 1 1 1/2	1,0 3,6 5,8	155 296 392	29 45 55
				Fire test concluded			

Table IV Continued

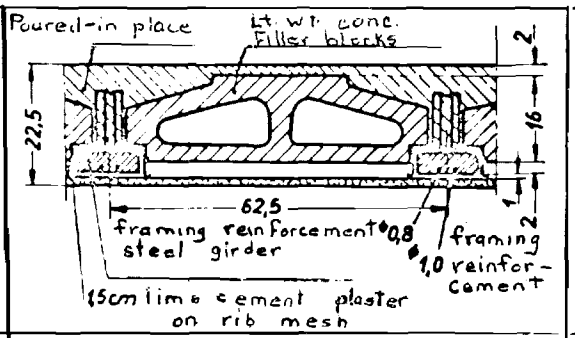
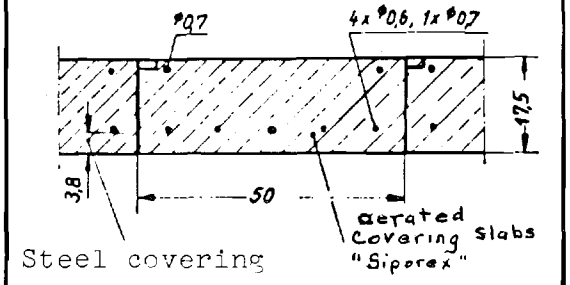
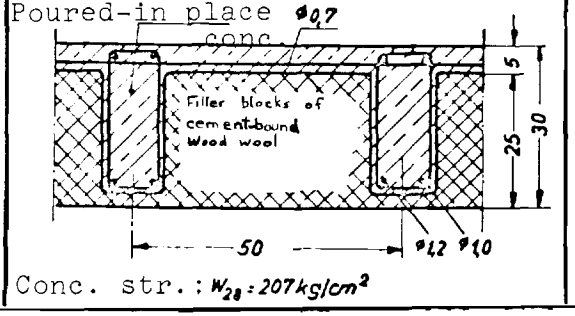
7	40		ℓ = 4,00 m simply supported M = 1280 mkg	1/2 1 1 1/2	1,1 1,4 2,1	53 98 124	11 27 44
8	22		ℓ =3,85 m simply supported M=unknown	1/2 1 1 1/2	1,75 3,9 7,75 Fire test concluded	- - -	- 26 46
9	41		ℓ = 3,85 m simply supported M = 2120 mkg	1 1/2 1 1 1/2 3	0,5 0,9 1,3 3,3 Fire test concluded	47 94 146 365	- - 13 30

Table V

Influence of aggregates on the fire resistance time of steel-reinforced concrete beams after ref. (28)

No.	Kind of aggregate	Fire-resistance time hours : minutes
1	Alluvial sand and gravel	3h : 03m
2	like 1, but about 6 cm crushed clay brick concrete on bottom of beam	3h : 32m
3	Leighton Buzzard sand + Torphin Whinstone (limestone)	more than 4h (test discontinued)

Table VI

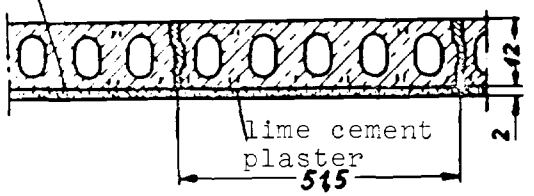
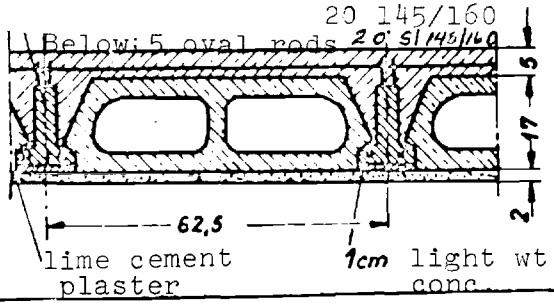
No.	Ref.	Floor design dimensions in cm	Span l (m) Bearing Bending moment at mid-bay M (mkg/m)	Behaviour in fire test: Test time in h	Deflection at mid-bay	Mean temp. in °C: at reinforcement	at side opposite fire
1	37	Reinforcement above: 4 twisted wires 2x25mm S1 150 H8/ below: 9 steel rods 3x8mm sl 145/160 	l = 4,00m	1/2	2,25cm	--	51
			simply supported	1	7,5 cm	--	72
			M = 1640mkg/m	1 1/2	15,0 cm	--	70
2	37	Reinforcement above: 1 oval rod 20 145/160 Below: 5 oval rods 20 S1 145/160 	l = 4,00m	1/2	0,9 cm	(93)	13
			simply supported	1	1,7 cm	(103)	33
			M =	1 1/2	2,3 cm	(172)	47

Table VII

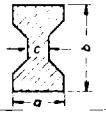
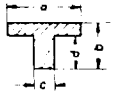
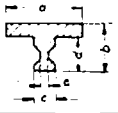
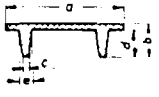
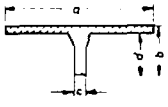
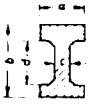
Nc	Ref.		Shape of cross-dimensions in cm				
			a	b	c	d	e
1	47 56 60		10 to 20	15 to 36	-	-	-
2	47 58 59		24 to 60	40 to 119	8 to 20,5	-	-
3	56		25,4 to 122	12,5 to 78,2	6,3 to 30,4	11,4 to 60	-
4	56		25,4 to 50,8	12,5 to 30,4	6,3 to 12,7	11,4 to 22,8	2,6 to 5,2
5	58		120 140	35 18	6 11,5	30 14	11 14
6	58		150 150 210	50 40 45	11,2 11,2 20	42 32 27,5	
7	63		15	24	50	14,5	
8	63		15	24	50	18,5	

Table VIII

Minimum prestressed reinforcement covering values after ref. (41)

Desired fire-resistance time hours	Required minimum reiforcement covering in cm for beam cross-sections of				
	100-200	200-500	500-1000	1000-2000	2000
1/2	3	2	2	2	2
1	-	5	4	3	3
1 1/2	-	-	6	4	3
2	-	-	-	6	5
3	-	-	-	-	8
4	-	-	-	-	10

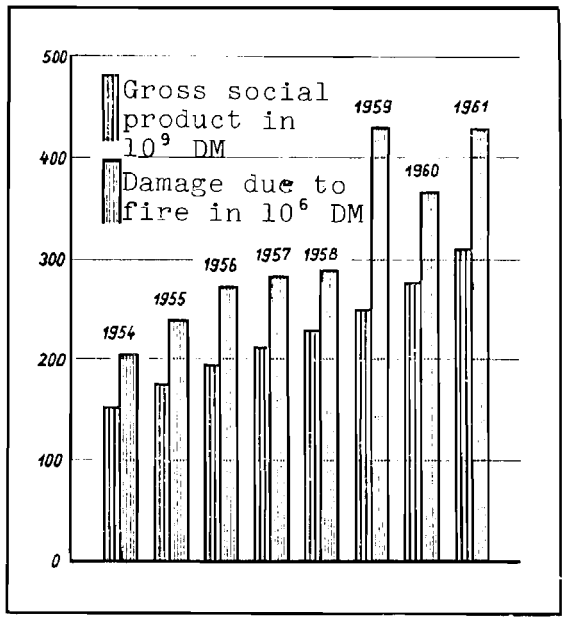


Bild 1. Brandschadensverlauf in der Bundesrepublik Deutschland in den Jahren 1954—1961

Fig. 1. Diagram showing fire damage in the German Federal Republic in the years 1954—1961.

Fig. 1. Dégâts par incendies dans la République Allemande durant les années 1954—1961

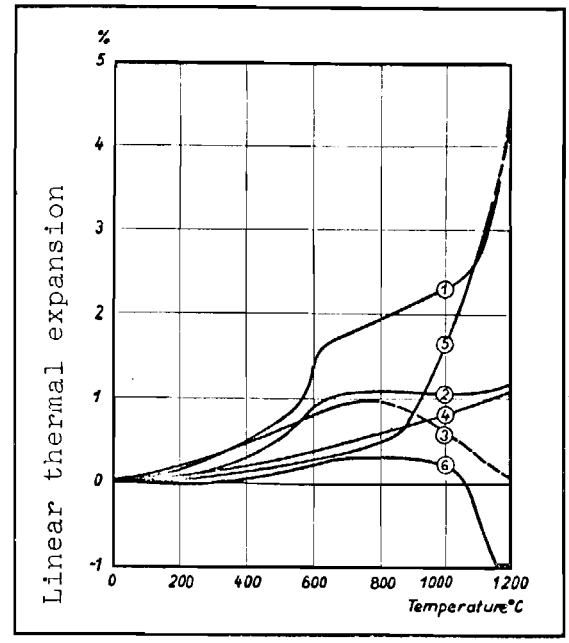


Bild 2. Lineare Wärmeausdehnung verschiedener Zuschlag-Gesteine nach (5). 1. Grauwacke, 2. Sandstein, 3. Kalkstein, 4. Hochofenstückschlacke, 5. Basalt, 6. Ziegelbruch
— — — — — Unsicherer Verlauf infolge Gasabgabe

Fig. 2. Linear thermal expansion of various aggregates according to (5): 1. greywacke; 2. sandstone; 3. limestone; 4. crushed blast-furnace slag; 5. basalt; 6. broken brick.
— — — — — uncertain due to evolution of gas

Fig. 2. Dilatation linéaire par la chaleur de divers agrégats selon (5). 1. Agglomérés, 2. Grès, 3. Calcaire, 4. Laitier de haut fourneau en morceaux, 5. Basalte, 6. Tuiles concassées
— — — — — Détermination incertaine à cause du dégagement des gaz

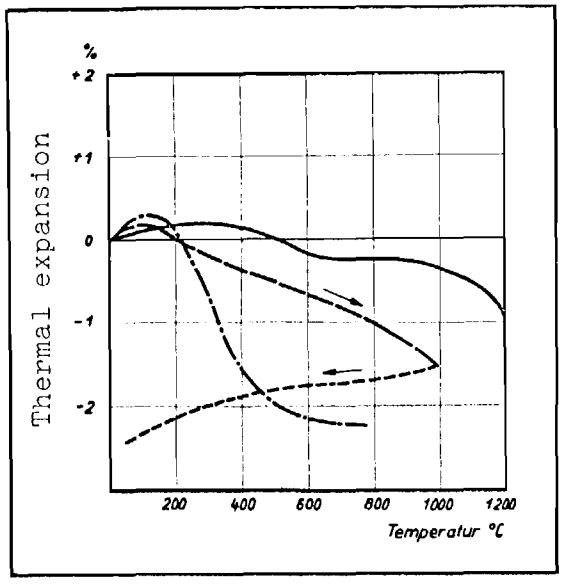


Bild 3. Lineare Wärmeausdehnung von Zementstein (Portlandzement) bei erstmaliger Erwärmung
——————— nach Endell (5)
— — — — — — Abkühlung } noch Nekrassow (6)
—— —— —— Erwärmung }
—— . —— . —— Philleo (7)

Fig. 3. Linear thermal expansion of hardened cement paste (Portland cement) on being heated for the first time.
——————— according to Endell (5)
— — — — — — cooling } according to Nekrassow (6)
—— —— —— heating }
—— . —— . —— Philleo (7)

Fig. 3. Dilatation linéaire par la chaleur du ciment durci (ciment Portland) chauffé pour la première fois
——————— selon Endell (5)
— — — — — — Refroidissement } Selon Nekrassow (6)
—— —— —— Echauffement }
—— · —— · —— Philleo (7)

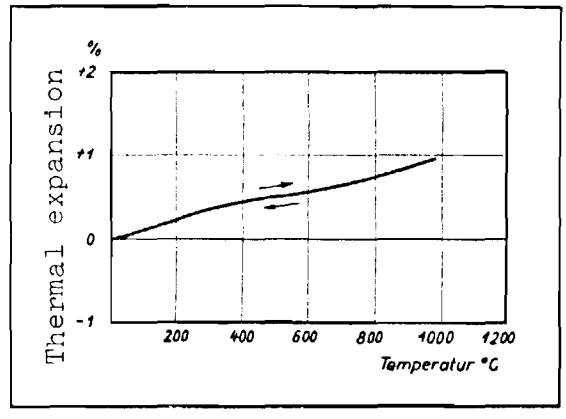


Bild 4. Lineare Wärmeausdehnung von Zementstein bei der zweiten Erhitzung (Erwärmung und Abkühlung) nach Nekrassow (6)

Fig. 4. Linear thermal expansion of hardened cement paste on being heated for the second time (heating and cooling) according to Nekrassow (6).

Fig. 4. Dilatation linéaire par la chaleur du ciment durci, chauffé pour une seconde fois (échauffement et refroidissement) selon Nerkassow (6)

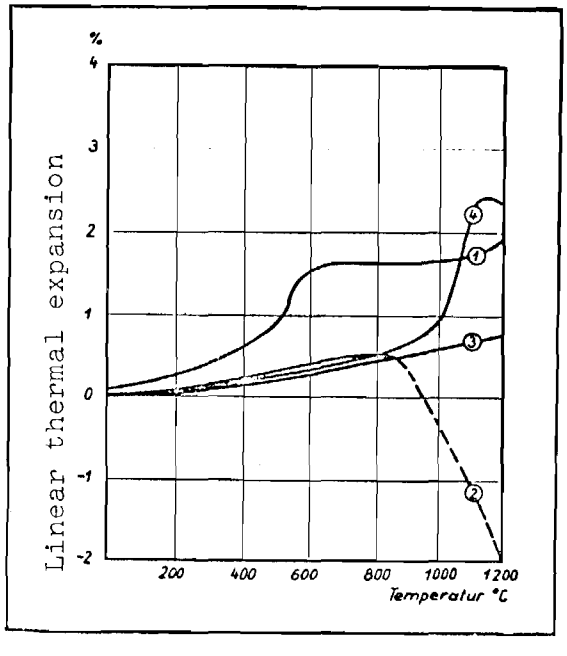


Bild 5. Lineare Wärmeausdehnung von Mörteln aus Portlandzement und verschiedenen Zuschlagstoffen nach (5)
Mischungsverhältnis 1 : 3 : 0,67 nach Gewichtsteilen
1. Rheinkiesel, 2. Kalkstein, 3. Hochofenstückschlacke, 4. Basalt
— — — — — — Unsicherer Verlauf infolge Abgabe von CO_2

Fig. 5. Linear thermal expansion of motars made with Portland cement and various aggregates according to (5)
Mix proportions 1 : 3 : 0.67 by weight
1. Rhine gravel; 2. limestone; 3. lump blast-furnace slag; 4. basalt
— — — — — — uncertain due to evolution of CO_2

Fig. 5. Dilatation linéaire par la chaleur de mortiers en ciment Portland avec divers agrégats selon (5)
Proportion du mélange 1 : 3 : 0,67 en poids.
1. Gravier du Rhin, 2. Calcaire, 3. Laitier de haut fourneau en morceaux, 4. Basalte
— — — — — — Détermination incertaine à cause du dégagement du CO_2

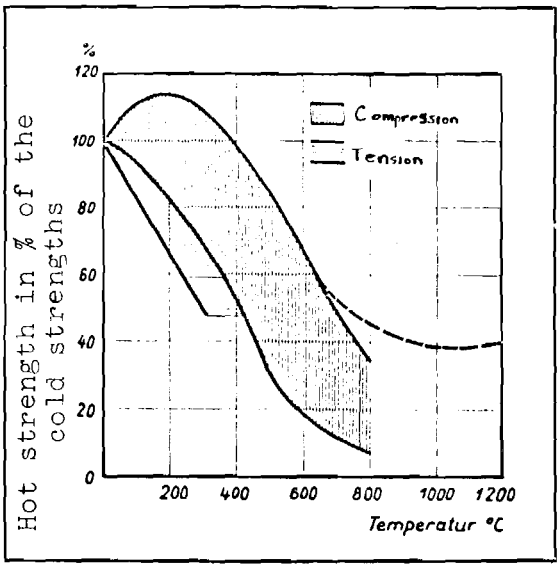


Bild 6. Druckfestigkeit von Beton in Abhängigkeit von der Erhitzungstemperatur

Fig. 6. Compressive strength of concrete as a function of the heating temperature

Fig. 6. Résistance à la compression du béton en fonction de la température d'échauffement

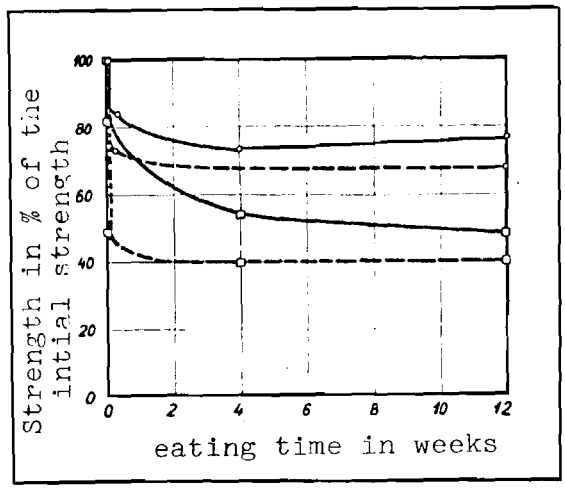


Bild 7. Druckfestigkeit im Beton nach mehrwöchiger Temperaturbeanspruchung. Mischungsverhältnis 1 : 6 n. Gew.-Teilen
Ausgangsdruckfestigkeit 246/kg/cm²
——— Prüfung in heißem Zustand
– – – – – Prüfung nach vorheriger Abkühlung
○ = Erhitzung auf 100° C
□ = Erhitzung auf 300° C

Fig. 7. Compressive strength of concrete after being subjected to elevated temperature for several weeks.
Mix proportions 1 : 6 by weight.
Initial compressive strength 246 kg/cm².
——— tested in hot condition
– – – – – tested after cooling
○ = heating to 100° C
□ = heating to 300° C

Fig. 7. Résistance à la compression du béton après soumission à une température élevée pendant quelques semaines. Mélange 1 : 6 en poids. Résistance initiale à la compression 246 kg/cm²
——— Essai à l'état chaud
– – – – – Essai après refroidissement
○ = Echauffement à 100° C
□ = Echauffement à 300° C

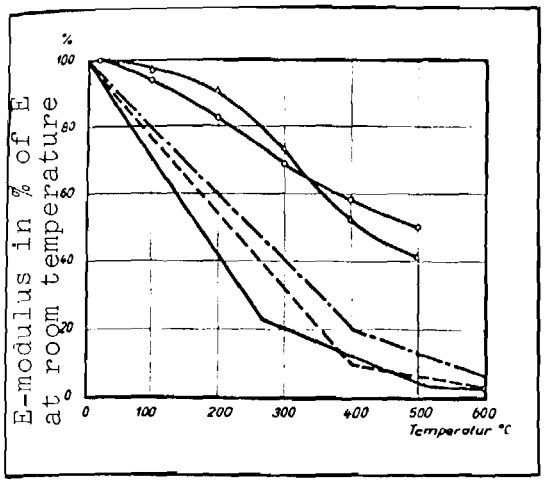


Bild 8. Einfluß höherer Temperaturen auf den E-Modul von Beton

○——○	W/Z = 0,40	E_{kalt} 400 000 kg/cm²	(19)
△——△	W/Z = 0,60	E_{kalt} 340 000 kg/cm²	(19)
– – – – –	Kalkstein-Beton	E_{kalt} 240 000 kg/cm²	(18)
———	Diabas-Beton	E_{kalt} 250 000 kg/cm²	(18)
—·—·—	Sandstein-Beton	E_{kalt} 290 000 kg/cm²	(19)

Fig. 8. Effect of elevated temperatures upon the modulus of elasticity (E) of concrete

○——○	W/C = 0.40	E_{cold} 400,000 kg/cm²
△——△	W/C = 0.60	E_{cold} 340,000 kg/cm²
– – – – –	limestone concrete	E_{cold} 240,000 kg/cm²
———	diabase concrete	E_{cold} 250,000 kg/cm²
—·—·—	sandstone concrete	E_{cold} 290,000 kg/cm²

Fig. 8. Influences de températures élevées sur le module E du béton.
W/Z = eau/ciment $E_{kalt} = E_{\text{à froid}}$

○——○	W/Z = 0,40	$E_{\text{à froid}}$ 400 000 kg/cm²
△——△	W/Z = 0,60	$E_{\text{à froid}}$ 340 000 kg/cm²
– – – – –	Béton avec calcaire	$E_{\text{à froid}}$ 240 000 kg/cm²
———	Béton avec diabas	$E_{\text{à froid}}$ 250 000 kg/cm²
—·—·—	Béton avec grès	$E_{\text{à froid}}$ 290 000 kg/cm²

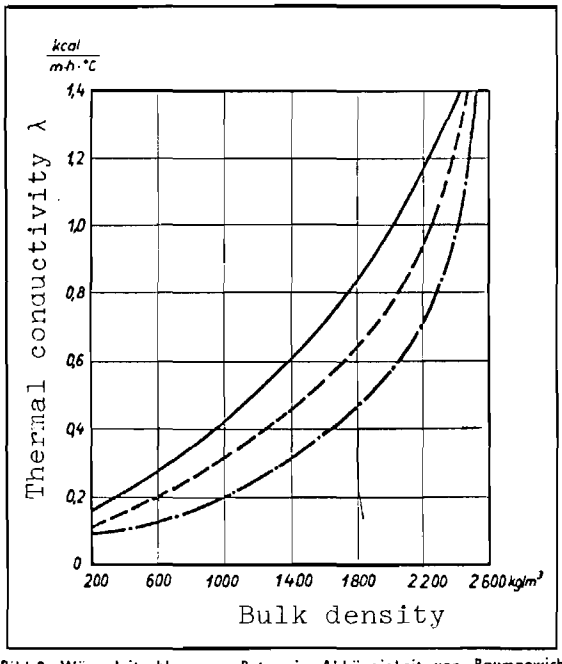


Bild 9. Wärmeleitzahlen von Beton in Abhängigkeit von Raumgewicht und Feuchtigkeitsgehalt bei Raumtemperatur nach (20)
——————— feucht
— — — — — lufttrocken
—— . —— . —— künstlich getrocknet

Fig. 9. Thermal conductivity values of concrete as a function of bulk density and moisture content at room temperature, according to (20)
——————— moist
— — — — — air dry
—— . —— . —— artificially dried

Fig. 9. Coefficient de transmission de la chaleur du béton en fonction de la densité et de la teneur en humidité à la température ambiante selon (20)
——————— humide
— — — — — sèche à l'air
—— . —— . —— sèche artificiellement

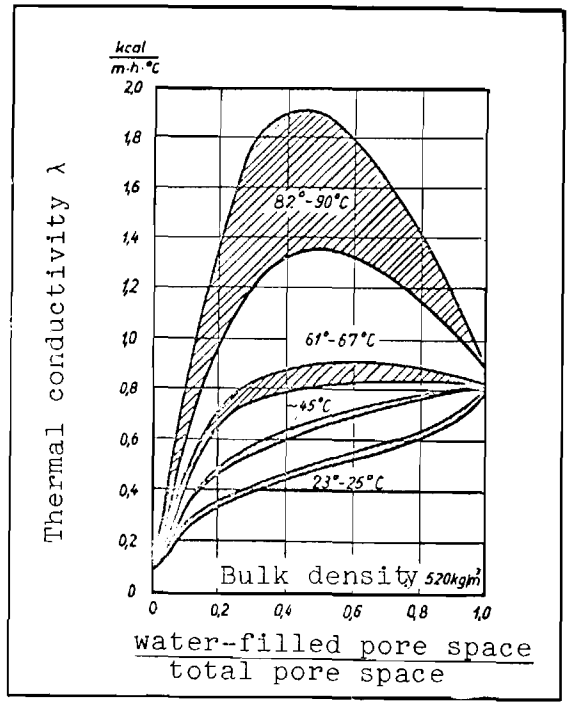


Bild 10. Wärmeleitfähigkeit λ eines Gasbetons in Abhängigkeit von Feuchtigkeit und Temperatur nach (21)

Fig. 10. Thermal conductivity λ of an aerated concrete as a function of humidity and temperature.

Fig. 10. Transmission de la chaleur λ d'un béton-gaz en fonction de l'humidité et de la température.

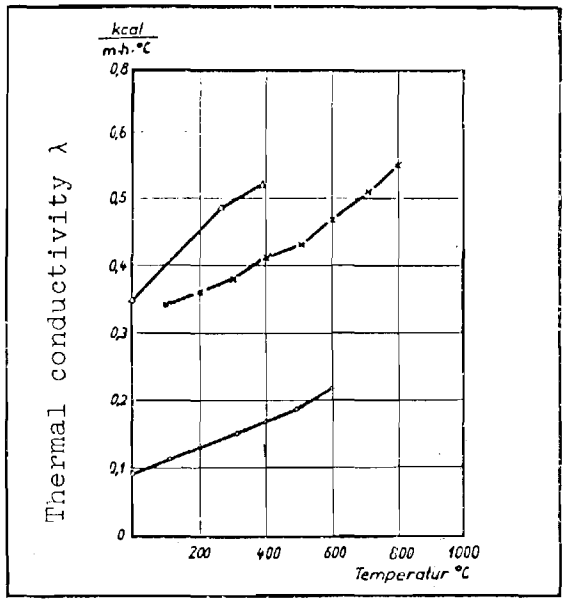


Bild 11. Wärmeleitzahl von Beton in Abhängigkeit von der Temperatur
○———○ Gasbeton R = 506 kg/m³ (22)
△———△ Schamotte-Beton R = 1670 kg/m³ (6)
×———× Hüttenbimsbeton R = 1153 kg/m³ (23)

Fig. 11. Thermal conductivity of concrete as a function of temperature.
○———○ aerated concrete R = 506 kg/m³ (22)
△———△ fire-clay concrete R = 1670 kg/m³ (6)
×———× foamed slag concrete R = 1153 kg/m³ (23)

Fig. 11. Coefficient de transmission de la chaleur du béton en fonction de la température
○———○ Béton-gaz R = 506 kg/m³ (22)
△———△ Béton avec Chamotte R = 1670 kg/m³ (6)
×———× Béton avec ponce de laitier R = 1153 kg/m³ (23)

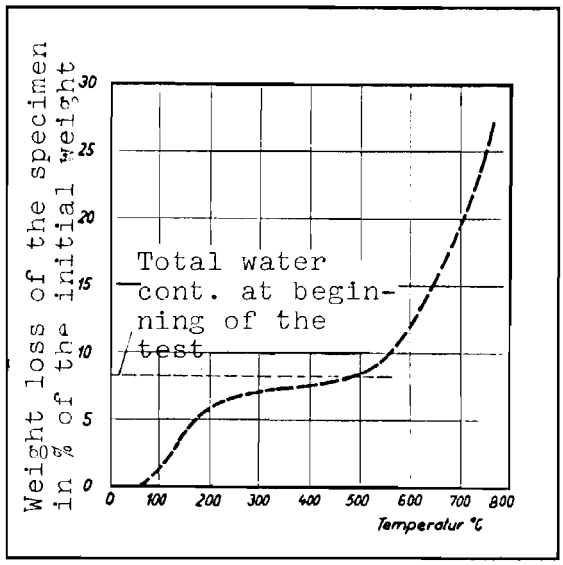


Bild 12. Gewichtsverlust von Beton aus kalkhaltigem Sand und Kies bei Erhitzung
Zementgehalt Z = 440 kg/m³
Wasserzementwert W/Z = 0,40

Fig. 12. Loss of weight of concrete made with calcareous sand gravel on heating.
Cement content C = 440 kg/m³
Water/cement ratio W/C = 0.40

Fig. 12. Perte de poids du béton avec sable et gravier calcaire lors de l'échauffement.
teneur en ciment Z = 440 kg/m³
eau/ciment W/Z = [illegible]

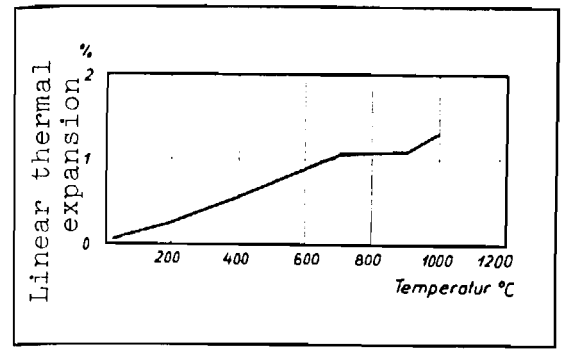


Bild 13. Lineare Wärmedehnung eines Stahles mit 0,4% C (24)
Fig. 13. Linear thermal expansion of a steel containing 0.4% C (24)
Fig. 13. Dilatation linéaire d'un acier avec 0.4% C (24)

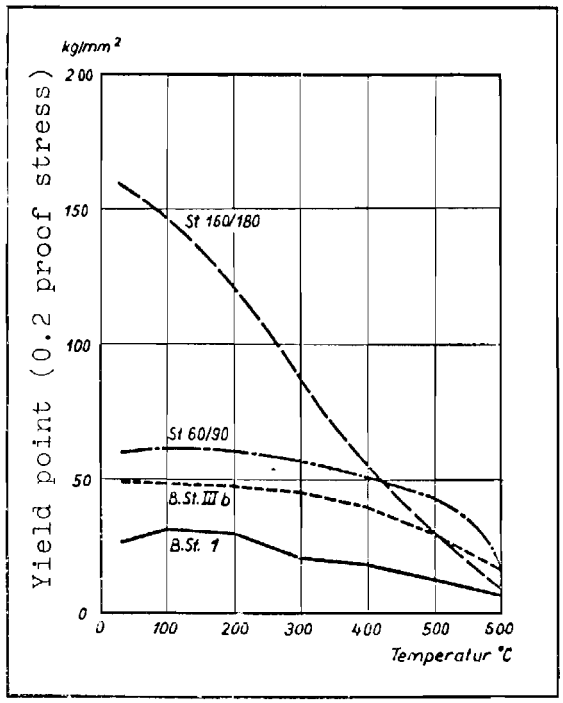


Bild 14. Warmstreckgrenze (0,2 Dehngrenze) deutscher Beton- und Spannstähle
——————— B St I Φ 8 mm
- - - - - - - - - - - - Torstahl Φ 8 mm
— — — St 160/180 Φ 5 mm, kaltgezogen
— . — . — . — St 60/90 Φ 26 mm, warmgewalzt

Fig. 14. Hot yield point (0.2% proof stress) of German reinforcing steels and prestressing steels.
——————— Structural steel B St I, 8 mm dia.
- - - - - - - - - - - - Tor steel, 8 mm dia.
— — — St 160/180, 5 mm dia., cold-drawn
— . — . — . — St 60/90, 16 mm dia., hot-rolled

Fig. 14. Limite de ductilite à chaud (0,2 limite d'extensibilité) d'aciers allemands pour béton et béton précontraints.
——————— B St I ø 8 mm
- - - - - - - - - - - - acier Tor ø 8 mm
— — — St 160/180 ø 5 mm, étiré à froid
— . — . — . — St 60/90 ø 26 mm, lamnié à chaud

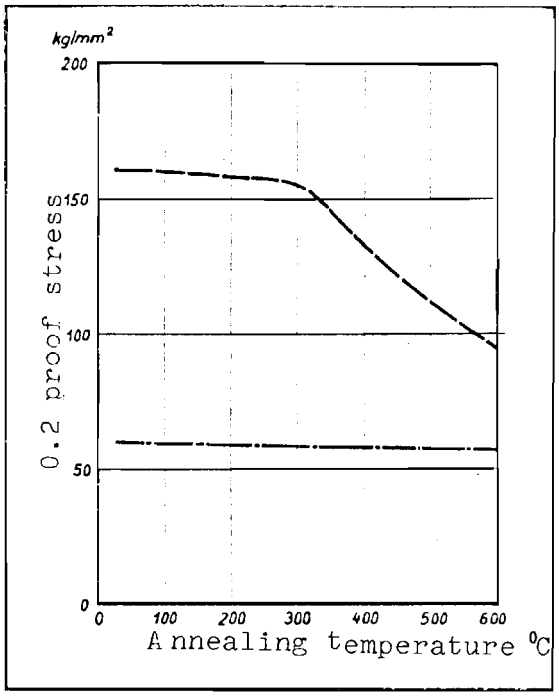


Bild 15. 0,2-Dehnprobe deutscher Spannstähle bei 20° C nach vorherigem Glühen (26)
—.—.— Warmgewalzter Stabstahl St 60/90 von 5,2 mm Φ
— — — Kaltgezogener Spanndraht St 160/180 von 5 mm Φ

Fig. 15. 0.2% proof test for German prestressing steels at 20° C after previous annealing (26).
—.—.— Hot-rolled bar, St 60/90, 5.2 mm dia.
— — — Cold-drawn prestressing wire, St 160/180, 5 mm dia.

Fig. 15. 0,2 essai à l'extensibilité d'aciers allemands pour béton précontraints à 20° C après l'échauffement à rouge (26)
—.—.— acier en baguette laminé à chaud St 60/90 de 5,2 mm ø
— — — Fil en acier étiré à froid pour béton précontraint St 160/180 de 5 mm ø

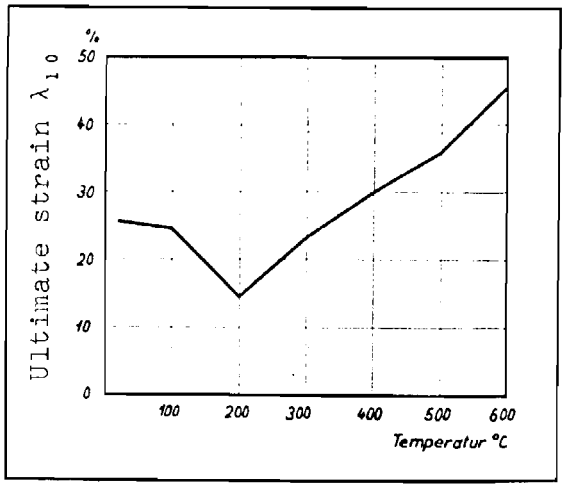


Bild 16. Bruchdehnung eines Flußstahles in Abhängigkeit von der Temperatur

Fig. 16. Ultimate strain of a mild steel as a function of temperature.

Fig. 16. Extensibilité jusqu' à la rupture d'un acier coulé en fonction de la température.

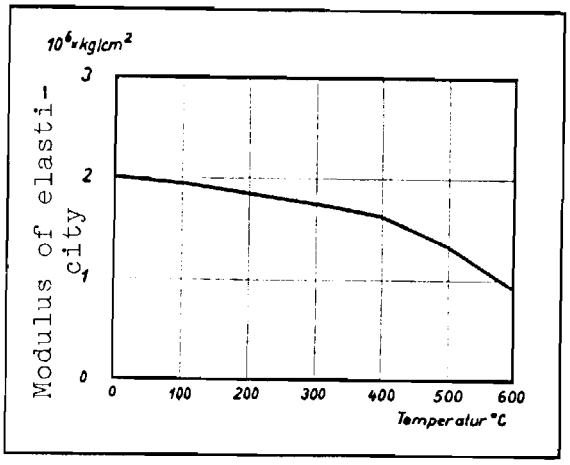


Bild 17. E-Modul von unsiliziertem Flußstahl in Abhängigkeit von der Temperatur

Fig. 17. Modulus of elasticity (E) of non-siliceous mild steel as a function of temperature.

Fig. 17. Module E d'un acier coulé sans silice en fonction de la température

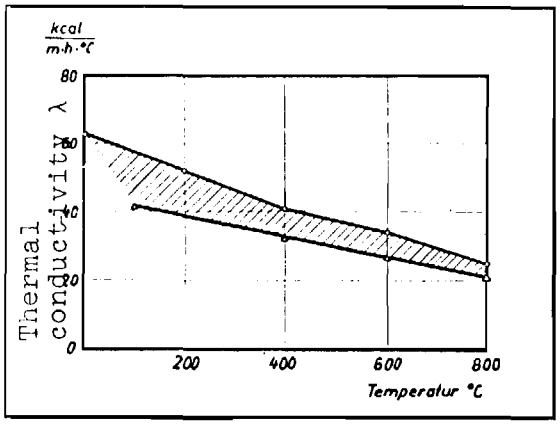


Bild 18. Wärmeleitzahl von Stählen in Abhängigkeit von der Temperatur
○———○ reines Eisen (18)
△———△ 0,8% C (22)

Fig. 18. Thermal conductivity of steels as a function of temperature.
○———○ Pure iron (18)
△———△ 0.8% C (22)

Fig. 18. Coefficient de transmission de la chaleur d'aciers en fonction de la température
○———○ Fer pur (18)
△———△ 0,8% C (22)

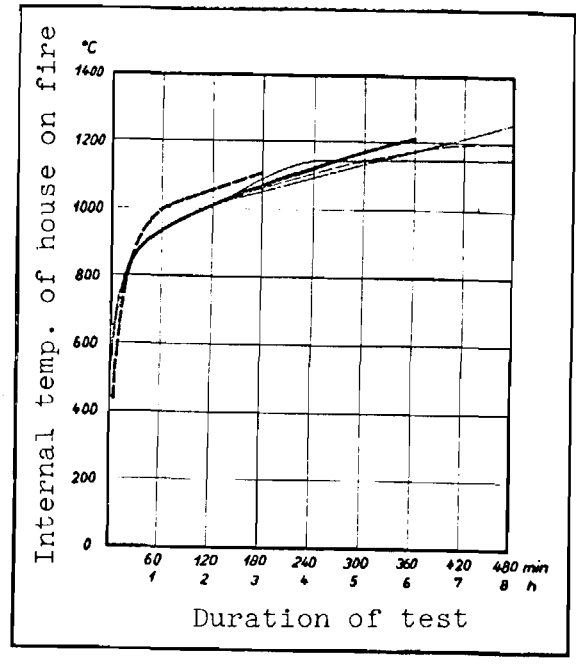


Bild 19. Temperaturzeitkurven verschiedener Länder
— — — Deutschland (*)
———— Niederlande, Frankreich, Großbritannien, ISO (Vorschlag)
— . — . — . — USA
— — — Schweiz
———— Schweden
(*) Die deutsche Einheitstemperaturzeitkurve wird demnächst der ISO-Kurve angepaßt

Fig. 19. Time-temperature curves of various countries.
— — — Germany (*)
———— Holland, France, Great Britain, ISO (proposal)
— . — . — . — USA
— — — Switzerland
———— Sweden
(*) The German standard time-temperature curve is soon to be adapted to the ISO curve

Fig. 19. Lignes de température-temps de divers pays
— — — Allemagne (*)
———— Pays Bas, France, Angleterre, ISO (Proposition),
— . — . — . — USA
— — — Suisse
———— Suède
(*) La ligne de température-temps allemande sera bientôt adaptée à la [illegible] de l'ISO

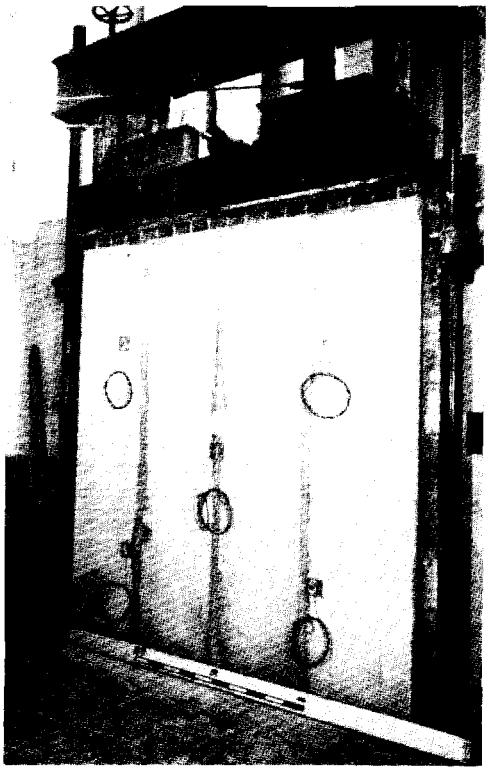


Bild 20. Zum Brandversuch vorbereitete Wand aus geschoßhohen Leichtbetonplatten.

Fig. 20. Wall of storey-high lightweight concrete slabs prepared for the fire test.

Fig. 20. Paroi d'essai pour la résistance au feu d'une hauteur d'un étage en plaques de béton léger

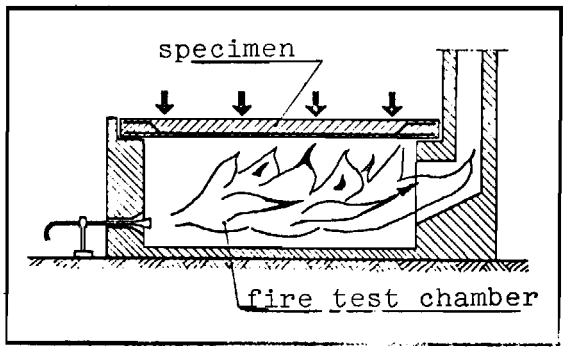


Bild 21. Brandkammer für Deckenprüfungen (Schema)

Fig. 21. Fire test chamber for the testing of floors (diagrammatic)

Fig. 21. Chambre pour l'essai aux plafonds

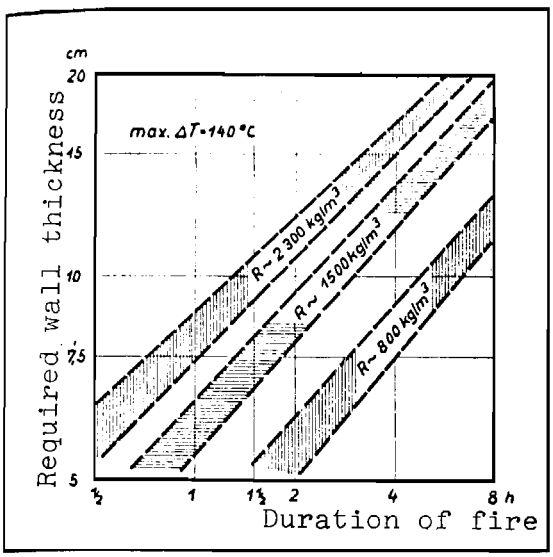


Bild 22. Erforderliche Dicke massiver Betonwände in Abhängigkeit von Branddauer und Raumgewicht R für eine maximale Temperaturerhöhung von 140° C auf der dem Feuer abgekehrten Seite

Fig. 22. Requisite thickness of solid concrete walls as a function of fire duration and bulk density R for a maximum temperature rise of 140° C on the side remote from the fire.

Fig. 22. Epaisseur nécessaire de parois massives en béton en fonction de la durée du feu et de la densité R pour une augmentation maximo de la température de 140° C du côté opposé au feu

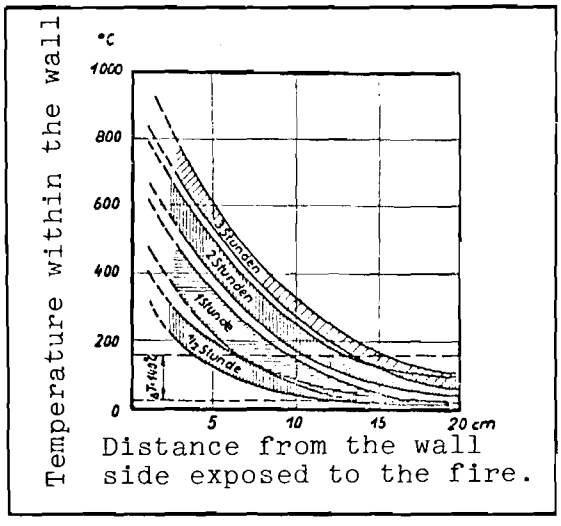


Bild 23. Temperaturverlauf in Schwerbetanwänden (R ~ 2300 kg/m³) verschiedener Dicke im Normenbrand nach ½, 1, 2 und 3 Stunden

Fig. 23. Temperature curve in dense concrete walls of various thicknesses, as obtained in standard fire after ½, 1, 2 and 3 hours

Fig. 23. Propagation de la température dans des parois à béton lourd de différentes épaisseures à l'essai ou feu normalisé après ½, 1, 2 et 3 heures

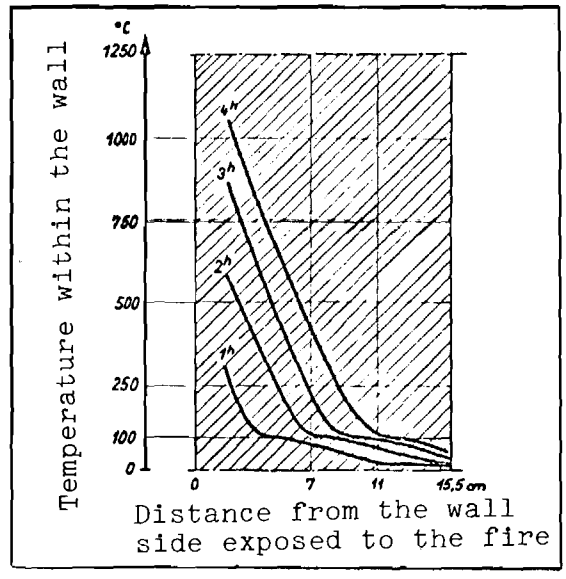


Bild 24. Temperaturverlauf in einer 15 cm dicken Gasbetonwand (R 900 kg/m³) im Normenbrandversuch. Feuchtigkeitsgehalt etwa 8 Prozent

Fig. 24. Temperature curve in a 15 cm thick aerated concrete wall (bulk density 900 kg/m³) in the standard fire test. Moisture content approx. 8%.

Fig. 24. Propagation de la température dans une paroi de 15 cm d'épaisseur en béton-gaz (R = 900 kg/m³) à l'essai ou feu normalisé, teneur en humidité environ 8%

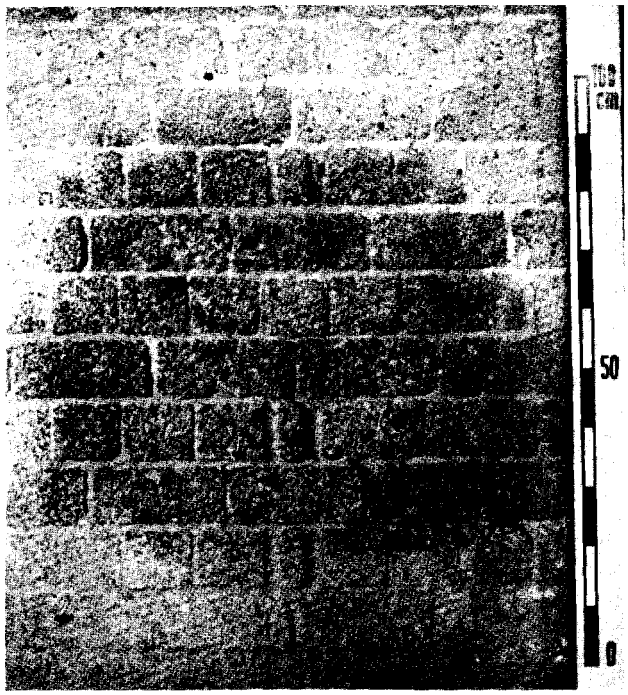


Bild 25. Wand aus Schlackenbetonvollsteinen nach einem Brandversuch von 3 Stunden Dauer

Fig. 25. Wall of solid slag concrete blocks after a fire test of 3 hours' duration

Fig. 25. Wall en blocs pleins en béton de laitier de haut fourneau après un essai au feu de 3 heures

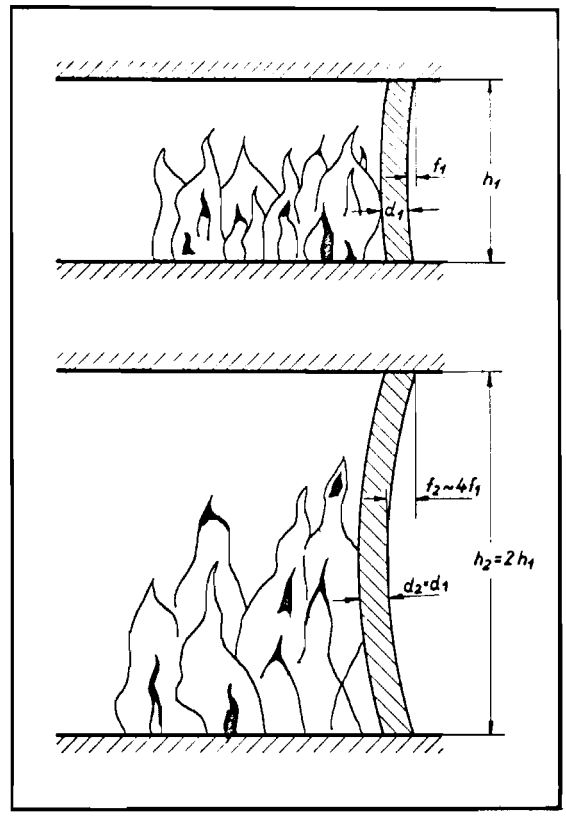


Bild 26. Abnahme der Standfestigkeit einer nicht eingespannten Wand mit der Bauhöhe h

Fig. 26. Reduction of the stability of a non-restrained wall with construction depth h.

Fig. 26. Diminution de la résistance d'une paroi non encadrée de la hauteur h.

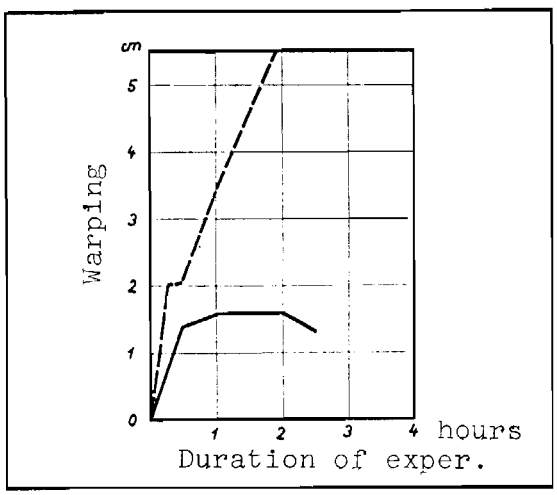


Bild 27. Verwölbung von Wänden im Brandversuch
——————— 15 cm dicke Schwerbetonwand 1,80 m hoch (33)
— — — 7,5 cm dicke Wand aus Leichtbetonhohlblocksteinen Wandhöhe 3 m (31)

Fig. 27. Warping of walls in the fire test
——————— 15 cm thick dense concrete wall, 1.80 m high (33)
— — — 7.5 cm thick wall of lightweight concrete hollow blocks; height of wall 3 m (31)

Fig. 27. Déformation de parois lors de l'essai au feu
——————— Paroi en béton lourd, épaisseur 15 cm, hauteur 1,80 m (33)
— — — Paroi en béton léger de blocs creux, épaisseur 7,5 cm, hauteur 3 m (31)

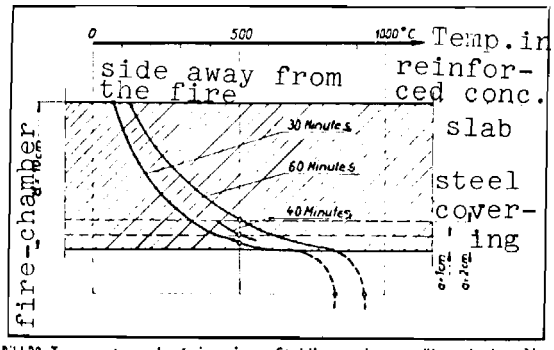


Bild 28. Temperaturverlauf in einer Stahlbetonplatte während des Normenbrandes

Fig. 28. Temperature curve in a reinforced concrete slab during the standard fire test

Fig. 28. Propagation de la température dans une plaque en béton armé durant l'essai au feu normalisé

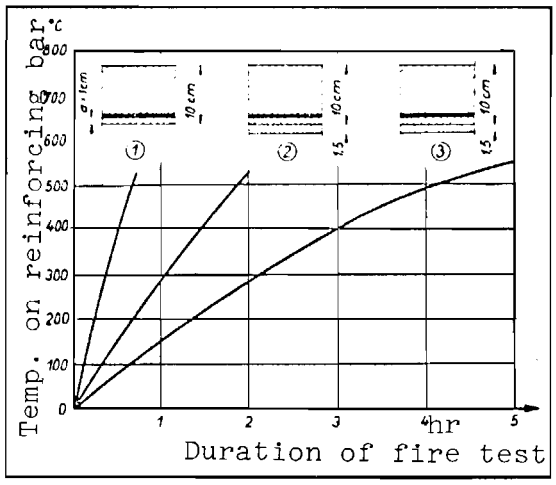


Bild 29. Ansteigen der Temperaturen an den Bewehrungsstählen von Stahlbetondecken ohne und mit Unterputz
(1) unverputzte Decken
(2) 1,5 cm Kalkzementputz
(3) 1,5 cm Zement-Kalk-Vermiculiteputz

Fig. 29. Temperature rise in the reinforcing bars of reinforced concrete floors with and without plastered undersides
(1) unplastered floors
(2) 1.5 cm cement-lime plaster
(3) 1.5 cm cement-lime-vermiculite plaster

Fig. 29. Augmentation des températures aux armatures de plafonds béton armé sans et avec crépissage
(1) Plafonds sans crépissage
(2) Crépissage en ciment et chaux, épaisseur 1,5 cm
(3) Crépissage en ciment, chaux et vermiculité, épaisseur 1,5 cm

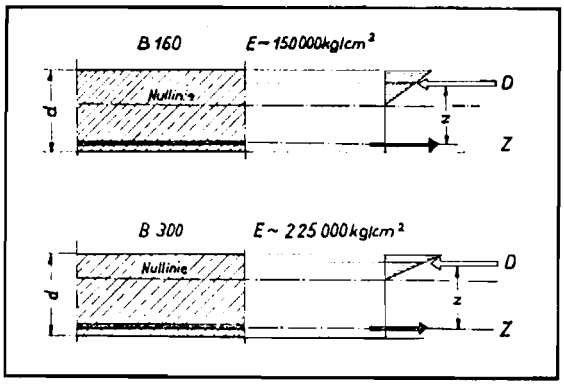


Bild 30. Veränderung des inneren Hebelarms z infolge Änderung der Betongüte unter sonst gleichen Verhältnissen (Schema)

Fig. 30. Variation of the internal lever arm z due to change of the concrete quality under otherwise equal conditions (diagrammatic).

Fig. 30. Changement du bras de levier interne z dû au changement de la qualité du béton dans les mêmes conditions (Schéma)

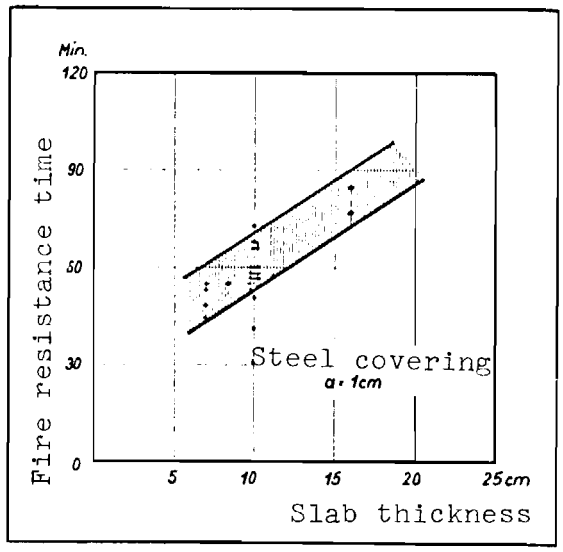


Bild 31. Einfluß der Deckendicke auf die Feuerwiderstandsdauer von Stahlbetonplatten

Fig. 31. Effect of floor thickness on the fire resistance period of reinforced concrete slabs.

Fig. 31. Influence de l'épaisseur d'un plafond en plaques de béton armé sur la résistance au feu

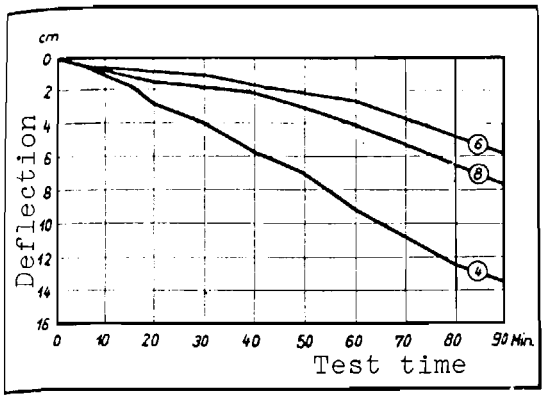


Bild 32. Durchbiegung von Fertigteildecken im Normenbrandversuch. Die Zahl bezieht sich auf Tafel 4, Spalte 1

Fig. 32. Deflection of precast floors in the standard fire test. The number refers to Table 4, column 1

Fig. 32. Déformation de plafonds en béton manufacturé à l'essai au feu normalisé. Le chiffre se rapporte au tableau 4, colonne 1

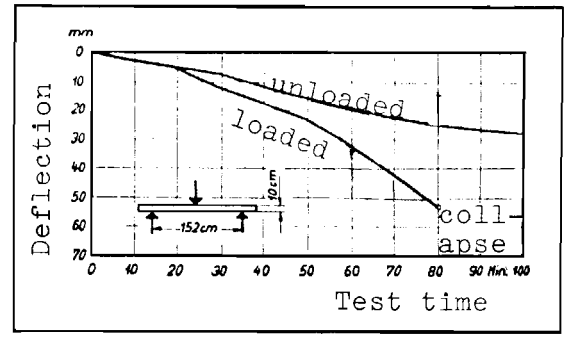


Bild 33. Einfluß der Belastung auf die Durchbiegung einer Gasbetondeckenplatte im Normenbrandversuch nach (37)

Fig. 33. Effect of loading on the deflection of an aerated concrete floor slab in the standard fire test according to (37)

Fig. 33. Influence de la charge sur la déformation d'un plafond en plaques de béton-gaz à l'essai au feu normalisé selon (37)

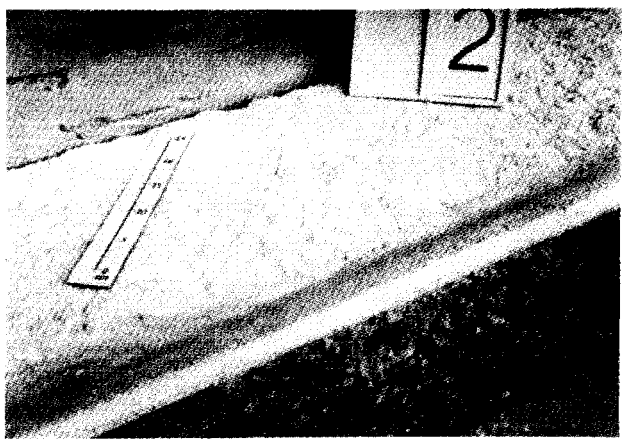


Bild 34. Brandausbreitung von oben nach unten. Oberseite einer Stahlbetonhohldiele nach dem Brand (38)

Fig. 34. Downward spread of fire. Top surface of reinforced concrete hollow flooring unit after the fire (38)

Fig. 34. Propagation du feu de haut en bas. Côté supérieur d'un plafond creux en béton armé après l'essai au feu (38)

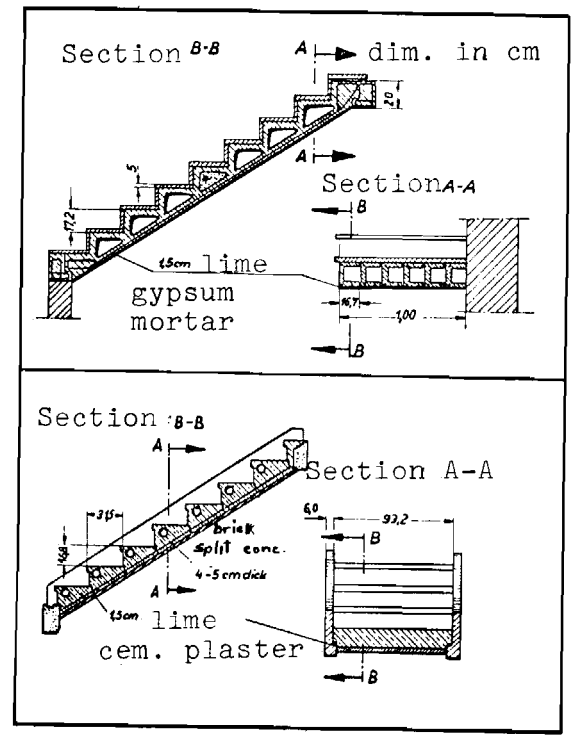


Bild 35. Betonfertigteile-Treppen

Fig. 35. Precast concrete stairs

Fig. 35. Escaliers en béton manufacturé

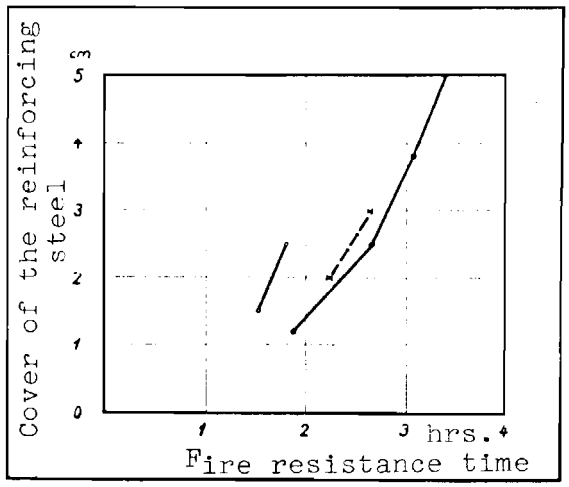


Bild 36. Einfluß der Überdeckung der Stahleinlagen auf die Feuer-Widerstandsdauer von Stahlbetonbalken
O———O britische Versuche
× - - - - - - - × niederländische Versuche
●———● deutsche Versuche

Fig. 36. Effect of the cover of the reinforcing steel upon the fire resistance period of reinforced concrete beams.
O———O British tests
× - - - - - - - × Dutch tests
●———● German tests

Fig. 36. Influence de la couverture sur l'armature en acier de poutres en béton armé sur la résistance au feu
O———O Essais britanniques
×———× Essais néerlandais
●———● Essais allemands

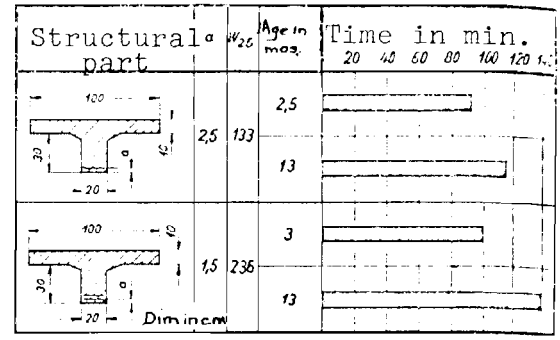


Bild 37. Einfluß des Alters von Stahlbetonbalken auf die Feuerwiderstandsdauer

Fig. 37. Effect of the age of reinforced concrete beams upon the fire resistance period

Fig. 37. Influence de l'âge de poutres en béton armé sur la résistance au feu

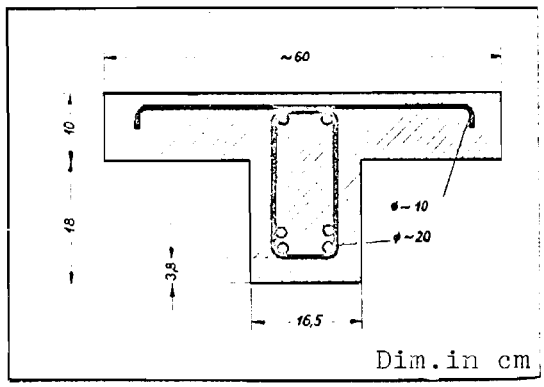


Bild 38. Brandversuche an Stahlbeton-Plattenbalken aus verschiedenen Zuschlagstoffen (28). Balkenquerschnitt

Fig. 38. Fire tests on reinforced concrete T-beams made with various aggregates (28). Cross-section of beam

Fig. 38. Essai au feu sur des poutres en plaques en béton armé avec divers agrégats (28) section des poutres

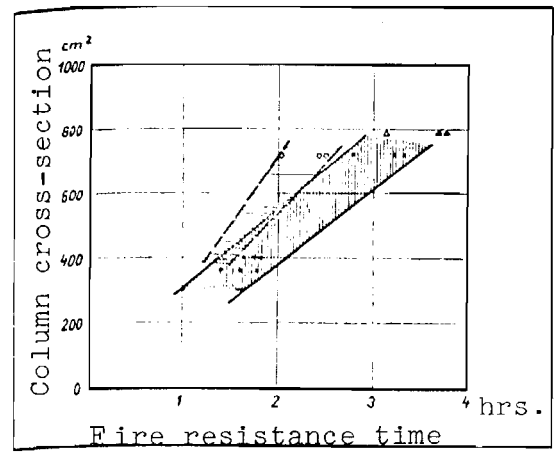


Bild 39. Feuerwiderstandsdauer von rechteckigen Stahlbetonsäulen in Abhängigkeit vom Querschnitt und Zuschlagstoffen
△ ○ quarzitischer Zuschlag
▲ ● Kalkstein-Zuschlag

Fig. 39. Fire resistance period of rectangular reinforced concrete columns as a function of cross-section and aggregates
○ quartzitic aggregate
● limestone aggregate

Fig. 39. Résistance au feu de colonnes en béton armé rectangulaires en fonction de la section et des agrégats
○ Agrégat avec quartzite
● Agrégat avec calcaire

Kreise: Deutsche Versuche; Dreiecke: Britische Versuche

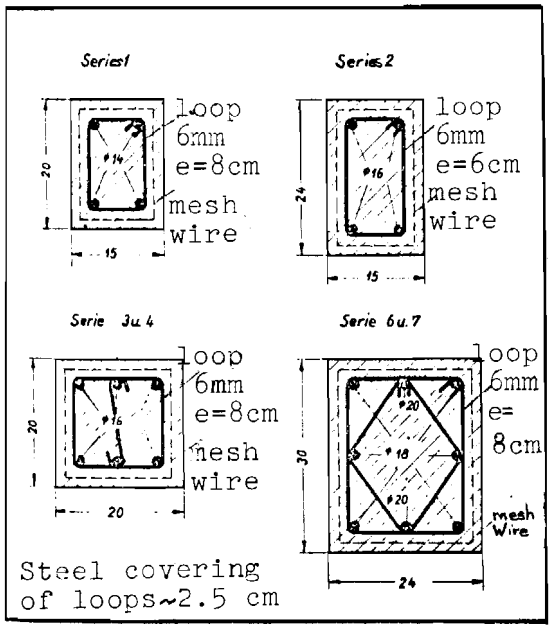


Bild 40. Querschnitte der Fertigteilstützen für die Brandversuche des Bundesverbandes der Betonsteinindustrie

Fig. 40. Cross-sections through precast columns for the fire tests of the Bundesverband der Betonsteinindustrie (German Precast Concrete Federation)

Fig. 40. Section des appuis préfabriqués destinés aux essais au feu de l'association des fabricants de béton manufacturé.

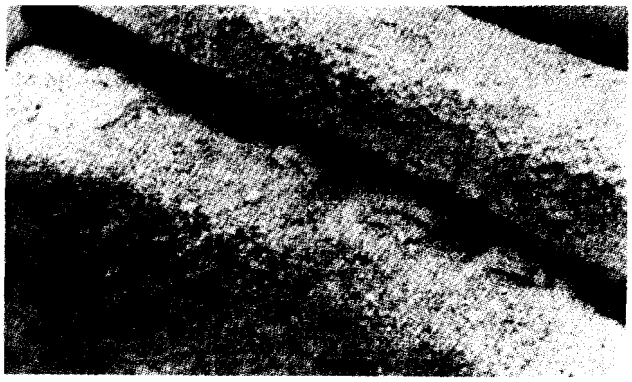


Bild 41. Stahlbetonstütze aus Kalksteinbeton mit Maschinendrahteinlage nach dem Brandversuch und anschließender Löschwasserprobe (Ausschn.)

Fig. 41. Reinforced concrete column made of limestone concrete with incorporated wire mesh after the fire test followed by spraying with water (part view)

Fig. 41. Appui en béton armé avec calcaire et filet en acier comme armature après l'essai au feu et ensuite arrosé avec de l'eau.

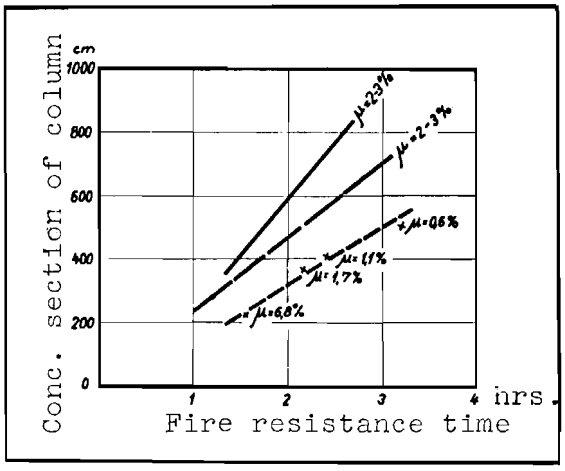


Bild 42. Einfluß von Betonquerschnitt und Bewehrungsverhältnis auf die Feuerwiderstandsdauer von Stahlbetonsäulen

——— deutsche Versuche; quarzitischer Zuschlagstoff μ = 2—3%, Säulenlänge 3,60 m

— — — deutsche Versuche; Kalkstein μ = 2—3%, Säulenlänge 3,30 m

—×—×—×—× französische Versuche; quarzitischer Zuschlagstoff μ = 0,6—6,8%, Säulenlänge 2,30 m

Fig. 42. Effect of concrete section and reinforcement proportion upon the fire resistance period of reinforced concrete columns

——— German tests; quartzitic aggregate μ = 2—3%; length of column 3.60 m

— — — German tests; limestone μ = 2—3%; length of column 3.30 m

—×—×— French tests; quartzitic aggregate μ = 0.6—6.8%; length of column 2.30 m

Fig. 42. Influence de la section du béton et de l'armature de colonnes en béton armé sur la résistance au feu.

——— Essais allemands, agrégat avec quartzite μ = 2—3%, Longueur de la colonne 3,60 m

— — — Essais allemands; Calcaire μ = 2—3%; Longueur d ela colonne 3,30 m

—×—×—×—× Essais français, agrégat avec quartzite μ = 0,6—6,8%,; Longueur de la colonne 2,30 m

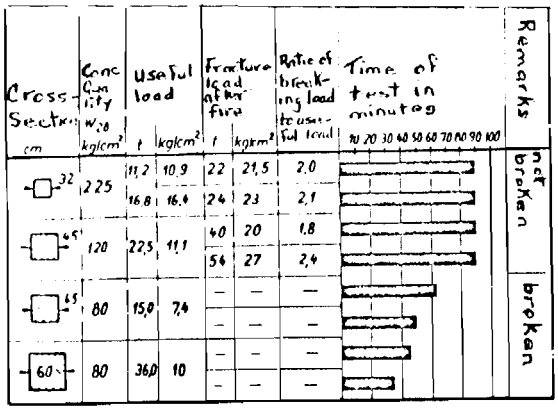


Bild 43. Brandversuche an unbewehrten Betonstützen nach (44)

Fig. 43. Fire tests on plain (unreinforced) concrete columns according to (44)

Fig. 43 Essai au feu à des appuis en béton non armé selon (44)

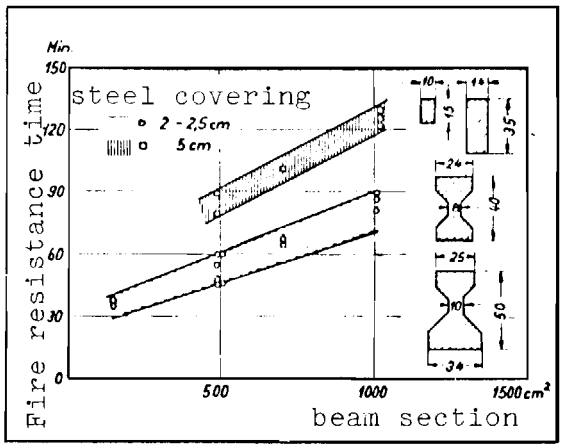


Bild 44. Einfluß von Balkenquerschnitt und Stahlüberdeckung auf die Feuerwiderstandsdauer von Spannbetonbalken mit sofortigem Verbund. (Gedrungene und Querschnitte)

Fig. 44. Effect of beam section and cover to steel upon the fire resistance period of prestressed beams with pre-tensioned tendons. (Squat sections and I-sections)

Fig. 44. Influence de la section et de la couverture sur l'armature de poutres en béton précontraint, assemblées immédiatement, sur la résistance au feu (Petites sections et en forme I)

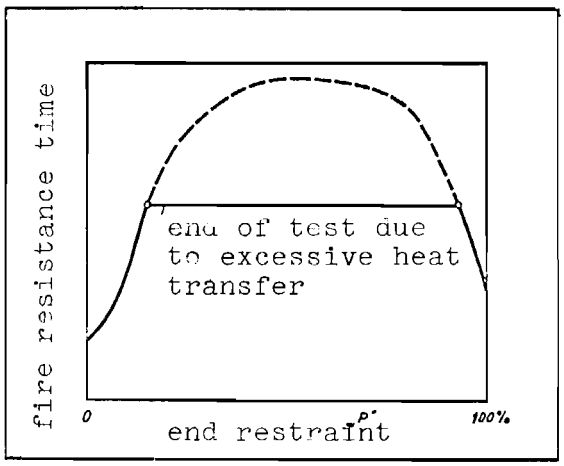


Bild 45. Wirkung einer Endeinspannung auf die Feuerwiderstandsdauer von Spannbeton (schematisch) (51)

Fig. 45. Effect of end restraint upon the fire resistance period of prestressed concrete (diagrammatic) (51)

Fig. 45. Effet d'une fixation d'un béton précontraint sur la résistance au feu (Schématique) (51).

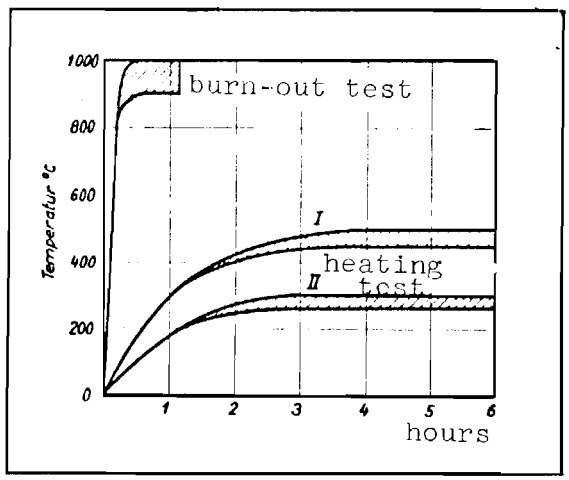


Bild 46. Temperaturzeitkurven für Ausbrenn- und Heizversuche an Schornsteinen nach DIN E 18 160

Fig. 46. Time-temperature curves for burn-out tests and heating tests on chimneys according to DIN E 18 160

Fig. 46. Courbes de température-temps pour des essais d'échauffement et au feu aux cheminées selon DIN 18160.

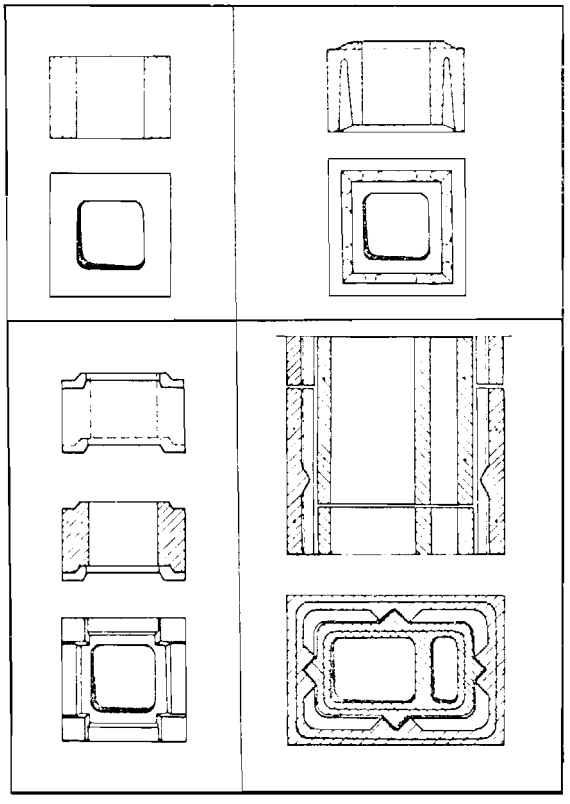


Bild 47. Beispiele von Schornsteinformstücken aus Leichtbeton
Fig. 47. Examples of lightweight concrete chimney units
Fig. 47. Exemples de pièces en béton léger pour cheminées.

Bild 48. Wand aus Glasbausteinen während des Normenbrandversuches
Fig. 48. Wall of glass building blocks during the standard fire test
Fig. 48. Paroi en blocs en verre durant l'essai au feu normalisé.

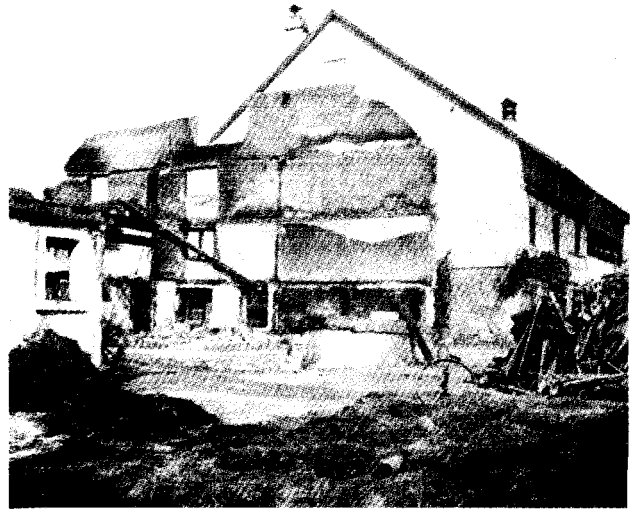


Bild 49. Verkleidung aus Asbestzementplatten schützte gegen Flugfeuer

Fig. 49. A facing of asbestos cement slabs afforded protection against flying sparks

Fig. 49. Parement en plaques d'amiante-ciment protégeait contre un feu emporté.

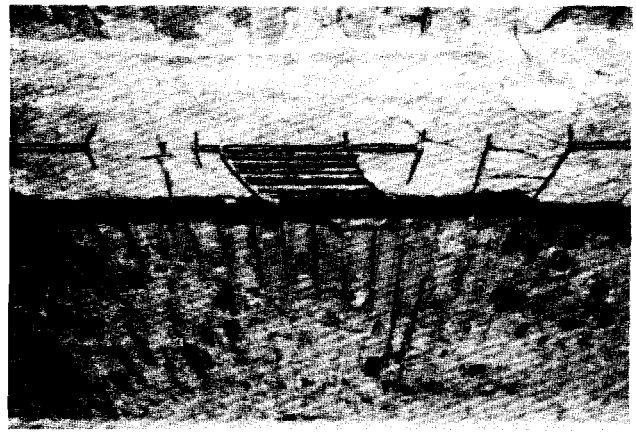


Bild 50. Stahlbetonbalken mit unterschiedlichem Bügelabstand nach einem Schadensfeuer (nach 58).

Fig. 50. Reinforced concrete beams with varying stirrup spacing after a fire (from 58)

Fig. 50. Poutres en béton armé à différentes distances de l'étrier après un incendie (selon 58).