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Effect of High Temperatures on Tensile Strength of Concrete

Draghållfastheten hos betong vid höga temperaturer

Sven Thelandersson

1. Introduction

Knowledge of strength and deformation properties of concrete at elevated temperatures is essential in estimating the behaviour and the load-bearing capacity of reinforced concrete structures when influenced by fire. The necessity of such knowledge has been emphasised during the recent years by the structural fire research which is tending towards a functionally based design practice. This practice involves the determination of the time-temperature curve, based on characteristics of the fire compartment and its fire load. The time-temperature curve thus obtained is used to determine the temperature field inside the structure in question. Starting from this temperature fields the behaviour and the load-bearing capacity of the structure are analysed. Comprehensive knowledge of strength and deformation properties of the material at elevated temperatures – usually up to 1000°C – is needed for this design procedure.

The use of prestressed concrete as a dominating construction material in nuclear power plants has aroused interest in the study of property changes of concrete at high temperatures. In such cases temperatures up to 400°C are involved.

In connection with the influence of fire on concrete, the existing studies have been mainly concerned with compressive strength. A frequently quoted reference is the report on the experiments performed by Malhotra [1] in 1956 on concrete with Portland cement and aggregate composed of flint stone and sand from River Thames. Malhotra found that the strength increases with decreasing cement-aggregate ratio, that compressive loading on specimens during heating results in an increased compressive strength, and that the strength in a hot state is greater than that after cooling.

Among other experiments on compressive strength of concrete at high temperatures the tests performed by Weigler and Fischer [2] and Abrams [3]

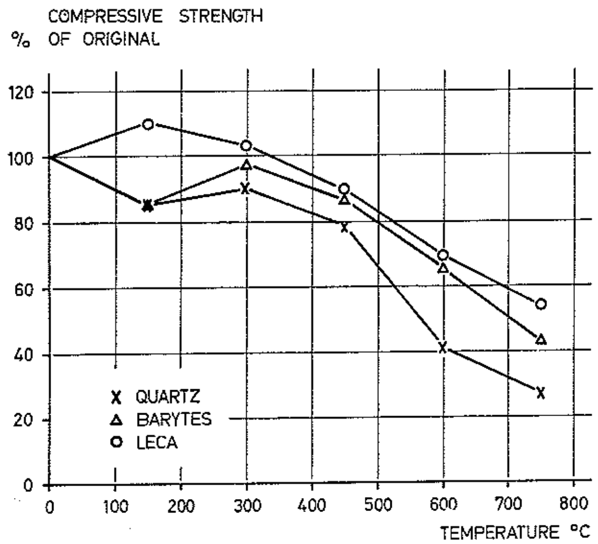


Fig. 1. Compressive strength of concrete made with various aggregates after heating and subsequent cooling as a function of test temperature [2] ● *Tryckhållfastheten hos betong med olika typer av ballast som funktion av temperaturen efter uppvärmning och följande avsvalning.*

should be mentioned. Weigler and Fischer have studied concrete with Portland and Hochofen cement combined with an aggregate of the type quartz, barytes or leca. The influence of a number of parameters on compressive strength is explained. Besides, weight loss, gas permeability and thermal expansion under loading are studied. An example of the results of these comprehensive tests is given in Fig. 1, which shows the variation of compressive strength with temperature for different aggregates. It is seen from Fig. 1 that an aggregate of quartz type is somewhat more unfavourable in respect of heat resistance than the other types of aggregate studied.

Abrams as well as Malhotra investigated the influence of compressive loading during heating. The compressive strength in the hot state and after cooling down to room temperature was determined. Other variables in Abrams' investigation were type of aggregate and concrete quality. In Fig. 2, which was obtained from this investigation, the compressive strength is shown as a function of the temperature for (a) specimens not loaded during heating and tested hot; (b) specimens loaded during heating with 40% of the original ultimate load and tested hot; and (c) specimens not loaded during heating and tested after cooling down to room temperature, 7 days after the heating period. The diagram, which corresponds to concrete with aggregate of quartz type, shows that compressive loading during heating results in increased compressive strength and that the residual strength is lower than the strength in a heated state.

As regards the tensile strength of concrete at high temperatures, the available information in the literature is rather scarce. Zoldners [4] has deter-

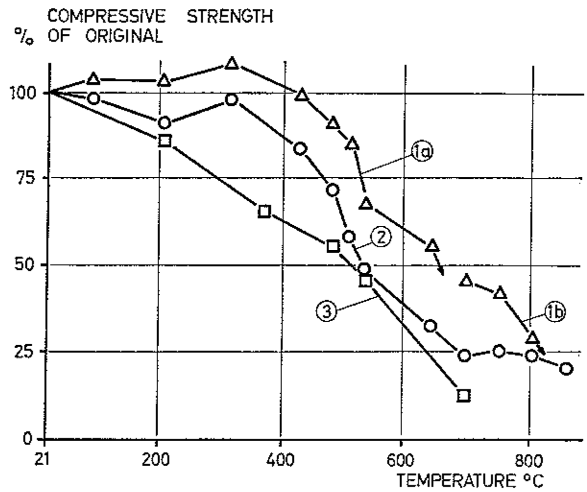


Fig. 2. Effect of temperature on compressive strength of siliceous aggregate concrete. The specimens, cylinders 76 mm in diameter, were cured in air until the relative humidity in the centre reached 75 per cent. Original compressive strength $f'_c = 27.5$ MN · m^{-2}. During heating the maximum temperature difference within the specimen was limited to 83°C. Curve 1: Specimens stressed to 0.4 f'_c (1 a) or 0.25 f'_c (1 b) during heating and then tested in the hot state. Curve 2: Specimens unstressed during heating and tested in the hot state. Curve 3: Specimens heated and then cured 7 days in air (21°C, 70–80 per cent RH) before testing. The specimens were unstressed during heating ● *Inverkan av temperaturen på tryckhållfastheten hos betong med ballast med hög halt av silikatmineraler. Provkropparna, cylindrar med diametern 76 mm, lagrades i luft tills relativa fuktigheten i centrum nådde värdet 75 %. Hållfastheten vid 21°C var f'_c = 27,5 MN/m². Under uppvärmningen översteg den maximala temperaturdifferensen inom provkroppen aldrig värdet 83°C. Kurva 1: Provkroppar belastade med spänningen 0,4 f'_c (1 a) eller 0,25 f'_c (1 b) under uppvärmningen och provade i varmt tillstånd. Kurva 2: Provkroppar obelastade under uppvärmningen och därefter provade i varmt tillstånd. Kurva 3: Provkroppar som värmts upp, långsamt avkylts och därefter lagrats 7 dygn i luft (21°C, 70–80 % RH) före provningen. Provkropparna var obelastade under uppvärmningen.*

mined the flexural strength of concrete with different aggregates at temperatures up to 700°C. Sullivan and Poucher [5] have recently made certain determinations of flexural strength of concrete and cement mortar at temperatures in the interval 20°C to 400°C. Otherwise, information on tensile strength of concrete at elevated temperatures is lacking in the literature.

The tensile strength is significant in connection with fractures of brittle character, such as rupture caused by shear and torsion. In a concrete construction exposed to fire or affected by temperature, in one way or another, thermal stresses usually arise owing to nonuniform temperature distribution. In order to estimate such stresses, it is necessary that the tensile strength at the current temperatures should be known.

In the present report, an account is given of results obtained from a graduate thesis by G. Jönsson and C. Lassen, Division of Structural Mechanics and

Concrete Construction, LTH [6], and from investigations performed by the Author[1]. The tensile strength of concrete was determined in the temperature interval 0–800°C by split-cylinder tests. The investigated parameters are heating and cooling rates, concrete composition and time after the day of heating prior to testing. The tests were made both in the hot state and after cooling down to room temperature.

The scope of the experiments and the test procedures are described in Section 3 in detail, while the results are reported in Section 4. In Section 2 a review is given of the mechanical and physical influences on the concrete exposed to elevated temperatures, with the purpose of providing a basis for the discussion of the results obtained from the experiments. Finally a summary of the conclusions obtained from the experiments is given in Section 5.

2. *General information on concrete at elevated temperatures*

The influence on the material properties of concrete caused by subjecting it to high temperatures can be characterised as follows:

Physical and chemical changes in cement paste and aggregate.
Internal stresses due to thermal incompatibility between cement paste and aggregate.
Internal stresses due to non-uniform temperature distribution.

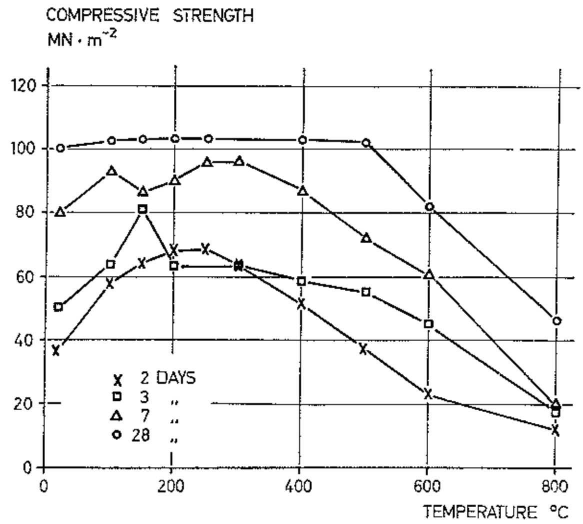
2.1 Physical and chemical changes in cement paste and aggregate

The physical changes taking place during heating are mainly caused by water leaving cement paste. Based on some extensive Russian research, Nekrassow, [7] and [8], explains the physical and chemical processes taking place in cement paste when exposed to high temperatures.

When cement paste is heated from 0°C to about 200°C, the evaporable water is extracted. At the same time the cement grains increase in volume. According to Nekrassow it follows that the internal structure is densified and compressive strength increased. The magnitude of the increase depends mainly on the degree of hydration, which varies with age and curing conditions. The *relative* increase in compressive strength in older, dried specimens becomes insignificant or vanishes. This fact can be deduced from Fig. 3, which shows the variation in compressive strength in Portland cement paste with temperature at different

[1] The investigations were carried out on a grant from the National Council for Building Research in Sweden.

Fig. 3. Compressive strength of cement paste as a function of temperature at different ages [7] ● *Tryckhållfastheten hos cementpasta vid olika åldrar som funktion av temperaturen* [7].



ages [7]. The curves refer to compressive strength immediately after cooling down to room temperature.

Russian tests show that no similar increase in flexural strength of cement paste occurs in the interval 0–200°C [8]. On the contrary a certain decrease is noticed at temperatures up to 300°C. The decrease quantitively depends on the composition of the cement and on curing conditions.

Between 200°C and 500°C water leaves the clinker hydrates, and this results in a considerable shrinkage. This shrinkage takes place despite a simultaneous increase in volume of calcium hydroxide crystals and non-hydrated cement grains. Owing to the heating, the internal structure of cement paste weakens and the strength decreases somewhat. This decrease in strength is, however, moderate compared with that in the critical temperature range, 500–600°C, as a result of dehydration of calcium hydroxides, Fig. 3. A slow deterioration of calcium hydroxide crystals starts at a temperature of about 400°C and gets faster as temperature rises. As a result of heating the cement paste to 600°C, the greater part of the calcium hydroxide in the cement paste decomposes into lime (CaO) and water, which evaporates. During this process the internal structure of the cement paste breaks down considerably. It has been shown that cement paste, which has been exposed to temperatures above 500–600°C, will undergo a further decrease in strength if it is stored in air at room temperature [7]. The same applies to concrete [2]. The reason, according to Nekrassow, is that the free lime (CaO) formed during the heating reacts with water vapor in the air and calcium hydroxide, $Ca(OH)_2$, is regained. Since this reaction takes place under a strong expansion, cement paste or concrete subjected to such a transformation must have its internal structure further weakened.

This explanation is, however, rejected by Weigler and Fischer [2]. They show that concrete which is cured in water after cooling down, does not undergo any structural deterioration, on the contrary it regains some of its strength some time after heating. They do not, however, give an alternative explanation of this phenomenon.

Regarding the resistance of the different aggregate types to high temperatures, it can be established that most of the minerals composing the aggregates have different coefficients of thermal expansion in different directions. This implies that internal stresses arise due to temperature rise and the strength weakens. In such minerals as granite, gneiss, quartzite, etc., containing quartz the quartz transformation at about 575°C is also significant, and this means that quartz turns from one allotropic modification to another under an intensive expansion. At the same time the strength weakens considerably. As a result of this fact, the rocks containing quartz exhibit a poorer heat resistance than those without quartz. Limestone is an example of the latter category.

2.2 Internal stresses in concrete due to thermal incompatibility between cement paste and aggregate

Probably the most important reason why the strength of concrete weakens as a result of heating is the fact that the aggregate expands during heating while the cement paste shrinks simultaneously. The volume changes versus temperature for cement paste and different types of aggregates, [7] and [9], are shown in Fig. 4. It can be seen that the cement paste somewhat expands up to about 150°C, but with further temperature rise it shrinks considerably. The volume of the aggregate, on the other hand, increases with the temperature with different intensities for different kinds of rocks. This means that we have a significant difference between the volume changes in the constituent materials, causing considerable internal stresses.

The curves reproduced in Fig. 4 should only be regarded as examples for qualitative comparison. Different experiments have given different results for the same rock, and the shrinkage of the cement paste depends on water-cement-ratio, age, curing conditions and other factors.

In Fig. 4 is also inserted the thermal expansion of concrete with an aggregate of granite type (curve 2). It can be observed that the thermal expansion of concrete is much closer to that of the aggregate than to that of the cement paste. It follows that a large deformation is imposed on the cement paste in concrete

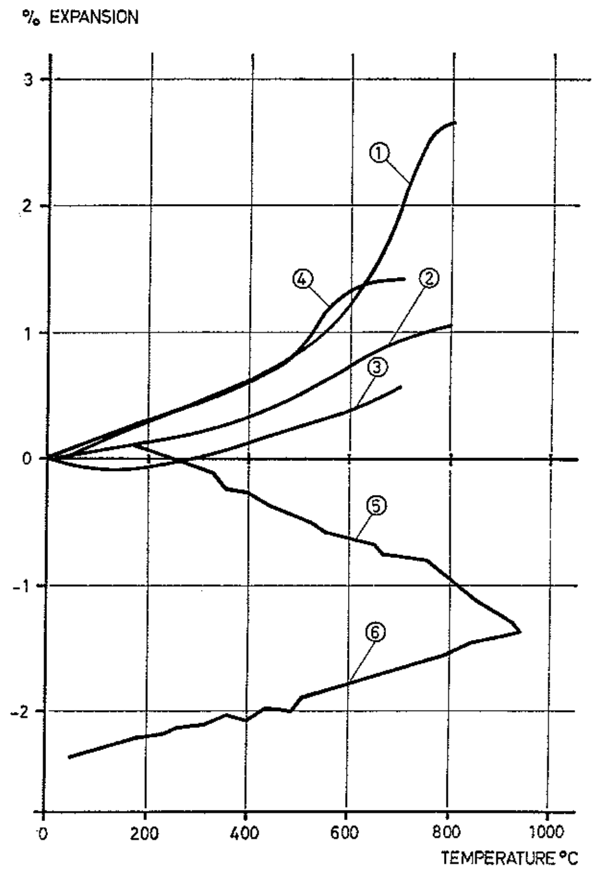


Fig. 4. Effect of temperature on thermal expansion of: 1. Granite; 2. Concrete made with granite aggregate; 3. Limestone; 4. Sandstone; 5. Cement paste in process of heating; 6. Cement paste in process of cooling ● *Termisk dilatation för: 1. Granit; 2. Granitbetong; 3. Kalksten; 4. Sandsten; 5. Cementpasta under uppvärmning; 6. Cementpasta under avsvalning.*

which is subjected to high temperatures. This leads rather quickly to crack formation in the cement paste.

In view of the fact that the thermal expansion of concrete is a sum of expansion of the aggregate and shrinkage of the cement paste, it is natural that a compressive stress applied to a concrete specimen strongly affects the thermal expansion of the specimen. This can clearly be seen from Fig. 5, which shows the thermal expansion of concrete for unloaded specimens as well as specimens loaded in compression at levels corresponding to $\frac{1}{6}$, $\frac{1}{3}$ and $\frac{1}{2}$ of the ultimate load at room temperature [2]. It can be seen that already when the load amounts to half of the ultimate load, the thermal expansion is completely eliminated and at 500°C a reduction in length is observed.

Experiments also show that specimens which are loaded in compression during heating exhibit a higher strength than unloaded specimens [1, 3], see

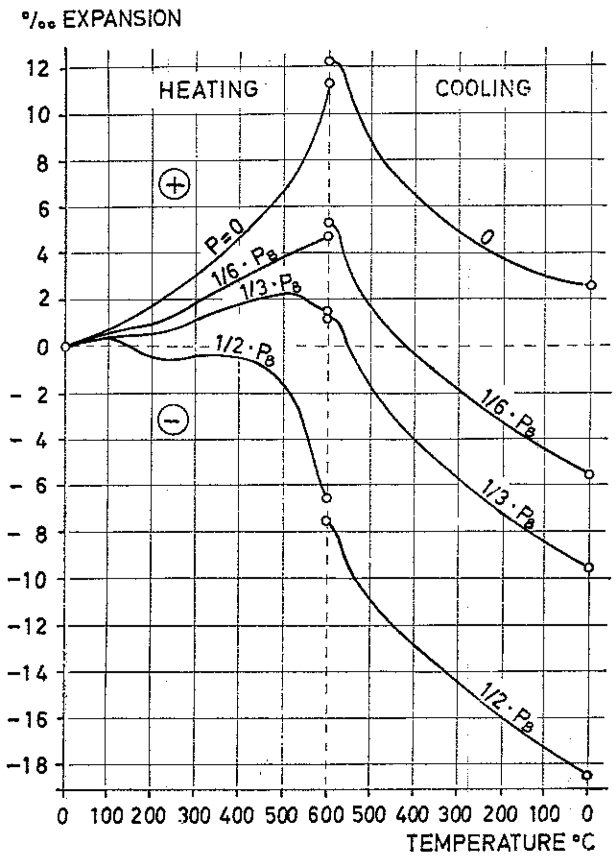


Fig. 5. Thermal movement of concrete specimens subjected to various compressive stress levels at a heating and cooling rate of 120°C · h^{-1}. The temperature was kept at 600°C for 3 hours. P_B = original compressive strength ● *Termisk dilatation hos betongprovkroppar belastade till olika spänningsnivåer. Uppvärmning och avsvalning skedde med hastigheten 120°C/h och temperaturen hölls konstant vid 600°C i 3 timmar.* P_B *betecknar hållfastheten vid 20°C.*

Fig. 2. The applied compressive stress results in a reduction of crack formation in the cement paste.

2.3 Internal stresses due to non-uniform temperature distribution

During the process of heating and cooling down the concrete specimens, internal stresses of varying magnitude arise owing to the appearance of temperature gradients. The temperature distribution during heating depends on rate of heating, shape and size of specimens and thermal properties of the concrete. The magnitude of the stresses caused by the non-uniform temperature distribution depends on thermal expansivity and deformation properties of the concrete, as well as on external loads. With the present knowledge it is impossible to satisfactorily determine the stresses caused by temperature, on the basis of a given temperature field.

A concrete specimen is affected in a similar manner by stresses due to thermal gradients developed during heating as it is by internal stresses due to thermal incompatibility, i.e. cracks are formed and the strength decreases.

3. *Scope of tests*

3.1 General introduction

In this paper the tests performed by Jönsson and Lassen [6] are designated by Series A while those performed by the Author are designated by Series B. The scope of the tests is given in Tables 1 and 2.

The interval 0–800°C was studied at 8 different temperatures. The precise temperature measurements by thermocouple are given in Tables 1 and 2.

The rate of heating in Series A was made to vary by covering some specimens with diabase wool during heating. In this way three different rates of heating, corresponding to specimens without insulation, specimens insulated with 5 mm thick diabase wool layer, and specimens insulated with 10 mm thick diabase

Table 1. Test series A.

Test series No.	Temp. t°C	Age of testing days	Number of specimens: 20°C 28 days	20°C age of testing	t°C no insul	t°C 5 mm insul	t°C 10 mm insul	Tensile strength at 20°C (MN · m⁻²): 28 days	age of testing	Mix-ture
	80	68	4	3	7	8	8	4.13	4.63	
	205	67	4	4	8	8	7	3.81	3.69	
	310	66	4	4	7	8	8	4.45	3.88	
A_1	420	68	3	3	7	7	7	3.59	3.82	A:I
	495	63	4	4	8	8	8	4.31	4.48	
	590	61	4	4	8	7	8	3.95	3.87	
	695	59	4	4	7	8	8	4.65	4.19	
	795	58	4	4	7	8	7	4.55	4.72	
	85	67	4	3	6	8	8	5.28	5.22	
	200	66	4	4	8	8	8	5.84	5.16	
	320	65	4	4	7	8	8	5.96	5.27	
A_2	425	65	4	4	8	8	8	5.66	5.50	A:II
	500	61	4	4	8	8	8	5.89	5.30	
	595	60	4	4	8	8	8	5.52	5.08	
	700	58	4	4	8	7	8	6.16	6.11	

Table 2. Test series B.

Test series No.	Temperature	Number of specimens 20°C	warm	cooled	Tensile strength at 20°C $MN \cdot m^{-2}$	Cube strength $MN \cdot m^{-2}$	Rate of heating	Mixture
	110	6	7	6	3.66	46.3		
	215	5	5	7	4.18	40.8		
	300	5	6	7	3.91	36.8		
B_1	415	6	7	6	3.23	34.8	Slow	B:II
	515	5	7	7	3.60	37.3		
	600	6	7	7	4.14	44.8		
	700	6	7	7	4.65	47.0		
	800	6	6	7	3.98	38.4		
	105	6	6	7	2.72	24.3		
	200	6	6	7	2.50	21.6		
	305	6	7	6	2.52	21.2		
B_2	445	6	7	7	2.73	21.6	Slow	B:I
	505	6	7	6	2.45	18.7		
	600	6	7	7	2.70	20.0		
	700	6	7	6	2.37	20.3		
	800	6	7	7	2.26	22.0		
	120	6	7	7	3.94	–		
	220	6	7	7	3.44	–		
	320	6	7	7	4.23	–		
B_3	410	6	7	7	4.07	–	Rapid	B:II
	540	6	6	7	3.72	–		
	620	6	6	7	4.29	–		
	700	5	7	6	3.76	–		
	400	6		7+7	4.33	–		
B_4	600	6		7+7	4.02	–	Slow	B:II
	800	6		7+7	4.43	–		

wool layer were obtained. This variation in rate of heating proved to have no significant effect on the tensile strength (see Section 4). In Series B, a comparison was therefore made between two extreme cases of rate of heating which are denoted in Table 2 by "rapid" and "slow" heating, respectively. The significance of these notations is explained in Section 3.3.

In both test series, the effect of cement-aggregate ratio was investigated. The information on the concrete composition is given in Section 3.2.

All the tests in Series A were performed after cooling down to room temperature, while in Series B the specimens were tested both in a hot state (strength at elevated temperatures) and a cooled state (residual strength).

A few experiments were also carried out in order to illustrate to a certain extent the change of residual strength during the period after the heating. These experiments are denoted by B_4 in Table 2 and include a determination of the residual strength corresponding to 7 and 28 days after the day of heating, respectively. The specimens were cured in a damp room with 65% relative humidity prior to testing.

3.2 Specimens

In all cases under consideration the specimens were cylindrical, the length of the cylinder being twice as long as the diameter. In Series A the diameter of the specimens measured 50 mm and that of Series B 94 mm. The specimens were cast in vertical tubular moulds and cured for one day in the moulds under wet sacking followed by 4 days in water, and the rest of the time prior to testing, they were cured in air at a temperature of 20°C and at a relative humidity of about 60%. In Series B the testing age was 28 days in all the cases. In Series A the age varied from 58 to 68 days. The precise age corresponding to each experiment is given in Table 1.

In both Series A and B two different concrete mixtures corresponding to 28-day cube strengths of 25 and 40 $MN \cdot m^{-2}$ (250 and 400 $kp \cdot cm^{-2}$) were used. They are denoted by A:I, B:I and A:II, B:II, respectively. In the different cases, the proportions between aggregate, fine sand, cement and water can be seen from Table 3.

The aggregate was composed of crushed stone from rocks containing siliceous minerals. The maximum particle size of crushed stone in Series A was 16 mm and in Series B 12 mm.

Table 3. Concrete mix design.

Concrete mix No.	Cement paste: Aggregate	Cement: Fine sand: Coarse aggr.	Water: Cement
A:I	1:4.7	1:3.15:4.80	0.68
A:II	1:3	1:1.82:2.68	0.50
B:I	1:4	1:3.15:3.85	0.75
B:II	1:3	1:2:2.65	0.55

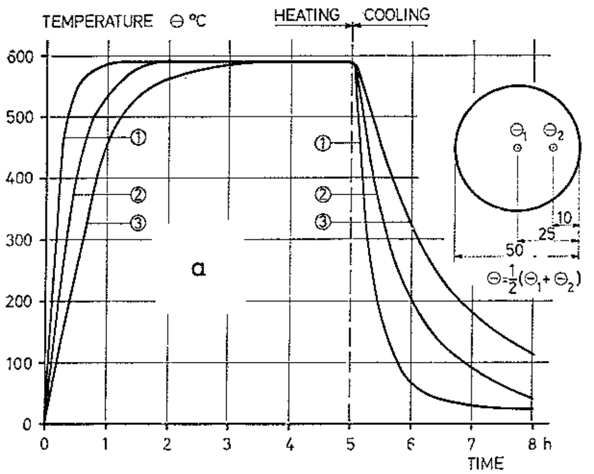


Fig. 6. Temperature-time curves for cylindrical concrete specimens subjected to "rapid" heating ● *Temperatur-tid kurvor för betongcylindrar utsatta för "snabb" uppvärmning.*

a) Test series A, diameter 50 mm. Curve 1: No insulation. Curve 2: 5 mm diabase wool insulation. Curve 3: 10 mm diabase wool insulation ● *a) Serie A, 50 mm diameter. Kurva 1: Utan isolering. Kurva 2: 5 mm mineralullsisolering. Kurva 3: 10 mm mineralullsisolering.*

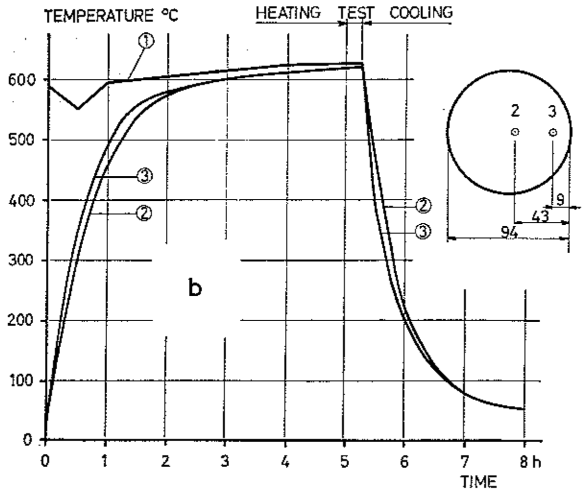
3.3 Heating

The heating of the specimens took place in an electrical oven with an internal volume of 1 m^3. The spiral heating system of the oven was connected in two separate groups, one being adjusted by a thermostat and the other by a hand-operated switch. Thus, a certain adjustment of the heating process was made possible and the temperature could be kept constant at a certain definite value. The thermostat was connected to a thermocouple whose junction was fitted in the upper part of the oven. The oven temperature was also measured in close neighbourhood of the specimens. During each heating period the temperature inside a reference specimen was measured by two embedded thermocouples, one in the centre of the specimen and the other about 10 mm away from the external surface.

The specimens were exposed to heating in two different ways which in Table 2 are denoted by "rapid" and "slow" heating, respectively.

"Rapid" heating implies that the specimens were inserted in the oven when the temperature in it had reached the desired value. The specimens were allowed to remain in the oven for 5 hours and this time was enough for the specimens to assume the constant temperature of the oven. The deviation of the temperature in the oven from the constant value during the heating period amounted to $\pm$ 20°C. At the end of the period the specimens were taken out of the oven and allowed to cool down to room temperature prior to testing. All the experiments in Series A were performed using "rapid" heating. The heating and cooling rates were made to vary by providing some of the specimens with 5 or 10 mm thick diabase wool insulation. In Fig. 6a the temperature is shown

b) Test series B, diameter 94 mm. Curve 1: Oven temperature. Curves 2–3: Temperature inside specimen ● *b) Serie B, 94 mm diameter. Kurva 1: Ugnstemperatur. Kurva 2–3: Temperatur inuti provkroppen.*



as a function of time for specimens with a diameter of 50 mm, in cases where the specimens were non-insulated or insulated with 5 and 10 mm thick diabase wool insulation, respectively. The curves correspond to an oven temperature of 600°C and the temperatures inside the specimens is expressed as the mean value of the temperature at the two measuring points, i.e. the centre of the specimen and 10 mm away from the external surface. In Fig. 6b the corresponding curves are shown for 2 points in a non-insulated cylinder with a diameter of 94 mm belonging to Series B.

"Slow" heating implies that the specimens were inserted in the oven while the oven was still cold. The oven was switched on to half the power capacity and after 1.5 hours to the maximum power. The temperature of the oven (Curve 1) and the temperature of the specimens (Curves 2–5) obtained in this way are shown in Fig. 7a. Upon testing, when the temperature of the oven reached a desired value, further heating was stopped and then the temperature of the oven was held constant for about 2 hours by the thermostat. During this time the temperature inside the specimens stabilized itself at the temperature in question, whereupon half of the specimens were tested in the hot state. Then the current to the oven was switched off while the remaining half of the specimens were left in the oven whose temperature decreased slowly. After 24 hours, when the specimens had cooled down to room temperature, the tests were performed. In Fig. 7b the temperature is shown as a function of time for such a process when the testing temperature is 600°C. During the slow heating, the heating rate never exceeded the value 2°C/min. This procedure implies that the total time from the start of heating until testing of the hot specimens varied from 3 hours at 100°C to 10 hours at 800°C.

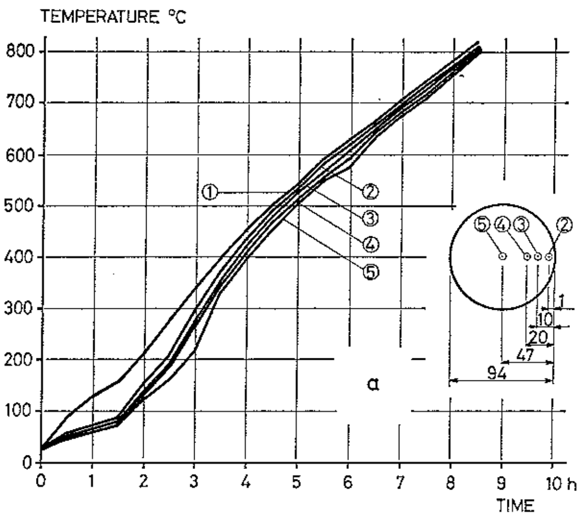
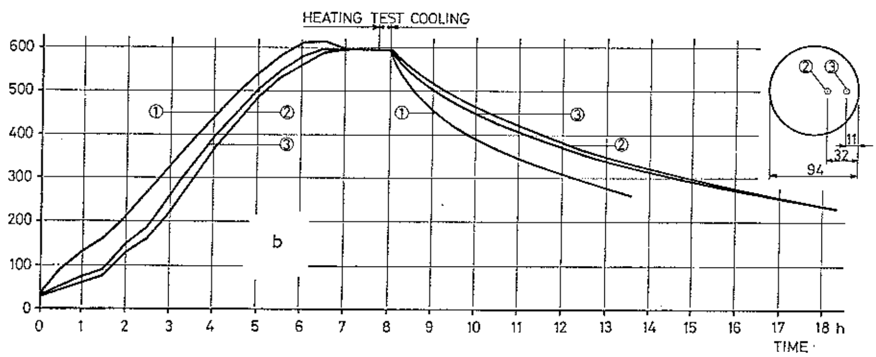


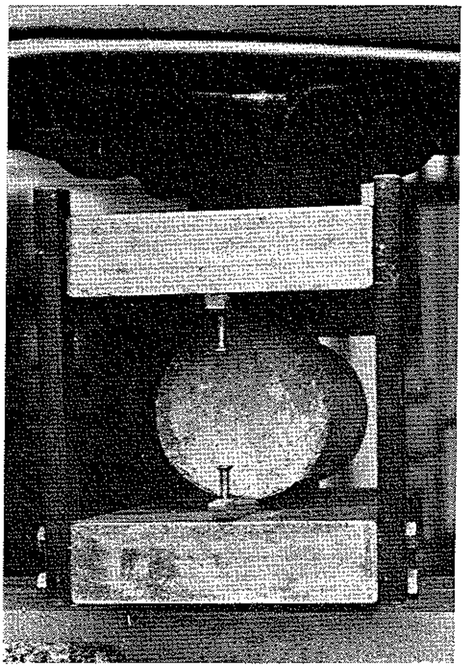
Fig. 7. Temperature-time curves for cylindrical concrete specimens subjected to "slow" heating. Curve 1: Oven temperature. Curves 2–5: Temperature inside specimen. a) Heating. b) Complete cycle at test temperature 600° C ● *Temperatur-tid kurvor för betongcylindrar utsatte för "långsam" uppvärmning. Kurva 1: Ugnstemperatur. Kurva 2–5: Temperatur inuti provkroppen. a) Uppvärmning. b) Fullständig cykel vid provningstemperaturen 600° C.*



3.4 Testing

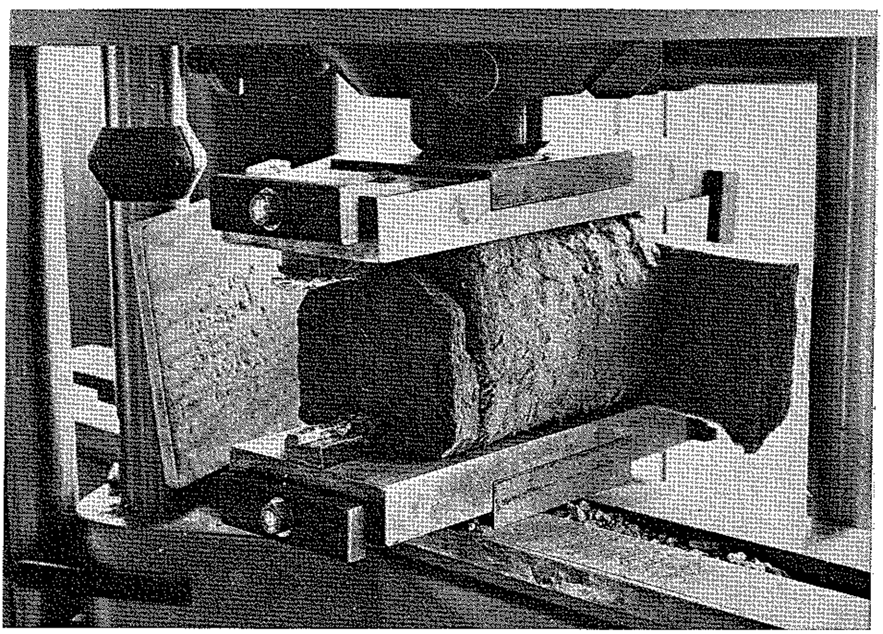
The tensile strength was determined indirectly by split-cylinder method. The cylinders were loaded along two opposite sides by line loads, see Fig. 8a. The load was applied through hard strips of fibre board. According to practice, cfr. [10], these strips should be 50 mm broad. Accordingly, in test series A 50 mm broad and 3.2 mm thick strips of fibre board were used. However, Bonzel [11] states that the width of the load-distributing strips should not exceed 1/10 of the cylinder diameter. The use of wider strips involves the risk of getting too high values of the tensile strength. Therefore, in Series B, all the strips used had a width of 9 mm. The possible errors in

Fig. 8. Split-cylinder test. a) Cold specimen. b) Hot specimen ● *Spräckprov. a) Kall provkropp. b) Varm provkropp.*



◀ a

b ▼



Series A, arising from using too broad strips, may be eliminated if the tensile strength is represented in a relative form, i.e. the ratio between the tensile strength at the current temperature and the tensile strength at room temperature for control specimens taken from the same batch.

Testing in the hot state presented special difficulties. The oven was placed in the immediate vicinity of the testing apparatus so that the testing might be performed rapidly. The specimens were taken out of the oven and wrapped in diabase wool and then placed in the testing apparatus as fast as possible. During the testing, the specimens were wholly packed in diabase wool as shown in Fig. 8b. The time between the removal of the specimens from the oven and the occurrence of fracture never exceeded 1.5 min. At higher temperatures the load-distributing strips had to be protected with aluminium paint and aluminium foil, in order to avoid charring.

The number of tested cylinders for each parameter combination is given in Tables 1 and 2. Owing to accidental loss of specimens, this number somewhat varies in different cases.

Each batch was checked with respect to tensile strength at room temperature. The number of such control tests is also shown in Tables 1 and 2. The control specimens were tested at the same time as the corresponding hot specimens. In Series A, some control specimens were taken at the age of 28 days as well. The cube strength of concrete in series B_1 and B_2 was checked and is indicated in Table 2 as the mean value for three cubes.

4. *Results*

4.1 General remarks

In order to make direct comparisons between different parameters possible and in order to eliminate the differences between different batches, the results are expressed in a relative form, i. e. the mean value of observations for a given parameter combination is divided by the mean value of the corresponding control specimens. Thus, the tensile strength of concrete is obtained in per cent of its strength at room temperature.

4.2 Dispersion in test values

A measure of the dispersion of the test values is the standard deviation estimated from the 6–8 observations for each parameter combination (cell). However, the number of observations is so small that an estimation of the corresponding standard deviation becomes uncertain. If instead we divide the tests according to the test procedure into three types, i.e. control, residual strength and strength in a heated state, then it is reasonable to assume that all the test results related to the same type exhibit the same coefficient of variation, ν_k. Then we can estimate this from a considerably greater number of observations.

An application to the results obtained from Series A yields the values $\nu_k = 10.2\%$ and 14.2% for the control and residual strength tests, respectively. For Series B a similar estimate results in the values 7.4%, 8.9% and 9.2% for control tests, residual strength tests and strength tests at elevated temperatures, respectively. Accordingly, the observations of the strength at elevated temperatures have the largest dispersion, and this is reasonable because of the fact that the test procedure in this case is the most complicated.

The relative mean error μ_i in the mean value of n_i observations is obtained from the equation:

$$\mu_i = \frac{\nu_k}{\sqrt{n_i}} \tag{1}$$

where μ_i is calculated in per cent of the corresponding mean value.

In the present report the ratio between two mean values is given. According to the classical theory of errors, the relative mean error μ_f in a ratio f between two magnitudes a and b can be calculated from the formula:

$$\mu_f = \sqrt{\mu_a + \mu_b} \tag{2}$$

An application of Eqs. (1) and (2) to the test values in Series A yields a mean error of 7.1–8.5%, depending on the number of observations. The corresponding values obtained for the test values in Series B are 4.6–5.3% for strength at elevated temperatures and 4.5–4.9% for the residual strength.

The above estimations of the dispersion are not rigorously substantiated by statistical theory because Eq. (2) assumes that μ_a and μ_b are known mean errors and not mere estimations as they are here. Likewise, the assumption that the coefficient of variation in different cells is the same is somewhat uncertain. If, in spite of this, a test of significance according to the normal distribution is made for the comparison of two mean values, it is found that a difference of about 15% for Series B and about 20% for Series A is required in order to obtain a 95% level of significance. This means, for example, that if, for given temperature and concrete composition, we obtain the value 40% for strength at elevated temperatures and $40 - \frac{40 \cdot 15}{100} = 34\%$ of the strength at room temperature, for the residual strength, then we have a difference which is exactly significant at a level of 95%. These figures should be kept in mind when interpreting the following results and when estimating the effects of the different parameters.

4.3 Remarks on the split-cylinder test

The tensile strength determined by split-cylinder test is calculated from the equation:

$$\sigma_d = \frac{2P}{\pi \cdot l \cdot d} \tag{3}$$

in which

P = ultimate load
d = cylinder diameter
l = cylinder length

In order to be rigorously valid the Eq. (3) assumes elastic material and brittle fracture.

Concrete which is subjected to high temperatures with the accompanying crack formation and material destruction does not strictly satisfy these conditions. Some test results indicate that the split-cylinder test overestimates the tensile strength if this value is small. A certain split strength could be obtained even if the concrete had obviously lost its tensile strength. In spite of this, the split-cylinder test can give a good picture of the capacity of concrete to resist tensile stresses when influenced by temperature.

Experiments have shown that the tensile strength determined by split-cylinder test is in general greater than that determined by uniaxial tensile test [11]. One reason is the difficulty to materialize axial loading in a uniaxial tensile test. Another reason could be that while the plane of fracture in a split cylinder test is determined in advance, the fracture in uniaxial tension can take place in any arbitrary section along the test specimen. Thus, in a uniaxial tensile test the tensile strength of the specimen is measured in the weakest section. The latter effect is possibly greater for concrete subjected to high temperatures, a fact which should be taken into consideration when interpreting the results.

In testing the cylinders in the hot state by this procedure, a certain fall of temperature in the external layer of the cylinders cannot be avoided in spite of the fact that the time between the removal of the specimens from the oven and failure is short. Temperature measurements showed that only the external 5 mm thick layer was affected by the temperature fall. This condition implies that the external layer shrinks, resulting in tensile stresses and crack formation. These tensile stresses are balanced by compressive stresses in the core inside the external layer. These, however, are not of such an order of magnitude as to significantly affect the results obtained from the split-cylinder test. Nor have the test results given any indication of this since the ratio between the strength at elevated temperatures and the residual strength is in agreement with earlier results obtained from compressive strength tests.

4.4 Influence of temperature

In Figs. 9–12 the tensile strength determined by split-cylinder tests is shown as a function of the temperature. The results are grouped in such a way that the effects of the different parameters can be studied directly.

4.4.1 Temperature interval 0–300°C

A common feature of all these curves is that the tensile strength somewhat decreases in the vicinity of 100°C, and then remains roughly constant up to about 300°C. This also applies to the Weigler-Fischer curves in Fig. 1, in which the strength was determined on specimens which had been dried in advance. Weigler and Fischer point out that the shape of the curve for concrete in the interval 0–300°C depends markedly on age and curing conditions. If the humidity in concrete is high, the heating results in drying and hardening of the concrete and, compared with the initial value, the strength increases. On the other hand, if the concrete has been dried prior to heating the increase of strength has already taken place and thus heating results in a relative decrease in strength. In the tests reported in this paper, the specimens were cured in air prior to heating and consequently were rather dry.

Besides, the split-cylinder method will possibly involve an influence due to non-uniform drying. The fact that the external layer of the cylinder is drier than the core gives rise to shrinkage stresses which comprise tensile stresses in the external layer and compressive stresses in the core. These stresses can result in an overestimation of the strength of the control specimens. On heating the specimens, they get dried and the stresses either decrease or vanish.

A further possible explanation of the decrease in strength is that the incompatibility between cement paste and aggregate causes crack formation already at moderate temperatures. Obviously this effect on the tensile strength is more pronounced than that on the compressive strength.

While the above mentioned factors result in a decrease in strength, the removal of water causes a stronger setting of the cement paste which maintains the overall strength at a relatively high level.

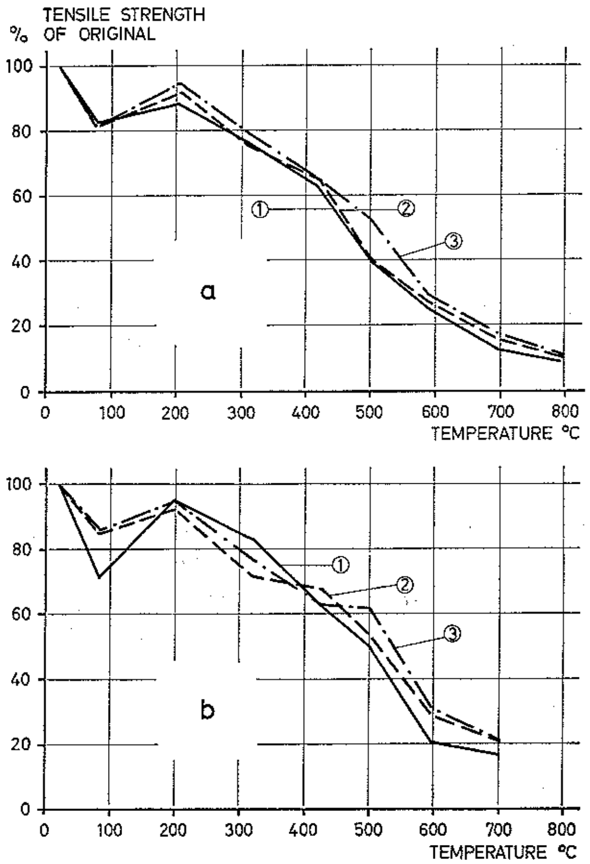


Fig. 9. Effect of temperature on split-cylinder tensile strength. Curve 1: No insulation. Curve 2: 5 mm diabase wool insulation. Curve 3: 10 mm diabase wool insulation. a) Series A_1, concrete mixture A:I. b) Series A_2, concrete mixture A:II ● *Spräckdraghållfasthetens variation med temperaturen. Kurva 1: Utan isolering. Kurva 2: Med 5 mm mineralullsisolering. Kurva 3: Med 10 mm mineralullsisolering. a) Serie A_1, betongblandning A:I. b) Serie A_2, betongblandning A:II.*

4.4.2 Temperature interval 300–600°C

When the temperature is increased from 300°C to 600°C the tensile strength decreases from about 80% to 20 or 30% of the strength before heating. The cause of this considerable deterioration is a combined effect of several mechanisms: dehydration of the clinkers in the cement paste, especially calcium hydroxide, thermal incompatibility between cement paste and aggregate, and transformation of quartz at approximately 575°C. These phenomena are described in Section 2.

4.4.3 Temperature interval 600–800°C

Raising the temperature above 600°C results in a further deterioration of the strength. At these temperatures concrete practically loses all its tensile strength.

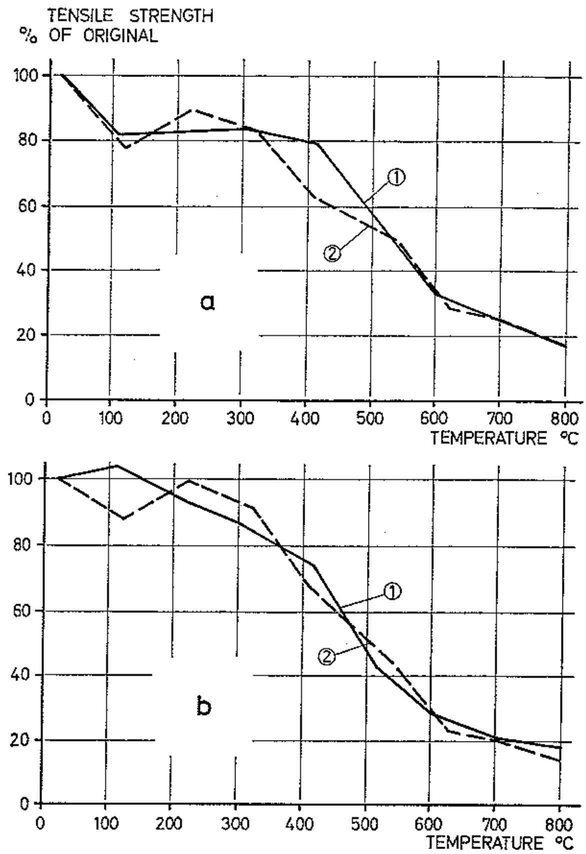


Fig. 10. Effect of temperature on split-cylinder tensile strength. Concrete mixture B:II. Curve 1: "Slow" heating. Curve 2: "Rapid" heating. a) Strength at elevated temperature. b) Residual strength ● *Spräckdraghållfasthetens variation med temperaturen. Betongblandning B:II. Kurva 1: "Långsam" uppvärmning. Kurva 2: "Snabb" uppvärmning. a) Varmhållfasthet. b) Resthållfasthet.*

As mentioned in Section 4.3, the split-cylinder test seems to overestimate the tensile strength, if the tensile strength has a low magnitude.

4.5 Influence of the rate of heating

In Fig. 9 the influence of the rate of heating is shown for the tests in Series A, where the rate of heating was made to vary by covering some of the specimens with diabase wool insulation of varying thickness. In Fig. 9 the strength is compared for specimens without insulation and those insulated with 5 mm and 10 mm thick diabase wool layer during heating and cooling. It is to be observed that the influence of insulation is generally very small. In view of the high dispersion in these experiments, (see Section 4.2), it cannot be claimed that the differences are significant. The relations between the measured values are however as expected over the whole temperature range.

This indicates that the insulation, which here controls the rate of heating, can have a certain influence, though it is slight.

In Fig. 10 the influence of the rate of heating is shown for tests in Series B with a comparison between "rapid" and "slow" heating as defined in Section 3.3. The diagrams refer to type B:II concrete mixture in Table 3 and the comparison is made both for tensile strength at elevated temperatures (Fig. 10a) and for residual tensile strength (Fig. 10b). In spite of the more marked difference in rate of heating, an obvious tendency cannot be discerned here either. Only at a temperature of 400°C in Fig. 10a do we find a difference which is significant according to Section 4.2. In this single case the reason is rather the difference in the length of heating time than in the heating rate. During "rapid" heating the specimens are exposed to the temperature 400°C for a longer time than that for "slow" heating. This can especially be critical at 400°C since dehydration of calcium hydroxide starts approximately at this temperature. The degree of dehydration depends then on the time during which concrete is subjected to heating.

To sum up, it can be established that the rate of heating has very little or no effect on the tensile strength of concrete. Thus the stresses due to non-uniform temperature distribution seem to have no decisive influence. This effect is probably negligible compared with the structural deterioration emanating from thermal incompatibility of cement paste and aggregate.

4.6 The influence of concrete composition

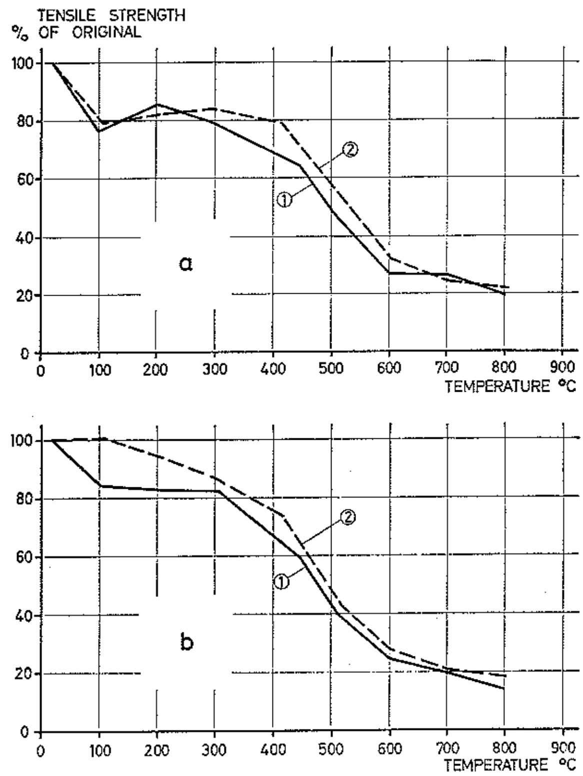
In Fig. 11a and b the influence of concrete mix design is shown for tests performed in Series B. Both mixtures B:I and B:II with cement-aggregate ratios of 1:7 and 1:4.65, respectively, are compared.

The diagrams show that the relative reduction in strength is somewhat higher for concrete mixture B:I, namely, for concrete of lower quality. The corresponding comparison between the mixtures A:I and A:II gives the same result. The difference, however, is small and hardly significant in view of the dispersion. This agrees with the results of Abrams [3], who found that the initial strength had very little influence on the relative compressive strength at elevated temperatures.

4.7 Residual tensile strength versus tensile strength at elevated temperatures

In Fig. 12a and 12b the tensile strength at elevated temperatures and the residual strength are compared for concrete mixtures B:I (Fig. 12a) and B:II (Fig. 12b).

Fig. 11. Effect of temperature on split-cylinder tensile strength when the "slow" heating process is used. Curve 1: Concrete mixture B:I cube strength 18.7–24.3 $MN \cdot m^{-2}$. Curve 2: Concrete mixture B:II cube strength 34.8–47.0 $MN \cdot m^{-2}$. a) Strength at elevated temperatures. b) Residual strength ● *Spräckdraghållfasthetens variation med temperaturen vid "långsam" uppvärmning. Kurva 1: Blandning B:I med kubhållfasthet 18,7–24,3 MN/m^2. Kurva 2: Blandning B:II med kubhållfasthet 34,8–47,0 MN/m^2. a) Varmhållfasthet. b) Resthållfasthet.*



It can be observed that the residual strength is as a rule somewhat lower than the strength at elevated temperatures. The difference is most obvious at temperatures above 400°C. A similar, yet more pronounced difference for the compressive strength has been observed by Malhotra [1] and Abrams [3], see Fig. 2.

The difference between the strength in the hot state and the strength immediately after cooling may be due to the further mechanical influence taking place under cooling. Furthermore, as mentioned in Section 4.3, the unintensional thermal stresses may cause some overestimation of the strength in the hot state.

4.8 Change of strength in the period after heating

In table 4 the relative tensile strength is shown for specimens heated to 400°, 600° and 800°C, respectively, in the hot state, immediately after cooling and 7

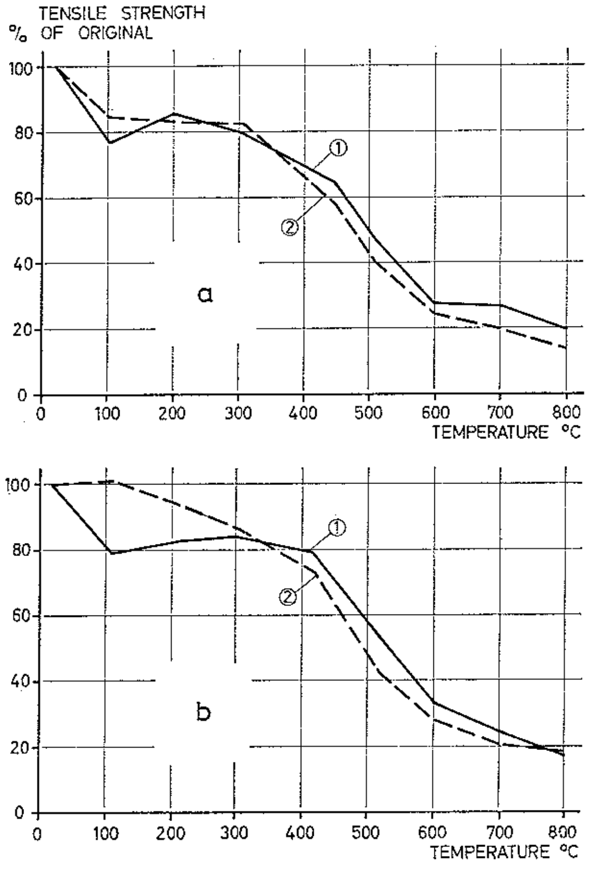


Fig. 12. Effect of temperature on split-cylinder tensile strength. "Slow" heating. Curve 1: Strength at elevated temperatures. Curve 2: Residual strength. a) Mixture B:I. b) Mixture B:II ● *Spräckdraghållfasthetens variation med temperaturen vid "långsam" uppvärmning. Kurva 1: Varmhållfasthet. Kurva 2: Resthållfasthet. a) Blandning B:I. b) Blandning B:II.*

and 28 days after the day of heating. It can be observed that the strength decreases significantly during the subsequent period of air curing. Specimens heated to 600°C and 800°C lose their strength completely and even at 400°C

Table 4. Effect of subsequent curing in air on split-cylinder tensile strength of concrete which has been heated to different temperatures. Strength is given in per cent of the original value.

Test temperature °C	400	600	800
Strength at elevated temperatures	78.9	33.0	21.2
Immediately after cooling to 20°C	71.6	28.0	18.3
After 7 days with 20°C and 65% rel. humidity	57.4	7.2	0
After 28 days with 20°C and 65% rel. humidity	52.3	0	0

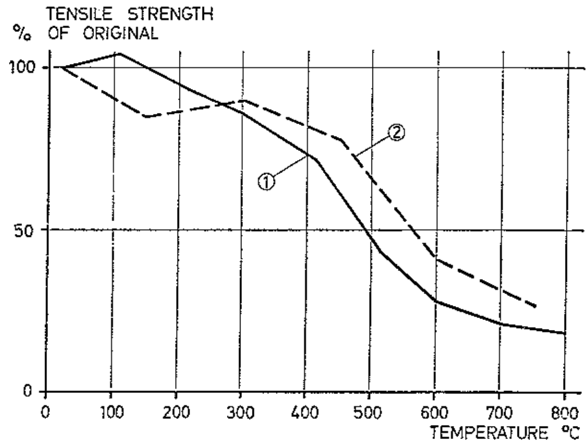


Fig. 13. Comparison between tensile strength and compressive strength. Curve 1: Split-cylinder strength. Curve 2: Compressive strength [2] ● *Jämförelse mellan drag- och tryckhållfasthet. Kurva 1: Spräckdraghållfasthet. Kurva 2: Tryckhållfasthet* [2].

there is a significant decrease in strength. Qualitatively this agrees with similar tests performed by Weigler and Fischer, see Section 2.1.

It is obvious that we cannot simply use the residual strength determined immediately after cooling to estimate the residual load-bearing capacity of concrete constructions under a longer period of time after they have been subjected to fire. One could probably, with reference to Weigler-Fischer's results [2], increase or maintain the residual strength in fire-exposed concrete structures by sprinkling them with water some time after the occurrence of fire. Further experiments are however required to prove this fact.

4.9 Comparison between temperature effects on tensile and compressive strength

In Fig. 13 the relative tensile strength has been compared with the relative compressive strength according to Weigler-Fischer [2]. The values of tensile strength have been taken from Series B_1 with concrete mixture B:II and "slow" heating. The compressive strength is determined for concrete with nearly the same mixture design (cement-aggregate ratio = 1:5.13 and water-cement ratio = 0.6) but at an age of 84 days. In both cases rocks containing quartz are used as aggregate. Apart from the differences in age, the two test series are quite comparable.

As is seen from Fig. 13, the decrease in tensile strength is relatively larger than that of the compressive strength. This is quite natural, since crack formation and decreased adhesion between cement and aggregate must affect the tensile strength more. The difference in age presumably implies that the difference in relative strengths appears to be smaller than it should be. The higher age of the specimens used for compression tests probably causes the whole curve to be displaced downwards.

5. Conclusions

From the test results presented in Section 4, the following brief conclusions can be drawn:

(1) Tensile strength of concrete decreases with increased temperature, with the fastest deterioration in the interval 300–600°C. At 600°C the strength amounts to 20 or 30% of the original strength.

(2) The decrease in the tensile strength of concrete is on the whole independent of the rate of heating, that is, the influence of non-uniform temperature distribution on *the strength* is negligible. *The stresses,* on the other hand, are of course dependent on the temperature distribution.

(3) The relative decrease of the tensile strength at elevated temperatures is somewhat less in concrete of higher quality, where the percentage of cement is larger than that in concrete of lower quality. The difference, however, is of little practical consequence. This is true only if the petrographical composition of the aggregate is not changed.

(4) The residual tensile strength immediately after cooling is somewhat lower than the strength in the hot state.

(5) The residual tensile strength decreases with time after heating if the specimens are cured in the air. Thus, the values of the residual strength determined immediately after cooling should not be used to estimate the residual load-bearing capacity of concrete structures exposed to fire.

(6) With comparable concrete compositions, the relative decrease in concrete strength with temperature is greater for the tensile strength than for the compressive strength.

Sammanfattning

De senaste årens utveckling inom brandforskningen pekar mot en mera funktionellt baserad brandteknisk dimensionering. Detta innebär att man på basis av grundläggande karakteristika för brandrummet och dess brandbelastning bestämmer brandens tid-temperaturkurva, som sedan får ligga till grund för bestämning av temperaturfälten i den aktuella konstruktionen. Med utgångspunkt från dessa temperaturfält skall sedan konstruktionens verkningssätt och bärförmåga analyseras.

För armerade betongbalkar och betongplattor innebär detta att temperaturen i armeringsjärnen beräknas och att därefter brottlasten bestämmes med kännedom om armeringsstålets egenskaper vid höga temperaturer. Det är sålunda i dag möjligt att systematiskt beräkna armerade betongbalkars brandmotstånd med hänsyn till böjbrott.

Parallellt med detta måste man emellertid ha något begrepp om hur andra typer av brott såsom skjuv- och vridbrott påverkas vid brandpåverkan. För en bedömning av detta är kännedom om betongens draghållfasthet vid höga temperaturer av stor betydelse.

Likaså är det nödvändigt att känna draghållfastheten vid beräkning av spänningar till följd av ojämn temperaturfördelning.

I denna undersökning bestämdes betongens draghållfasthet genom spräckprov i temperaturintervallet 20–800°C. Förutom temperatur varierades uppvärmningshastighet och betongsammansättning. Provningen skedde dels i varmt tillstånd och dels efter avsvalning till rumstemperatur. Några försök gjordes också för att belysa hur hållfastheten förändras under tiden efter uppvärmningstillfället.

Resultaten framgår av figurerna 9–13 samt tabell 4. Draghållfastheten minskar i stort sett med ökande temperatur, med den starkaste reduktionen i intervallet 300–600°C. Hållfastheten är i stort oberoende av uppvärmningshastigheten och procentuellt sett endast svagt betoende av ursprunglig hållfasthet. Vidare uppmättes i allmänhet lägre värden i avsvalnat tillstånd än i varmt tillstånd och vid lagring i 20°C och 65% r.f. under tiden efter uppvärmningstillfället försämrades hållfastheten ytterligare (tabell 4). Den relativa hållfasthetsreduktionen är vid jämförbara betongsammansättningar större för draghållfastheten än för tryckhållfastheten.

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