HETEK

Control of Early Age Cracking in Concrete
State of the Art

Report No.52
1996

Road Directorate

IRRD Information

Title in English	HETEK -Control of Early Age Cracking in Concrete - State of The Art
Title in Danish	HETEK - Styring af revner i ung beton - Status for emnet
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Subject classification	Field 32 Concrete

Key words

Analysis	7152
Cracking	5211
Material (constr)	4555
Numerical	6432
Properties	5925
Simulation	9103
Thermal stress in material	5575

Abstract

This report forms a part of the Danish Road Directorate's research programme called High Performance Concrete - The Contractor's Technology (abbreviated to HETEK). This report describes the "State-of-The-Art" on the subject: Control of Early Age Cracking. Due to thermal strains and shrinkage during the hardening process of concrete, cracks can be formed. By means of stress calculations a method of construction not resulting in cracks can be chosen.

UDK 625.84
625.891.8

ISSN 0909-4288

ISBN 87-7491-733-1

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0. Preface

This project on control of early age cracking is part of the Danish Road Directorate's research programme, High Performance Concrete - The Contractor's Technology, [1] abbreviated to HETEK.

In this programme high performance concrete is defined as concrete with a service life in excess of 100 years in an aggressive environment.

The research programme includes investigations concerning the contractor's design of high performance concrete and execution of the concrete work with reference to the required service life of 100 years.

The total HETEK research programme is divided into segments parts with the following topics:

- chloride penetration
- frost resistance
- control of early-age cracking
- compaction
- curing (evaporation protection)
- trial casting
- repair of defects.

The Danish Road Directorate invited tenders for this research programme which is mainly financed by the Danish Ministry for Commerce and Industry - The Commission of Research and Development Contracts.

The present report refers to the part of the HETEK project which deals with control of early age cracking.

For durability reasons reinforced structural members should be well protected against penetration of water, chloride etc. This means that cracks should be avoided or at least the crack-width limited. Formation of cracks can take place already during the hardening process. An evaluation of the risk of crack formation involves a stress analysis. In stress analysis of hardening concrete structures, the load consists of the differences in thermal strains that arise from the heat of hydration. The mechanical properties (including autogenous shrinkage) of the concrete also change during the hardening process. If a stress analysis shows high stresses compared to the tensile strength there is a high risk of crack formation.

The purpose of this project is to investigate these effects and to prepare a guideline regarding Control of Early Age Cracking.

In Danish: Højkvalitetsbeton - Entreprenørens Teknologi

The project was carried out by a consortium consisting of:

Danish Concrete Institute represented by:

Højgaard & Schultz A/S
Monberg & Thorsen A/S
RAMBØLL
COWI

and

Danish Technological Institute, represented by the Concrete Centre

and

Technical University of Denmark, represented by the Department of Structural Engineering and Materials.

Two external consultants, professor Per Freiesleben Hansen and manager Jens Frandsen, are connected with the consortium.

1. Scope

In order to avoid durability problems in reinforced concrete structures it is intended to limit the formation of cracks during the hardening period. Cracks can of course be caused by external loads, settlements, etc., but there is also a risk of crack formation during the hardening process. The main cause of cracks in this period is thermal strains arising from the heat development of the concrete. Thermal strains have traditionally been limited by requiring the contractor to plan the concrete work so that large temperature differences between different parts of the structure do not arise. Strains arising from shrinkage play a secondary but significant part in the concretes normally used in Denmark. The shrinkage can be divided into two parts: shrinkage arising from evaporation and autogenous shrinkage. The strains arising from autogenous shrinkage cannot be affected by the contractor except through the choice of concrete mix design, but shrinkage arising from evaporation can be avoided by making an appropriate evaporation protection. How to handle this protection is reported in another part of the HETEK-project, "Curing", and in the present report stresses are only assessed on the basis of strains from temperature and autogenous shrinkage. The influence of cracks on durability is assumed to be dealt with in the segments of the HETEK-project which concern frost-resistance and chloride penetration.

2. Introduction

Due to the strains from temperature and autogenous shrinkage a 3-dimensional stress-state is generated in hardening concrete structures. This stress-state is traditionally considered as arising from two sources:

- stress due to different displacements between adjacent structural components
- stress due to internal strain-profiles

These are described in the following sections.

2.1 Stress due to different displacements between adjacent structures

Stress arising from different displacements between adjacent structures is shown in Figure 1 a-d. A concrete specimen is cast between two structures. At first the concrete is a liquid, but hardening takes place as a result of the chemical reaction between cement and water (Figure 1c). At the same time heat is generated and this results in a temperature rise (Figure 1b). Due to the rise in temperature the concrete expands, but the expansion is restrained to a certain extent by the adjacent structures. The restraint transforms the expansion into an axial compression as shown in Figure 1d.

When the temperature begins to fall (and autogenous shrinkage develops), the contraction generates tensile stresses that are much higher than the compressive stresses in the heating period. This is because the stiffness is much greater during the cooling than in the heating period. As a result of the hardening process the structure remains in a tensile stress-state.

The stress level obviously depends on:

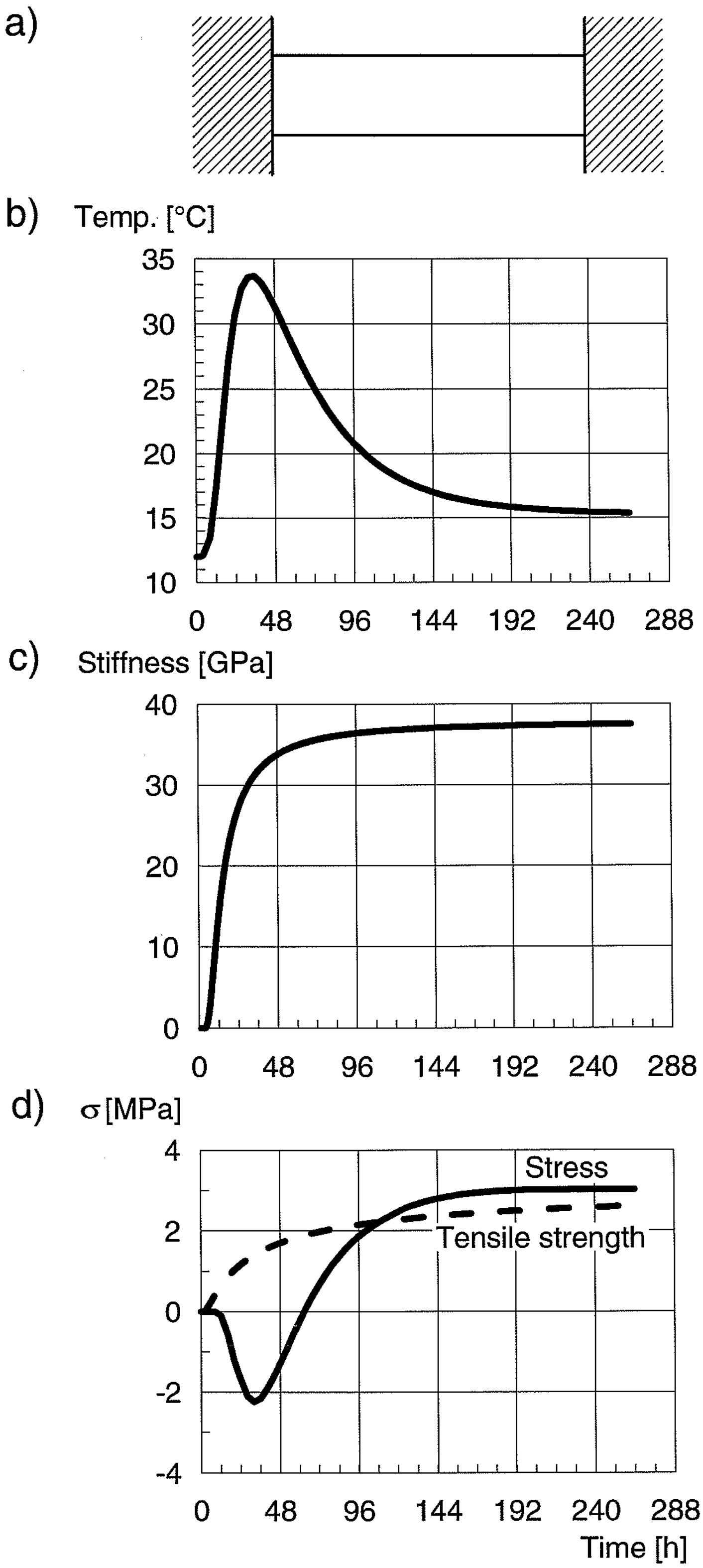
- the temperature change
- the changes in concrete properties (including autogenous shrinkage)
- the stiffness of adjacent structural components

If the stresses exceed the tensile strength, which is also a function of the degree of hydration (Figure 1d), cracks will be formed. The crack-width is determined by

- the temperature change and the magnitude of the shrinkage
- the amount of reinforcement
- the stiffness-ratio between the cracked structure and the adjacent structural components

Cracks formed in the cooling period will often pass through the structure and remain open. This type of crack will make costly repair necessary or will be a threat to durability.

Figure 1: Development of stresses in concrete cast between two stiff structures.



2.2 Stress due to internal strain-profiles

Stresses are also generated within the cross-section. Figure 2 shows a temperature distribution at the moment when the maximum temperature is attained. Usually there is a loss of energy to the surroundings. The difference in thermal strain between the surface and the core induces stresses acting in the plane of the cross-section, as shown in Figure 3. If the temperature difference gets too high, cracks will form at the surface due to the expansion of the central part of the cross-section. These cracks will close again when the temperature profile has become level. The tension at the surfaces will even change to compression when the hardening process is completed (see Figure 4). This is because the highest thermal contraction takes place at the core of the cross-section.

The stress level clearly depends on:

- the temperature profile
- changes in concrete properties (including autogenous shrinkage)
- the geometry of the cross-section.

Normally surface-cracks tend to close again and become invisible, but they will probably result in a reduced capability of the surface layer to protect against penetration of water and chlorides.

In practice, both types of stress are generated in the same structure but at different times. Surface cracks are generated in the heating phase or at the time at which formwork is removed, while through-cracks are generated in the cooling phase.

The only parameter that can be controlled by the contractor on site is the temperature. In Denmark the normal limit is a maximum difference of 15° C between average temperatures in adjacent segments and 20° C within a cross-section according to DS411. The 20 ° C limit is based on full-scale tests in 1978 at Storebælt [BKF-Centralen, 1979]. The temperature requirements do not cover the effects of stiffness-ratios, material properties, etc. on the stresses, and consequently the temperature requirements have to be conservative.

Figure 2: Temperature distribution when temperature is a maximum.

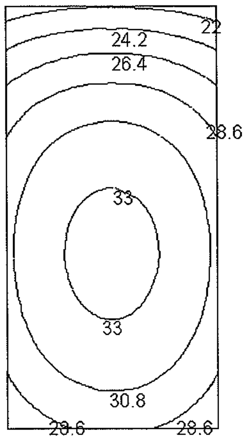


Figure 3: Vertical stress component when temperature is a maximum.

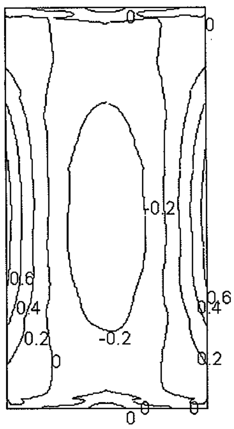
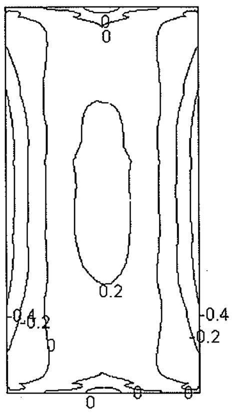


Figure 4: Vertical stress component after hardening.



3. Basis for stress assessment

A computational assessment of the stresses in a hardening concrete structure involves:

- A transient thermal analysis which determines the temperature history. The temperature enters the stress calculation as a "load" consisting of thermal strains and at the same time it determines the development of material properties.

- A transient stress analysis that takes into account transient material properties incl. autogenous shrinkage, the thermal history, statical boundary conditions, etc.

- A mathematical description of the material behaviour as a function of degree of hydration, temperature, stress, etc.

The development of the material properties and stresses also depends on the moisture content and moisture transport in the concrete. Also deformations due to changes in moisture content influence the stresses. In sealed concrete structures differences in moisture content across the section arise due to the temperature profile. However, these differences are normally considered small and their influence on the stresses is neglected.

3.1 Thermal analysis

The temperature distribution and its variation with time determine:

- The thermal strains
- The rates of change of mechanical properties

in a hardening concrete structure.

The temperature history is normally determined by a previous analysis, after which the result is used for the stress calculation. The temperature is determined by solving the heat conduction equation. The equation includes:

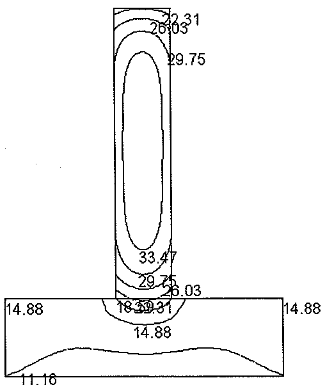
- Heat of hydration
- Heat capacity, conductivity, density
- Thermal boundary conditions in the form of convection, radiation, cooling pipes, heating cables, etc.

Computer programmes are normally used for the numerical solution, as both the transient states and a 2- or 3-dimensional temperature field can be specified. In principle, a transient analysis is a series of stationary analyses for which the initial condition in a given time-interval is the result of the calculation for the previous time-interval.

As the heat of hydration and the radiation boundary conditions make the problem non-linear, an iteration procedure is necessary for the individual time-intervals. However, calculation of a temperature field is much less resource-demanding than stress calculations.

Often determination of 1- or 2-dimensional temperature distributions are sufficient (Figure 5). This type of evaluation can be carried out by the computer programmes mentioned in Sect. 3.2 as well as by many others. In these programmes it must be assumed that transport of moisture to the surroundings is prevented, as moisture profiles are not taken into account in the determination of the temperatures.

Figure 5: Wall cast on hardened foundation. 2-dimensional temperature distribution.



3.2 Stress analysis

The stress analysis must be incremental as the stress is generated by a non-stationary (transient) temperature strain that depends on the temperature- and stress-history and material properties that depend on the temperature-history (Figure 1d),. This means that the hardening period is divided into a number of time-intervals, and it is assumed that the properties of the concrete remain constant within each time-interval. The incremental analysis is then carried out by a stationary analysis for each time-interval. The initial condition of the analysis for each interval is the result of the analysis for the previous interval. With the starting point given in Sect. 3.3, expressions for the strain contributions of:

- Autogenous shrinkage
- Thermal expansion/contraction
- Creep
- Crack formation

can be set up. These contributions create a state of eigen-strain in the structure. As the thermal expansion/contraction as well as creep and crack formation depend on the stress, the analysis for each time-interval must also be iterative.

The large amount of computation (many time-intervals, iteration for each interval) requires the use of computer programmes. A number of commercial programmes are capable of carrying out the transient stress calculation taking the temperature- and property-history into account.

Depending on the programme, a one-, two- or three-dimensional analysis can be carried out. Three-dimensional analyses can of course describe any stress-field, but require long computation times. This can be done by use of e.g. FIESTA [Grodtkjær, 1993]. The stress-state can often be described in a simpler way by means of approximations.

A classical casting problem, in which a wall is cast on a hardened foundation, is shown in Figure 6. Cracks extending through the wall can be observed; these are due to differences in temperature movements and autogenous shrinkage, cf. Sect. 2.1. Studies based on 3-dimensional analyses show that the stress-state does not change from cross-section to cross-section, when the area between the end-zones is consider-ed. As the normal stress in the end-zone cross-sections is zero, the normal stresses in the intermediate zone are converted to shear stresses in the end-zones (Figure 7).

Figure 6: A classical casting problem.

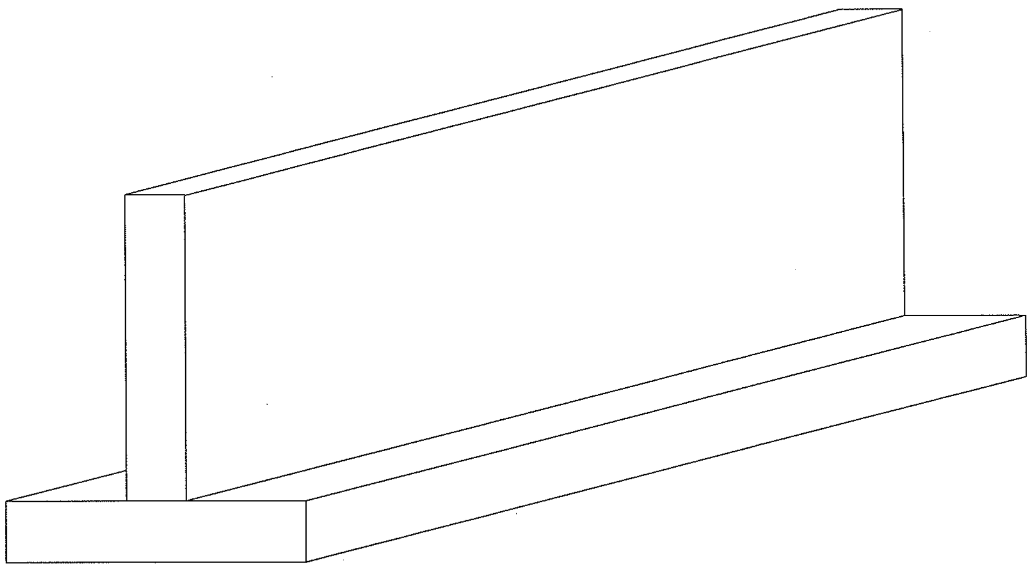
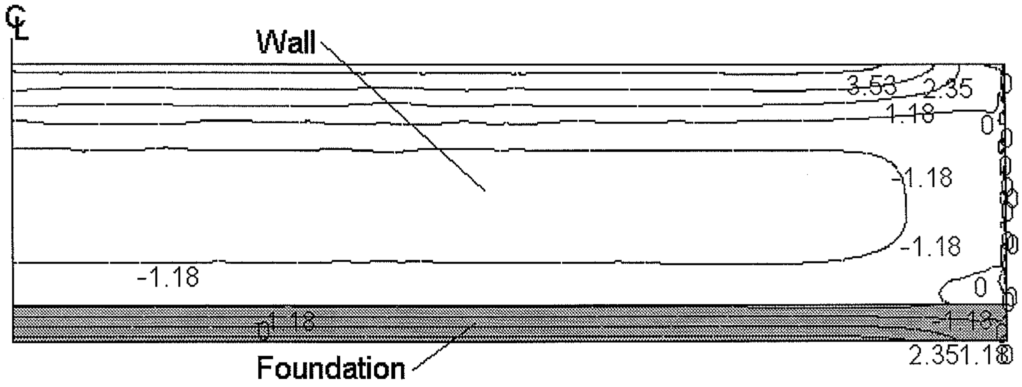


Figure 7: Iso-curves for the longitudinal stress component, σ_z



In the intermediate zone, where there are no shear deformations, it can be assumed that plane sections remain plane. The normal stress distribution across the section can then be determined by a transient section-analysis involving eigen-strains and material properties, as mentioned above. This method is used i.a. in HEAT [Roelfstra, 1994] and CIMS-2D [Pedersen, 1995], and is known as the "Compensation Plane Method", which has been described by e.g. T. Tanabe [Tanabe, 1992].

In the sections under consideration there are also stresses in the plane of the section arising from temperature differences between the core and the surface (cf. Sect. 2.2), and due to the transverse contraction of the concrete (Poisson's ratio) there is a connection between these stresses and the normal stress σ_z. This method gives the same result as a full 3-dimensional analysis when "the intermediate zone" is considered. The method is described in [Pedersen, 1994] and is used in CIMS-2D.

In short structures, where there is no "intermediate zone" without shear strains, the assumption that "plane sections remain plane" will result in an overestimation of the stress. In this situation a 3-dimensional analysis, that takes the load-relieving effect of shear-strain on the eigen-stress-state into account, must be carried out. An approximate calculation can, however, be made using 2-dimensional analysis. On the basis of a thermal calculation of the cross-section in Figure 6, the course of mean temperature for each ordinate can be determined. A 2-dimensional stress analysis of the wall slab is then carried out.

The calculated mean temperatures enter into the analysis, partly in the above-mentioned strains and partly in the rates of change of material properties. This method is "correct" when the temperature variation across the wall thickness is minimal. It has not been shown that the method is usable when the temperature profile has variations due to cooling pipes, heating cables, cooling of surfaces, etc. This approximate me- thod can be used in programmes which can perform 2-dimensional analysis e.g. HACON [Nordström, 1995] and CIMS-2D.

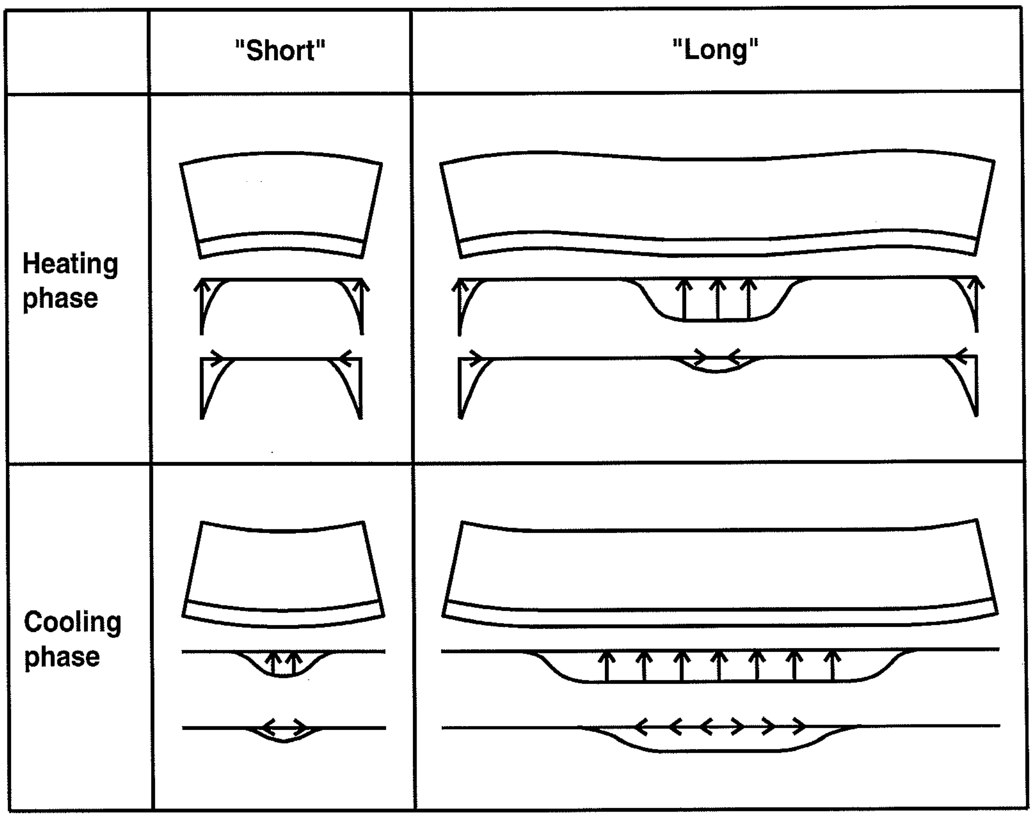
3.2.1 Statical boundary conditions

A transient stress calculation corresponds in principle to a series of statical calculations.

In a statical calculation the supports are very important. This also applies to the calculation of stresses in hardened concrete structures, cf. [Pedersen, 1995] and "Sprickor i betongkonstruktioner" [Svenska Betongföreningen, 1994]. As concrete is a brittle material, especially in the cooling phase, even small movements of the supports can cause cracking. The modelling of the type and location of the supports is most difficult for structures with a large surface in contact with the soil, e.g. tunnels, retaining walls, long foundations.

Figure 8 shows a number of schematic reaction distributions in the heating and cooling phases of "short" and "long" structures. In the "long" structures a stretch in the middle of the structure retains a linear geometry due to the dead load. The reaction distributions shown (vertical and horizontal) are difficult to calculate. A 3-dimensional FEM-analysis is needed, in which the underlying soil volume is taken into account, e.g. as a friction material with a given modulus of elasticity, cohesion, angle of friction, etc.

Figure 8: Schematic deformations and reaction distribution for wall cast on foundation.



As it is very complicated to carry out such calculations, assumptions on the safe side are often made in practical applications. The structure shown in Figure 6 can be modelled e.g. based on each of two assumptions:

- Curvature is permitted
- Curvature is not permitted.

Both situations arise in "long" structures, cf. Figure 8, while "short" structures do not have a fixed curvature, as the dead load is less important. It is, however, a problem to determine whether a structure is "short" or "long". According to "Sprickor i betongkonstruktioner" [Svenska Betongföreningen, 1994] and "Betonghandbok" [AB Svensk Byggtjänst och Cementa AB, 1994], the interaction with the sub-base has not been clarified, but is of great importance for the development of stress.

3.2.2 Evaluation of cracking risk/crack widths

The risk of cracking is determined in practice as the ratio of the maximum principal tensile stress to the tensile strength of the concrete. If the cracking is to be prevented, the uni-axial tensile strength of the concrete should be used in the evaluation. The tensile stress/strain curve of the concrete at normal loading rates is usually assumed to be linear until failure occurs. At low loading rate non-linearity is observed; this is ascribed to creep. A high degree of utilization of tensile strength and a low loading rate can result in tensile failure [Emborg, 1989]. This situation often arises during the cooling phase (Figure 1d).

This effect is taken into account in [Huckfeldt, 1994] by introducing an "equivalent tensile age"; this enables a tensile strength dependent on the stress history to be determined on the basis of experimental data. The fact that the eigen-stress state is relieved when creep strains increase is not taken into account. The redistribution of stress can result in the absence of crack formation. This can be taken into account only by means of a creep model that describes the tensile stress situation up to the failure point.

Methods for evaluating the stresses that arise after cracks have formed can be found in the literature. They are based on stress-crack width correlations in connection with "smeared approach", e.g. [Dahlblom, 1990]. When cracks are permitted, the requirement for utilization of tensile strength is replaced by a requirement for crack-width. However, it is not feasible to make a computational evaluation of the widths of the cracks that will arise in a structure such as that shown in Figure 6. Such an evaluation requires (in addition to the requirements for an uncracked structure):

- Knowledge of the relation between stress-crack width and degree of hydration
- Taking into account the reinforcement and its bond with the concrete as a function of the degree of hydration
- A 3-dimensional analysis to determine the distance between cracks and thus the crack-widths.

Crack-widths can be calculated only with a wide margin of uncertainty even in hardened reinforced concrete structures. DIN 1045 indicates how crack-widths in hardened concrete can be determined, taking the concrete development into account.

But as in all crack-width formula, the stress in the reinforcement must be known. The reinforcement stress depends on the temperature history of the structure and on the stress redistribution that occurs when cracks form. The literature contains only a few attempts [Mishima, 1994] to incorporate the influence of the reinforcement on the evaluation of hardening stresses and crack-widths.

3.3 Material models

The behaviour of the concrete can be described by considering a number of distinct properties, determined independently by prediction and/or testing. When these properties are "put together" in a calculation, the behaviour of any concrete structure can be described in principle. A correct description of the properties and any mutual correlations they may have is very complicated. The description must therefore be simplified but still sufficiently detailed to achieve a reasonable degree of accuracy without a large amount of calculation and testing.

As the concrete develops from a plastic to a fully hardened material, its thermal and mechanical properties change. In order to calculate the stresses in a hardened concrete structure, the properties at every stage of the hardening period must be known (cf. Sect. 3.2). The relevant properties are reviewed in the following Sections.

3.3.1 Maturity/Degree of hydration

As the chemical reaction between cement, mineral additives and water proceeds, the properties of the concrete develop. As the temperature rises, the rate of the reaction rises too. However, the temperature does not develop equally across the cross-section which means that the properties develop at different rates in different locations. This, of course, has consequences for the build-up of stresses as a result of the hardening, and thus for the risk of cracking. To apply the correct value of a property at any moment and at any point in a hardening structure, it is necessary to define a parameter that indicates how far the chemical reaction - the hydration - has proceeded.

According to [Breugel, 1994] a comprehensive and in all respects satisfactory definition of the degree of hydration is hard to give as the different compounds of the cement (and mineral additives) hydrate at different rates. As the reaction proceeds, heat is developed. The cumulative heat developed up to a given moment as a fraction of the total heat of hydration is an indicator of the degree of hydration [Breugel, 1994]. This definition can be used in describing the development of the properties during the hardening process (e.g. [Kishi, 1994]).

Several researchers have used the Arrhenius function to describe the hydration rate as a function of the temperature [Freiesleben Hansen, 1978], [Wang, 1994], [Roelfstra, 1994] and [Emborg, 1989]. The maturity can be introduced on the basis of this hydration rate. The maturity corresponds to the equivalent age if the process had proceeded at a constant temperature of 20° C. In Denmark the Arrhenius function and with it the maturity are based on compressive strength while [Roelfstra, 1994] uses the heat development as the basis for the maturity. When the properties are determined as functions of the maturity, the degree of hydration and its dependence on temperature are thus taken into account indirectly. In this way the development of the various properties can be compared.

It should be emphasized that it has not been proved that a single maturity-function is able to describe development-rates for different properties as compression strength, E-modulus, autogenous shrinkage etc.

The maturity does not take into account that high temperatures may change the micro-structure of the concrete and as a consequence reduce the final strength.

As the maturity and the degree of hydration are mutually transformable via the heat development, there is no difference in principle between the two methods of describing the development of properties.

3.3.2 Heat of hydration

As mentioned above, concrete develops heat as a result of the hydration process. The heat development is not only significant for the degree of hydration and thus for the development of the concrete properties, cf. Sect. 3.3.1, but also for the temperature history of a given concrete structure.

The basis for calculating the temperature history and the temperature distribution in a hardening concrete structure is knowledge of the heat development measured or calculated under adiabatic conditions, i.e. without heat transfer to or from the surroundings.

Koenders and van Breugel [Koenders, 1994] calculate the heat development on the basis of micro-structure models. A "prediction" of the heat development is extremely difficult, as it is necessary to take into account the significance of the cement type, mineral additives (silica fume and fly-ash) and other additives (e.g. plastifying agents).

In experimental determinations of heat development, a measured temperature rise is transformed into a function of the maturity (or degree of hydration). The measured heat development process is frequently approximated by means of regression, so that the heat development can be described by few parameters.

3.3.3 Specific heat capacity

The specific heat capacity is defined as the amount of thermal energy required to raise the temperature of one kg of the material by one degree Celsius. It can usually be estimated on the basis of the composition of the concrete.

The heat capacity varies little with the type of cement and other component materials. It declines, however, with decreasing water content. As water is bound in the chemical reactions of the hardening process, the heat capacity will decline as the concrete matures. According to [Freiesleben Hansen, 1978] the heat capacity declines by 10 - 20%, while [Breugel, 1980] mentions a decline of 5 - 8%. The heat capacity rises slightly with rising temperatures [Freiesleben Hansen, 1978].

Typical values of the specific heat capacity are in the range 0.8 - 1.2 kJ/kg/° C.

3.3.4 Thermal conductivity

The thermal conductivity is an indicator of how well a material transmits heat. The conductivity is a function of the concrete composition, maturity, moisture content and temperature. [Maréchal, 1972] and [Sassedateljew, 1970] observed an increase in thermal conductivity at the start of the hardening period, while [Hundt, 1978] and [Brown, 1970] observed a decline in the course of hardening. Typical values of the thermal conductivity are in the range 5.8 - 9.0 kJ/m/h/° C [Emborg, 1989].

3.3.5 Coefficient of thermal expansion

Laboratory measurements of the coefficient of thermal expansion at an early age are difficult to carry out, as it is difficult to separate thermal movement and shrinkage (cf. Sect. 3.3.8). The measured value of the coefficient will depend on how this is done, but the problem is seldom treated in the literature, which makes an evaluation of the results difficult.

A number of measurements indicate, however, that the coefficient declines rapidly at a very early age, from approx. $20 \cdot 10^{-6}$ per °C after 5-8 hours to approx. $10 \cdot 10^{-6}$ per °C during the following 4-12 hours according to [Nolting, 1989] and [LaPlante, 1994]. As described in [Pedersen, 1995], this is of great significance for the risk of cracking when a segment of a concrete structure is cast against another.

For hardened concrete, tests carried out at temperatures in the range 20 - 800° C show that the coefficient of thermal expansion is a function of the stress level [Thelandersson, 1983 and 1987]. If this also applies at an early age it will affect the temperature movements and the risk of cracking. However, this has not been verified experimentally [Emborg, 1989].

3.3.6 Modulus of elasticity

The modulus of elasticity increases as the hardening process proceeds. In the literature the modulus of elasticity is frequently expressed as a function of the compressive strength. However, these expressions are normally based on the correlation at 28 days and do not take into account the fact that the elastic modulus develops more rapidly than the compressive strength. The modulus of elasticity should therefore be expressed as a function of the maturity or the degree of hydration.

Paulini and Gratl [Paulini, 1994] have set up a model for the calculation of the development of the elastic modulus on the basis of the concrete composition. This model, however, does not take into account the effects of additives (silica fume, fly-ash, plastifying agents, etc.), and is therefore not suitable for general use. Instead, the modulus should be determined experimentally at different maturities/degrees of hydration, so that the development process can be described. This has been done by [Rostásy, 1994] and [Pedersen, 1995].

It has been found that the experimental value of the modulus of elasticity depends on the loading rate; this is due to the influence of creep (cf. Sect. 3.3.7). Separating the deformation into an elastic component and a creep component is therefore a matter of definition. It is, of course, important to use the same definition when determining the modulus of elasticity and the creep properties.

It is generally assumed that the modulus of elasticity in compression is identical to that in tension. As mentioned in Sect. 3.3.7, non-linear creep effects appear at relatively low tensile stresses, which makes it difficult to separate the initial elastic deformations and the creep deformations.

3.3.7 Creep

Creep is defined as time-dependent deformation under load. Creep is a function of

- age at loading
- duration of loading
- load type (tensile or compressive)
- temperature
- moisture content
- concrete composition (cement type, W/C ratio, aggregates).

Modelling of creep at an early age is complicated by the fact that the micro-structure is continuously changing during the hardening process.

Temperature has an important influence on creep in a hardening concrete structure. On the one hand, a higher temperature will accelerate hardening and thereby reduce creep. On the other hand, a higher temperature will increase the rate of creep. This is shown in [Emborg, 1989] and by experiments described in [Umehara, 1994]. These experiments do not take into account the change in the degree of hydration resulting from hardening in the test-period. The investigations cover both compressive and tensile creep. Furthermore, there is an increase of creep in connection with temperature changes.

The moisture content also influences creep; the higher the moisture content, the greater the creep. Furthermore, there is an increase of creep in connection with moisture transport [Neville, 1983].

Umehara's tests indicate that in addition to the temperature-dependence, there is a difference between tensile and compressive creep; other tests confirm this [Morimoto, 1994]. In contradiction to these results, Bournazel and Martineau have compared tensile creep with models for compressive creep and found a fairly good agreement [Bournazel, 1993].

For compressive stresses up to 30-75% of the strength, a linear relation between creep and stress has been observed [Neville, 1993]; see Figure 9. At an early age the linearity limit may well be lower. As the compressive stresses in a hardening structure are small relative to the compressive strength, compressive creep can be considered linear. For stresses above the linearity limit failure will occur before creep has terminated, cf. [Emborg, 1989] and Beton-Bogen [Aalborg Portland, CtO, 1979]; see Figure 10.

The tensile stresses, however, will be close to the tensile strength; this means that tensile creep will be under-estimated by a linear analysis. On the other hand, creep itself will reduce all stresses, as it is a question of an eigen-stress state, so that the practical effect is likely to be minimal.

Figure 9: Relation between creep after one minute under load to the stress/strength ratio for a five days old concrete. Based on results given by Jones and Richard (1936) cited in [Neville, 1983].

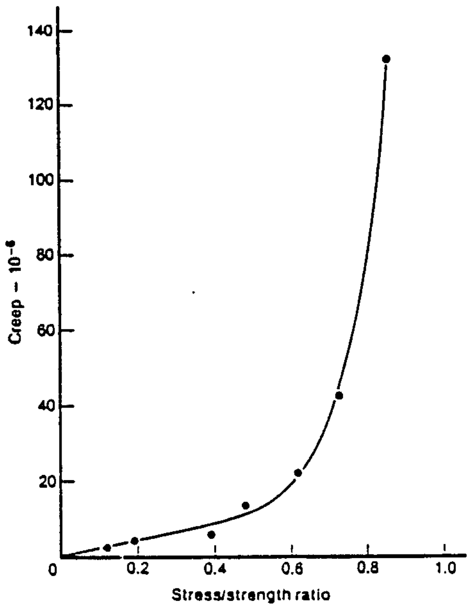
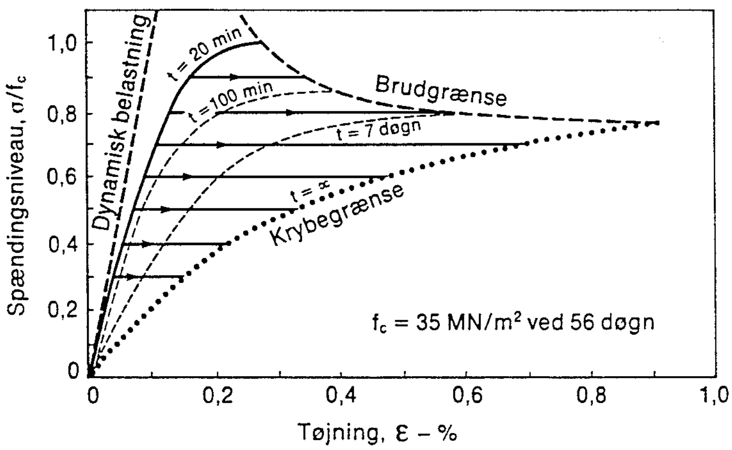


Figure 10: Isochrone relation between stress/strength ratio and strain for a 56 days old concrete (f_c = 35 Mpa). Based on results given by Rüsch (1960) cited in [Aalborg Portland, CtO, 1979].



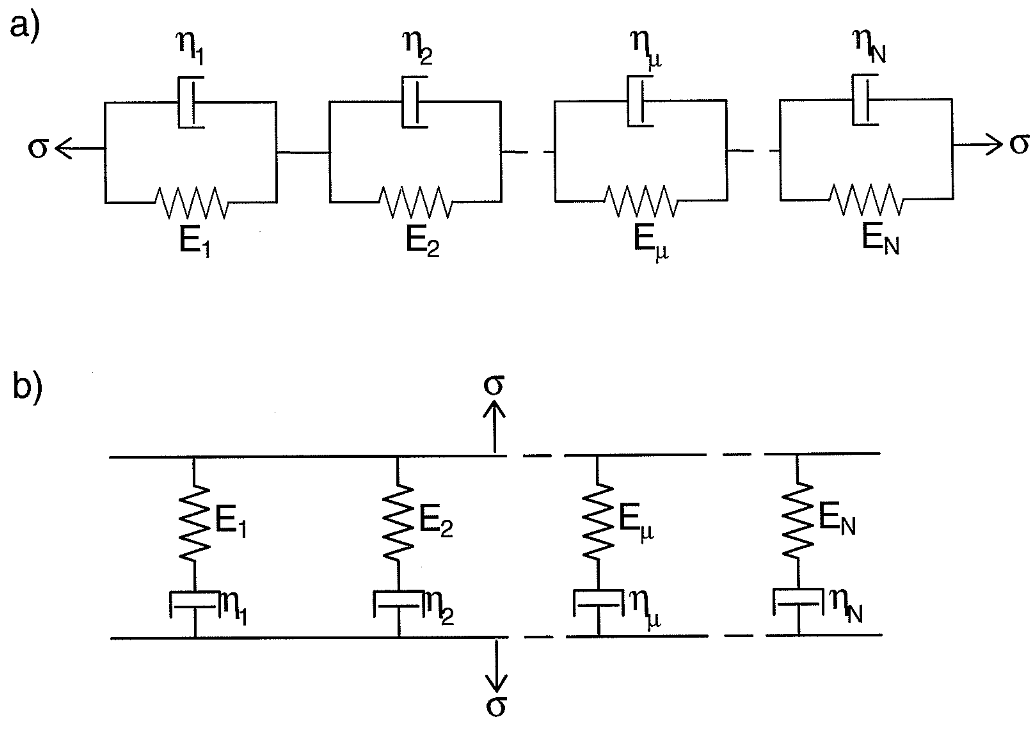
There are a number of models that describe the visco-elastic behaviour of concrete. These models can be divided into two types: Integral-type formulations and rate-type formulations.

To calculate the creep resulting from loading up to a given time by means of an integral-type model, it is necessary to know the entire loading history. As the loading in a hardening structure is continuously changing (due to temperature changes), this method is unsuitable for modelling creep in the hardening period.

Rate-type models are based on increments. This means that it is not necessary to know the entire loading history in order to determine the creep arising from loading up to a given time.

The integral-type formulation can be transformed into a rate-type formulation by expanding the creep-formation into Dirichlet series [Emborg, 1989]. This transformation results in differential equations which describe the action of the rheological models in figure 11.

Figure 11: Generalized spring and dashpot models: (a) Kelvin-Voight Chain Model and (b) Maxwell Chain Model.



Instead of using expansion of integral-type formulation, the properties of the components involved can be determined directly by the method of least squares in order to obtain the best possible agreement between the measured and the calculated deformations.

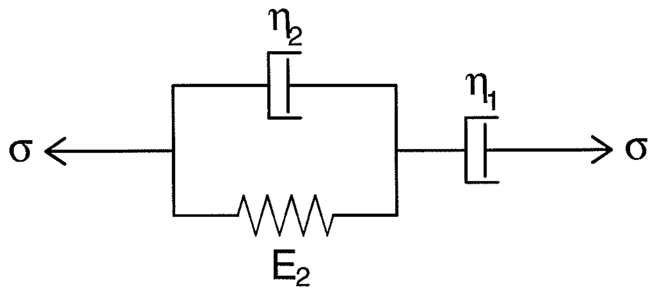
Both Kelvin-Voight and Maxwell models allow for change of properties of individual components (springs and dashpots) with degree of hydration. These developments can be described in principle by arbitrary functions.

The greater the number of components included in the model, and the greater the number of degrees of freedom the chosen functions that describe the development of properties have, the better will a given set of experimental results be accounted for. A situation could arise in which the deformation history from the test under consideration is described perfectly, but the model developed cannot be applied to another deformation history. To avoid this, the chosen developments in properties must be based on explicable phenomena. Similarly, the number of components included must be determined in relation to the amount of experimental data available.

The Maxwell model is used by [Emborg, 1989], but the number of components included is not stated. The HACON stress calculation programme computes creep deformations using a Maxwell model with 5 branches (i.e. a parallel coupling of 5 series-connections of spring and dashpot). The properties of each component are assumed to be constant.

The Kelvin-Voight model is used i.a. in the stress calculation programme CIMS-2D. The creep deformations are modelled with a dashpot in series with a parallel coupling of spring and dashpot (cf. Figure 12).

Figure 12: Creep model used in CIMS-2D.



The parallel coupling describes the reversible deformations and the dashpot describes the permanent deformations. An associated experimental programme is described in TI-B 102. Here it is also suggested that the development of the properties of each of the three components be described by an exponential function governed by two parameters.

3.3.8 Shrinkage

Shrinkage can arise as a result of loss of moisture to the surroundings. This drying shrinkage is assumed to be negligible in the hardening period if the concrete has been carefully covered. If the covering is inadequate surface cracking (crackle) may appear, but the risk of through-going cracks is unchanged because the moisture-content of the deeper of the structure is unaffected by surface drying.

Also in sealed test-specimens (i.e. no moisture transport to the surroundings), shrinkage movements can be observed. During the hydration process chemical shrinkage takes place. This leads to internal pore formation resulting in external deformations. Because of the pores the relative humidity of the binder changes leading to shrinkage in the binder resulting in external deformation. The external deformation of sealed test-specimens at constant temperature is named autogenous shrinkage.

For concretes with a low W/C ratio, typically below 0.4, the observed total autogenous shrinkage movement during the first 28 days corresponds to a temperature movement of 5-10° C according to the Danish experience.

For a given concrete composition, the development of autogenous shrinkage can be measured in the laboratory on sealed specimens. In practice it is difficult to avoid a slight rise in temperature caused by the hydration. This temperature rise results in a thermal expansion that must be compensated for (cf. Sect. 3.3.5).

According to [Tazawa, 1994] and [Bournazel, 1994], the autogenous shrinkage can be assumed to follow the degree of hydration without further temperature dependence.

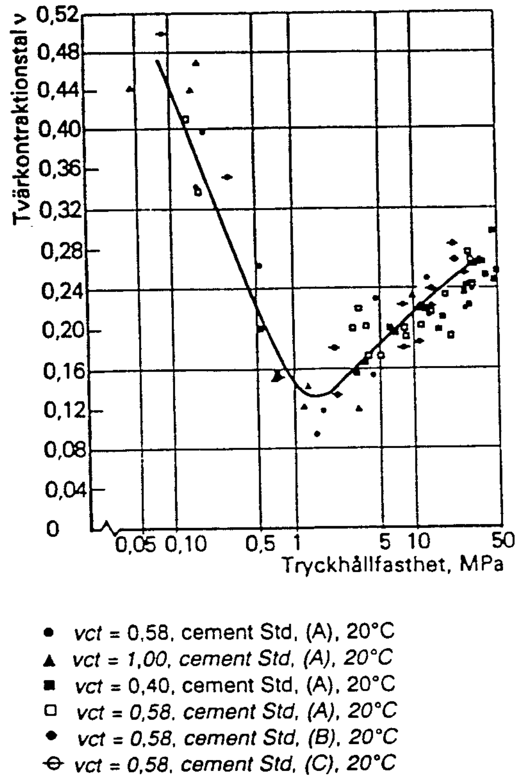
The autogenous shrinkage is caused by shrinkage of the binder. This shrinkage is restrained to some extent by the aggregate, resulting in a state of eigen-stress between binder and aggregate. When the shrinkage in the binder is high (low W/C ratio), it may result in micro-cracking around the aggregate.

3.3.9 Poisson's ratio

Immediately after casting the concrete can be compared to a liquid, which has a Poisson's ratio of 0.5. The ratio then decreases rapidly, and subsequently rises slowly to a value in the range 0.15 - 0.25 [AB svensk Byggtjänst].

Figure 13 shows the Poisson's ratio as a function of compressive strength. As the compressive strength develops during the hardening process, this gives an impression of the development of the Poisson's ratio.

Figure 13: Relation between Poisson's ratio and compressive strength. [Byfors, 1980].



A sensitivity analysis described in [Pedersen, 1995] shows, however, that whether Poisson's ratio is assumed to be constant or to vary during the hardening process makes little difference to the risk of cracking. A constant value of 0.17 is suggested.

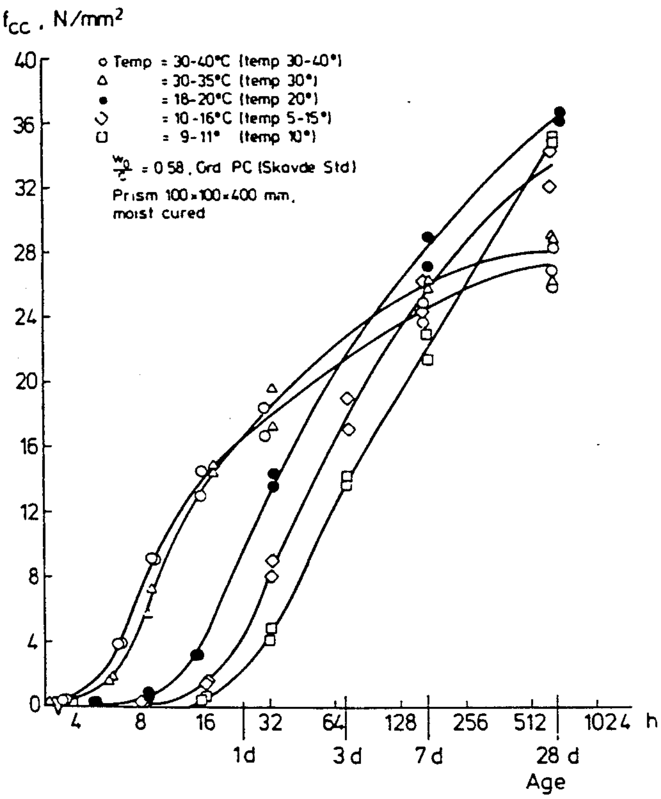
The analysis also investigates Poisson's ratio for creep, and finds that the value adopted is of little significance.

3.3.10 Tensile strength

As described in Sect. 3.2.2, the risk of cracking is defined as the tensile stress/tensile strength ratio. It is therefore necessary to know how the tensile strength develops during the hardening process. The tensile strength is frequently considered to be a function of the compressive strength, but the expressions are normally based on the correlation at 28 days and do not take the difference in the rates of development into account. The tensile strength should therefore - analogously to the modulus of elasticity - be described as a function of the maturity or the degree of hydration. Such a description implies that the strength at a given maturity or degree of hydration is independent of the previous temperature history.

However, compressive strength tests show, that this is not the case for specimens cured at high temperatures (over 30-40° C), which have lower strengths than specimens cured at 20° C, cf. Figure 14, e.g. [Byfors, 1980] and SSB-Forundersøgelser [BKF-Centralen, 1979]. It is likely that this also applies to tensile strengths.

Figure 14: Compressive strength gain at different curing temperatures [Byfors, 1980].



In a hardening structure, temperatures can rise as high as 55-60° C, but the temperature rises slowly, unlike that in specimens cured at a constant high temperature. Tests reported in [Rostásy, 1994] show that the uni-axial tensile strength determined on specimens cured with a temperature history corresponding to that of a hardening structure is close to the tensile strength of specimens cured at 20° C.

The uni-axial tensile strength is difficult to determine, as loading rate, eccentricity, rigidity of the testing machine, moisture conditions, etc. can have a considerable influence on the result [Emborg, 1994]. The split-cylinder tensile strength and the flexural tensile strength are therefore often used as an indicator of tensile strength.

The split-cylinder tensile strength and the flexural tensile strength are determined as the stress at failure on the assumption of linear elastic theory. As the concrete does not behave elastically throughout the loading period, the value for tensile strength will depend on the dimensions of the test specimen. Due to the non-linear behaviour of the concrete, redistribution of stress can occur in split-cylinder and flexural tests; this means that the split-cylinder and flexural tensile strengths will always be higher than the uni-axial tensile strength.

The relation between the uni-axial tensile strength and the split-cylinder tensile strength at an early age is unclear.

In general, caution should be exercised if the uni-axial tensile strength is estimated on the basis of other types of tests, as the correlation between e.g. the split-cylinder strength and the uni-axial strength may depend on the concrete composition. For example, the formation of micro-cracks as a result of shrinkage (cf. Sect. 3.3.8) may be significant.

4. Practical planning today

Requirements for maximum stress/strength ratios and maximum crack-widths have been specified for some larger projects. The contractor must document, by means of simulations, that the chosen method of execution can fulfil these requirements. When an appropriate method of execution has been decided upon (for the concrete composition in question), the temperature development to which the structure will be subjected can be determined. During casting, the contractor registers the measures taken during the hardening period (formwork, times at which formwork is removed, insulation etc.) Furthermore the temperature is monitored at a number of points with the aid of thermo-sensors. By this the contractor documents that the planned method of execution has been employed. The contractor's possibilities for controlling the temperature development are:

- Choice of concrete
- Casting temperature
- Casting schedule, the times at which formwork is removed
- The use of cooling pipes/heating cables
- Form materials, insulation
- Shelter from sun or wind

4.1 Temperature calculation

Temperatures are normally calculated based on the heat development, specific heat capacity, density and thermal conductivity, taking heat exchange with the surroundings into account. Temperature measurements in the field show that temperatures can be predicted with an accuracy of 2-3° C, according to SSB-Forundersøgelser [BKF-Centralen, 1979]. The heat development and density are determined by tests on the concrete in question whereas the values of heat capacity and thermal conductivity are usually based on experience.

4.2 Stress calculation

The stress calculation is based on:

- Eigen-strains from temperature changes and shrinkage
- Development of the mechanical properties of the concrete (including autogenous shrinkage)
- Statical boundary conditions (including adjacent structural components)

It is important that the development of the following properties

- Heat
- Autogenous shrinkage
- Modulus of elasticity
- Tensile strength
- Creep
- Thermal expansion coefficient

have the correct mutual relations. For a given concrete this is ensured by determining the courses of development by laboratory tests.

The testing for instance in connection with the Øresund Link and associated onshore works will be carried out at normal laboratory temperature.

It is assumed that tensile and compressive creep can be described in the same way with the aid of rheological models of the Maxwell and/or Kelvin type - without taking non-linear tensile creep into account.

With regard to the statical boundary conditions, it is often necessary to select limiting cases for the support conditions (cf. Sect. 3.2.1), between which the actual support conditions are assumed to lie. This applies to the distribution of both horizontal and vertical reactions.

The planning carried out by the contractors is often based on the tensile stress/strength ratio is determined, whereas evaluations of crack-widths are omitted. The stress/strength ratio is the relation between the maximum principal tensile stress and the tensile strength at a given point at a given time. The requirement for the risk of cracking is often specified as 60-90% of the split-cylinder tensile strength.

The stress with which the strength is compared is calculated on the basis of a number of material properties. These properties are expressed as mean values.

As these properties have a certain variation, the calculated stresses also have a certain variation. If all the properties, including the tensile strength, were uncorrelated, cracks would form in 50% of the cases if the maximum permissible stress were equal to the tensile strength i.e. 100% utilization of the tensile strength. But the properties are not uncorrelated; e.g. a high modulus of elasticity is correlated with a high tensile strength. It is difficult to determine the permissible degree of utilization of the tensile strength. An excessively high level will result in many cases of damage, while an excessively low level means stricter requirements for the measures to be taken during hardening and consequently an increased construction cost.

The requirements should be such that damage occasionally arises. Observations on the onshore works of the Øresund Link show that there is usually no damage when the calculated degree of utilization of tensile strength is below 60%, while a degree in excess of 100% of the split-cylinder tensile strength always results in damage.

5. Further investigations

It is proposed that the following fields are the most appropriate to investigate further within the framework of HETEK:

- All tests related to practical applications are performed at constant laboratory temperature, while the temperature in structures undergoes a temperature history e.g. 15-35-15 °C. It shall be clarified whether the development of the mechanical properties of concrete (incl. autogenous shrinkage) follow the maturity function (based on compressive strength).

- The criterion for crack formation is the ratio of tensile stress to tensile strength. A high degree of utilization of the tensile strength results in large creep deformations. This effect on creep should be investigated.

- It shall be experimentally verified that the distinct descriptions of the development of the concrete properties is adequate, and that these are "put together" correctly in the calculations. This can be done in a one-dimensional test frame with the possibility of applying compression and tension loads.

- It shall be experimentally verified that it is permissible to calculate a three-dimensional stress state on the basis of mechanical properties determined by one-dimensional testing. This can be done in a "full-scale" test involving two adjacent concrete segments cast at different times with well-known statistical boundary conditions. Such a test will also determine whether the connection between the segments has been modelled correctly.

- The distribution of the reaction from the sub-base should be clarified as it has significant influence on the risk of cracking.

- The permissible tensile strength should be determined on the basis of observations on cracks in existing structures.

The above-mentioned topics shall be clarified as far as can be justified by the practical application of stress calculations for early-age concrete. This means that an isolated phenomenon shall not be investigated in detail if it can be shown that it is only of marginal significance for the problem as a whole.

6. Literature

Aalborg Portland, CtO: "Beton-bogen". 1. udgave, 1979.

AB Svensk Byggtjänst och Cementa AB: "Betonghandbok - Material", 1994.

BKF-Centralen: "SSB-Forundersøgelse, bind 2: Forsøg". Juni 1979.

Bournazel, J.P. and Moranville-Regourd, M.: "Thermal Effects, Cracking and Damage in Young Massive Concrete". RILEM Symposium, 1994, pp 329-336.

Bournazel, J.P. and Martineau, J.P.: "A Laboratory Test to Analyse Creep Under Tension of Young Concrete, Proceedings of the Fifth International RILEM Symposium on Creep and Shrinkage of Concrete, Barcelona, Spain, Editors: Bazant and Carol, 1993.

Breugel, K. van.: "Artificial Cooling of Hardening Concrete". Delft University of Technology, Research Report No. 5-80-9, Delft, 1980.

Breugel, K. van.: "General Report, Session B: Prediction of Temperature development", General Reports of the International Symposium Thermal Cracking in Concrete at Early Ages, Munich, 1994.

Brown, T.D. and Javaid, M.Y.: "The thermal conductivity of concrete". Matéraux & Construction, Vol. 3, No 18, 1970, pp 411-416.

Byfors: "Plain concrete at early ages". Swedish Cement and concrete Institute, Fo 3:80, Stockholm, 1980.

Dahlblom, O. and Ottosen, N.S.: Smeared crack analysis using generalized fictitious crack model, Journal of Engineering Mechanics, Vol. 116, No. 1, pp. 55-76, 1990.

DIN 1045

Emborg, M.: "Thermal Stresses in Concrete Structures at Early Ages". Luleå University of Technology, 1989.

Emborg, M.: "Development of Mechanical Behaviour at Early Ages including Mathematical Models". Draft No. 4 of RILEM State-of-The-Art Report, Chapter 6, 1994.

Freiesleben, Hansen, P.: "Hærdeteknologi-2, Dekrementmetoden". Aalborg Portland og BKF-centralen, 1978.

Grodtkjær, Erik and Bergholdt, Mogens: "FIESTA. Generel elementmetode program. Brugermanual, Revision G". Dansk Konstruktions Analyse A/S, Februar 1993.

Huckfeldt, J.; Duddeck, H.; Ahrens, H.: "Numerical simulation of Crack Avoiding Measures". RILEM Symposium, 1994, pp 255-264.

Hundt, J. and Wagner, A.: "Einfluβ des Feuchtigkeitgehalts und des Reife grads aud die Wärmeleitfähigkeit von Beton". Deutscher Ausschuβ für Stahlbeton, heft 297, 1978.

Kishi, T. and Maekawa, K.: "Thermal and Mechanical Modelling of Young Concrete based on Hydration Process of Multi-component Cement Minerals". RILEM Symposium 1994, pp 11-18.

Koenders and van Breugel: "Numerical and Experimental Adiabatic Hydration Curve Determination", RILEM Symposium, 1994, pp 3-10.

LaPlante, P. and Boulay, C.: "Evolution du coefficient de dilatation thermique du beton en fonction de sa maturité aux tout premiers âges". Materials and Structures, 27, 1994, pp 596-605.

Maréchal, J.C.: "Determination simultanée de la diffusivité et de la conductivité thermique du beton pendant son hydration". Contribution to the Task Group W 40 of the Conseil International du Batiment (CIB), 1972.

Mishima, T.; Umehara, H.; Yamada, M.; Nakamura, M.: "Thermal Stress in Full Size RC Box Culvert". RILEM Symposium, 1994, pp 171-178.

Morimoto, H. and Koyanagi, W.: "Estimation of Stress Relaxation in Concrete at Early Ages", RILEM Symposium, 1994, pp. 95-102.

Neville, A.M.; Dilger. W.H.; Brooks, J.J.: "Creep of plain and structural concrete". Construction Press, London and New York, 1983.

Nolting, E.: "Zur Frage der Entwicklung lastunabhängiger Verformungen und Wärmedehnzahlen junger Betone". Dissertation, Inst. Baustoffkunde und Materialprüfung, Technische Univ., Hannover, Heft 56, 1989, pp 206.

Nordström, Erik: "HACON-S, Version 2.0, Users Manual", Vattenfall Utveckling AB, report UC1995:Ö2, 1995.

Paulini, P. and Gratl, N.: "Stiffness Formation of Early Age Concrete". RILEM Symposium 1994, pp 63-70.

Pedersen, E.S.:"Prediction of Temperature and Stress Development in Concrete Structures". RILEM Symposium 1994, pp 297-304.

Pedersen, E.S; Olesen, S.Ø.; Lykke, S.: "Curing Technology on the Øresund Link. Basis for Stress Calculations". Nordic Symposium on Modern Design of Concrete Structures, Aalborg University, Denmark, May 3-5, 1995.

Roelfstra, P.E.; Salet, T.A.M; Kuiks, J.E.: "Defining and Application of Stress-analysis-based Temperature Difference Limits to Prevent Early-Age Cracking in Concrete Structures". RILEM Symposium 1994, pp 273-280.

Rostasy, F.S.: "Session D: Determination and Modelling of Mechanical Properties". RILEM State of the Art Reports (Drafts), 1994.

Sassedateljew, J.B. and Mischin, G.W.: "Einfluβ der Strukturbildung im Beton auf die Wärmeleitung". Baustoffindustrie, 1970, pp 116-119.

Svenska Betonföreningen. "Sprickor i betonkonstruktioner - särskilt temperatursprickor." Betongrapport nr. 3, 1994.

Tanabe, T.: "Compensation plane and compensation line method". Nagoya University of Technology, International report, 1992, pp 8.

Tazawa, E.; Matsuoka, Y.; Miyazawa, S.; Okamoto, S.: "Effect of Autogenous Shrinkage on Self Stress in Hardening Concrete". RILEM Symposium, 1994, pp 221-228.

Thelandersson, S.: "On the multiaxial behaviour of concrete exposed to high temperature". Journal of Nucl. Eng Des., Vol 75, No 2, 1983, pp 271-282.

Thelandersson, S.: "Modelling of combined thermal and mechanical action in concrete". J Eng Mech Div (ASCE), Vol 113, No 6, June 1987, pp 893-906.

TI-B 102: Strains from creep and early-age shrinkage, 1994.

Umehara, H; Uehara, T.; Iisaka, T.; Sugiyama, A.: "Effect of Creep in Concrete at Early Ages on Thermal Stress". RILEM Symposium, 1994, pp 79-86.

Wang, Ch. and Dilger, W.H.: "Prediction of Temperature Distribution in Hardening Concrete", RILEM Symposium 1994, pp 21-28.