HETEK

Control of Early Age Cracking in Concrete Guidelines

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Abstract

This guideline is a part of the Danish Road Directorate's research programme called High Performance Concrete - The Contractor's Technology (abbreviated to HETEK). For durability reasons reinforced concrete 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. Cracks can form 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 relative to the tensile strength there is a high risk of crack formation.

This guideline points out how concrete structures can be designed and builded without harmful formation of cracks during the hardening process.

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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, 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 contractors'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 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 guidelines deal 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 in the concrete. Contrary to ordinary stress calculations the mechanical properties (including autogenous shrinkage) change during the hardening process. If a stress analysis shows that stresses are high compared to the tensile strength there is a high risk of crack formation.

The purpose of these guidelines is to contribute to the design and execution of structures with satisfactory durability, functioning and appearance, i.e. where crack formation is limited or if necessary avoided.

The guidelines assist

- the owner in specifying relevant requirements in preparation of specifications (Chapter 7)
- the designer in assessing the risk of crack formation during the hardening process in the design phase (Chapter 3)
- the contractor in controlling crack formation by means of planning and choice of construction method (Chapter 5) and documentation of the hardening process (Chapter 6)

Efforts in these fields should result in structures being executed with a satisfactory, but not unnecessarily high safety against harmful crack formation. This should result in low overall costs for construction, maintenance and repair.

Chapters 1 and 2 describe the hardening principles while Chapters 4 deals with material properties.

Each chapter constitutes a whole, and therefore repetitions may occur.

The project was carried out by a consortium consisting of:

The Danish Concrete Institute represented by:

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

and

The Danish Technological Institute, represented by the Concrete Centre

and

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

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

1.0 Introduction

In order to obtain satisfactory

- durability
- serviceability (e.g. tigthness)
- appearance

efforts are made to limit formation of cracks in concrete structures. Cracks can 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 differences in thermal strains in the structure.

Traditionally thermal strains have been limited by requiring the contractor to plan the concrete so that large temperature differences between different parts of the structure do not arise during the hardening process.

The requirements comprise:

- a maximum temperature difference between the surface and inner zone of a structure
- a maximum difference between the average temperatures of adjacent structural components.

The requirements seem simple to use, but in practice there is an uncertainty in determining which parts of a structural component are to be included when the average temperature is determined.

Requirements for maximum temperature differences do not consider the interaction with other structural components (including the sub-base) nor are the rates of change of the mechanical material properties of the concrete in question included.

Consequently a general temperature requirement is not always appropriate. For some structures the temperature requirement will involve unnecessary measures, and for others the risk of generation of cracks will be too high. In order to evaluate the risk of crack formation in a structure realistically it will be necessary to carry out a stress analysis. A stress analysis shows whether the stress at any time will exceed the tensile strength of the concrete, thereby generating formation of cracks.

Apart from thermal strains, the shrinkage of the concrete contributes to the risk of crack formation. Shrinkage can be:

- drying shrinkage caused by exchange of moisture with the environment
- autogenous shrinkage with no exchange of moisture with the environment.

The drying shrinkage can be deferred by the curing described in the guidelines on curing [HETEK-Curing, 1997], while the autogenous shrinkage can be controlled only through the choice of concrete mix.

When the drying shrinkage is postponed by means of treatment the risk of crack formation in connection with the hardening process is not affected. Therefore the present guidelines only deal with control of crack formation caused by thermal strains and autogenous shrinkage.

The guidelines describe normal practice. The background of the directions given in the guidelines is based on the know-how and experience gathered by institutes, consultants and contractors. A part of the backgrond is gathered during the work on the HETEK project on “Control of early age cracking in concrete”. This part is summarized in “HETEK - Control of early age cracking in concrete - Main report” [Spange & Pedersen, 1997].

2.0 Background

Due to strains from temperature and shrinkage a 3-dimensional stress-state is generated in hardening concrete structures.

The stress-state is traditionally considered as arising from two sources:

- stress due to different displacements between adjacent structural components including the sub-base
- stress due to internal strain-profiles.

In practice both types of stress are represented in the same structure, where they give rise to risk of crack formation at different periods. The risk of surface cracks is highest when the temperature is a maximum, while the risk of cracks passing through the structure is highest after the cooling period. These stress-states are described in the following two Sections.

2.1 Stress due to different displacements between adjacent structures - through-going cracks

Stress arising from different displacements between adjacent structures is shown in Figure 2.1d. A concrete strut is cast between two adjacent hardened structures. At first the concrete is a liquid, but hardening takes place and the strut becomes stiff (Figure 2.1c).

At the same time heat is generated and this results in a temperature rise (Figure 2.1 b). Due to the rise in temperature the concrete strut expands longitudinally, but the expansion is restrained by the adjacent structures and compressive stresses are generated (Figure 2.1d) in the strut.

When the temperature begins to fall the strut will contract, and because the stiffness is much greater during the cooling period than during the heating period (Figure 2.1 c), tensile stresses will be generated in the strut after the hardening process.

The stress level depends on:

- the temperature change
- the changes in concrete properties (including shrinkage and creep)
- the stiffness ratio between the strut and adjacent structural components

If the tensile stress exceeds the tensile strength, which is also a function of the degree of hydration, cracks will be formed.

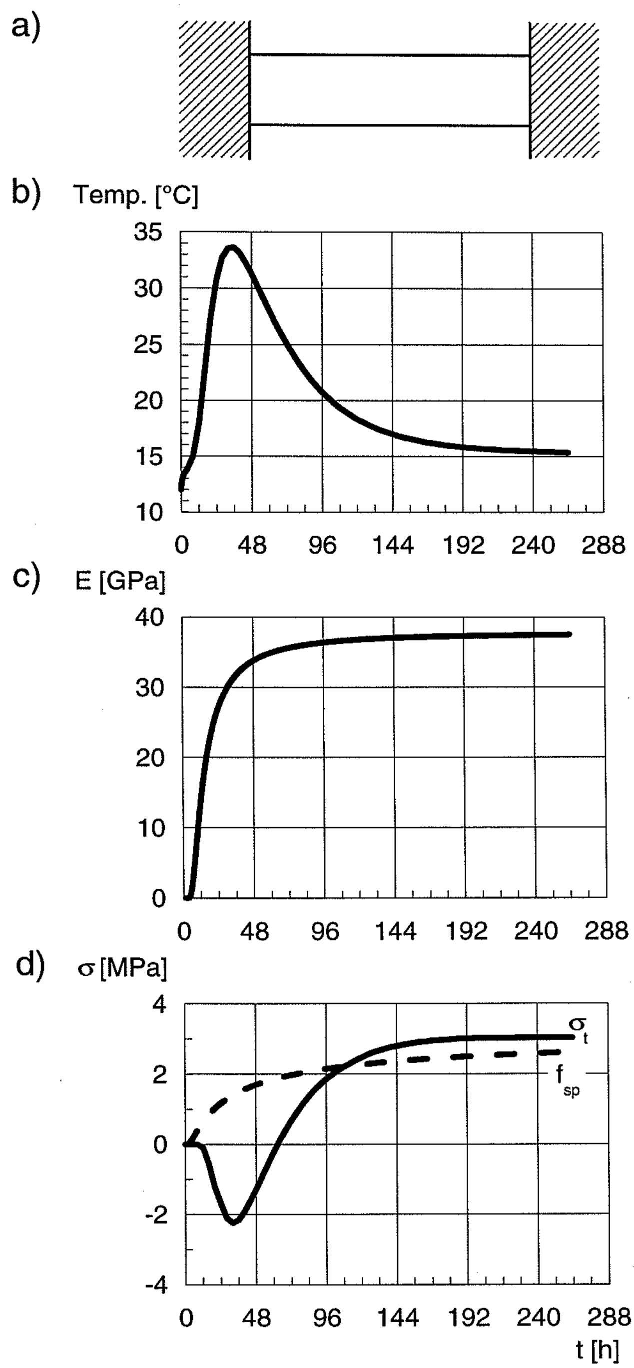

Figure 2.1: Development of stresses in concrete cast between two rigid structures.

Cracks generated during the cooling period will often pass through the structure and remain open. This type of crack is observed when a wall is cast on an existing sub-base as shown in Figure 2.2.

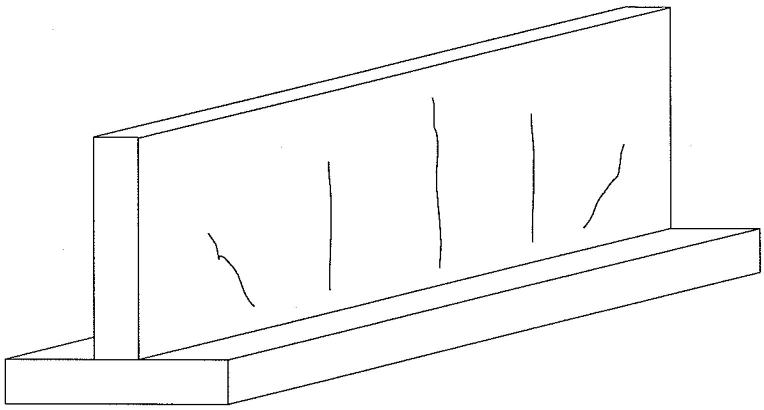

Figure 2.2: Wall casted on hardened foundation.

Traditionally it has been tried to avoid or at least to limit the occurrence of this type of crack by reducing the difference in average temperature between adjacent structures, hereafter referred to as D_{ext}.

2.2 Stress due to internal strain-profiles - Surface cracks

Due to internal strain-profiles stresses are also generated within the cross-section. Figure 2.3 shows the temperature distribution in the cross-section of the strut (Figure 2.1) at the moment when the maximum temperature is attained. The rise in temperature in the core of the cross-section is greater than at the surface and the thermal expansion is consequently greater. This creates an eigenstress-state where the core of the cross-section is in a state of compression while the surfaces are under tension. Figure 2.4 shows the distribution of the vertical stress component when the temperature is a maximum.

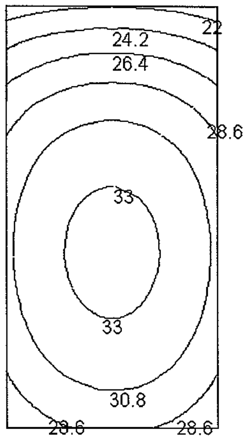

Figure 2.3: Temperature distribution when temperature is a maximum [°C]

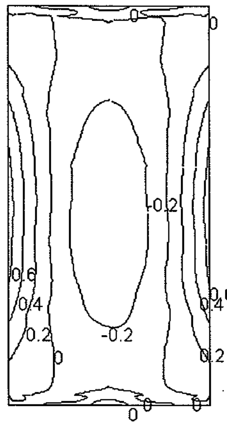

Figure 2.4: Distribution of vertical stress component when temperature is a maximum [MPa]

If the temperature difference between the core of the cross- section and the surface is too high, cracks will form at the surface. These cracks will close again when the temperature profile has become level during the cooling period. At this time the surfaces are subjected to compression because the highest thermal contraction takes place at the core of the cross-section, which is subjected to tension (Figure 2.5).

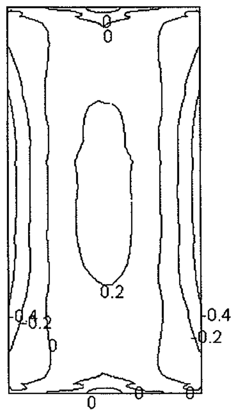

Figure 2.5: Distribution of vertical stress component after hardening period [MPa].

The stress level depends on:

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

Normally surface cracks tend to close again and become difficult to detect by inspection of the concrete surface, but they will provide paths for penetration of water and chlorides through the surface layer. The moisture and temperature changes during the subsequent service life of the structure will enhance the risk of penetration through surface cracks.

Traditionally it has been tried to avoid or to limit this type of crack by reducing the difference between the average temperature of the cross-section and the surface temperature, in the following referred to as D_{int}.

2.3 Calculation models for assessment of risk of crack formation

Practically the risk of crack formation is assessed as the ratio between maximum principal tensile stress and an indicator of the tensile strength of the concrete. The calculation of the principal tensile stress requires computers. An evaluation of the risk of cracking involves:

- an analysis in which the temperature changes during the hardening period are determined for each location in the structure
- an analysis of the stress development in each location as a function of the thermal strains, mechanical properties (including autogenous shrinkage), support conditions etc.
- a model for change of the mechanical properties of the concrete (stiffness, strength, etc.) during the hardening process.

The temperature variations can to a certain extent be controlled by the contractor through his choice of execution method. These temperature variations determine partly the thermal strains and partly the "maturity" of the mechanical properties of the concrete.

The risk of crack formation is assessed by comparing the calculated tensile stresses to the tensile strength of the concrete. In practice the splitting tensile strength is used, because it is easily determined by testing. Observations of a number of structures have shown that there is a risk of through-going cracks, cf. Section 2.1, when the calculated tensile stresses exceed approximately 80% of the splitting tensile strength (according to [DS423.34]), while cracks are rarely observed below this level. [Pedersen, E.J., 1977].

2.4 Thermal analysis

The temperature distribution and its variation with time determine

- the thermal strains
- the rates of change of mechanical properties (incl. autogenous shrinkage)

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:

- casting temperature
- heat of hydration
- heat capacity, conductivity, density
- thermal boundary conditions in the form of convection, radiation, cooling pipes, heating cables etc.
- geometry of structure
- geometry and temperature of adjacent structures,if any.

The temperature distribution of hardening concrete is 1-, 2- or 3-dimensional depending on geometry and boundary conditions. The structure in Figure 2.2 presents a 3-dimensional temperature distribution in the end zones because much energy is dissipated at the end surfaces. Between the end zones the temperature distribution is 2-dimensional as shown in Figure 2.6. The more the end zones are insulated the shorter are the end zones with a 3-dimensional temperature distribution. In practice, 2-dimensional temperature distributions are sufficient.

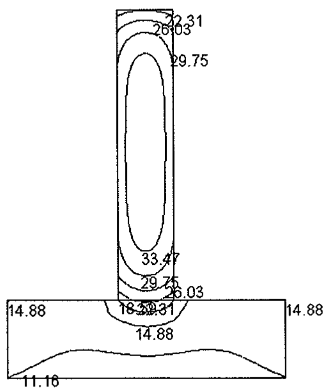

Figure 2.6: Wall cast on existing sub-base. 2-dimensional temperature distribution [°C].

Normally computer programmes are 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-interval. However, calculation of a temperature field is much less resource-demanding than stress calculations.

In temperature calculations it must be assumed that transport of moisture to the surroundings is prevented as moisture profiles are not taken into account.

2.5 Stress analysis

The stress analysis must be incremental as the stress is generated by a non-stationary (transient) temperature strain (Figure 2.1d) and material properties that depend on the temperature- and stress-history. 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.

During the hardening process strains arise from

- drying shrinkage
- autogenous shrinkage
- thermal expansion/contraction
- creep
- crack formation

A short-term result of drying shrinkage may be shrinkage strain at the surface. During the heating period, when the risk of formation of surface cracks is greatest, curing is normally applied which postpones the drying process. The risk of formation of through-going cracks later on during the hardening process is considered to be independent of a slow drying of the surfaces. Therefore drying shrinkage is normally not included in practical calculations.

The material properties that determine the remaining strain contributions depend on the maturity of the concrete. And there may also be a certain dependence on the absolute temperature and stress level as described in the State-of-The-Art report [Pedersen et al, 1996]. For practical calculations the material models described in Chapter 4 can be used. In these models all properties are constant or functions of the maturity.

In practice no stress calculations are carried out where formation of cracks is considered. A calculation of crack width and distance between cracks requires a 3-dimensional analysis of crack mechanism that is too time-consuming in practice. Furthermore, the correlation between crack width and lack of durability has not been clearly demonstrated.

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-histories into account.

Depending on the programme a 1-, 2- or 3-dimensional analysis can be carried out. 3-dimensional analyses can of course describe any stress-field, but require long computation times. 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 2.7. Cracks extending through the wall can be observed; these are due to differences in temperature movements and autogenous shrinkage between wall and foundation, cf. Section 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 considered. As the normal stress (σ_z) at the ends is zero, the normal stresses in the intermediate zone are converted to shear stresses in the end-zones (Figure 2.7).

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 in several commercial computer programmes.

In the sections under consideration there are also stresses in the plane of the cross-section arising from temperature differences between the core and the surface (cf. section 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.

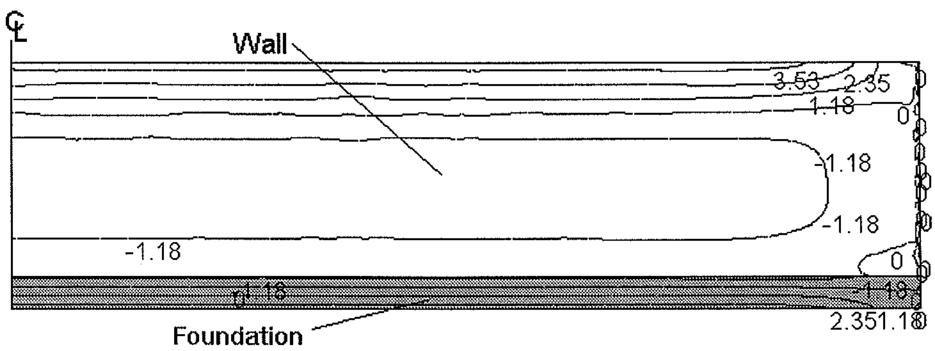

Figure 2.7: Isoquants for the longitudinal stress component σ_z [MPa].

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 level. 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.

3.0 Design of structure

As early as during the design phase a number of choices have to be made that concern the execution of the structure in order to avoid formation of cracks caused by the hardening process. A number of questions must be answered as for instance:

- expansion joints ?
- location of construction joints ?
- amount of reinforcement with a view to crack distribution ?

An inappropriate design may mean that the structure cannot - or can only with difficulty - be built without generation of hardening cracks. Therefore efforts should be made so that the structure can be built in such a way that temperature- and shrinkage strains can take place without the generation of harmful stresses.

3.1 Expansion joints

A structure that is designed in such a way that temperature and shrinkage strains are prevented is very difficult to carry out without generating hardening cracks.

Example 3.1

An example is a long tunnel typically divided into a number of sections. The insertion of expansion joints between sections - as opposed to direct casting together - will make it possible for temperature strains to take place. This reduces the risk of formation of cracks passing through the wall during the cooling period. If joints are placed in wall and deck, but not in the foundation, the risk of crack formation in the wall is increased as compared to the situation where joints are placed in the foundation too. This is because the long foundation will hinder temperature and shrinkage strains in the wall and deck to a greater extent.

Experience shows that it is difficult to carry out expansion joints correctly, which results in damage occurring more often close to expansion joints than at construction joints or as hardening/shrinkage-cracks. The number of expansion joints should thus be as few as possible. The ends of reinforcement bars at expansion joints should be given the normal concrete cover.

3.2 Construction joints

As regards durability, construction joints are considered to be a weak location in the structure, and therefore construction joints are avoided at certain places in the structure. For instance in the splash zone (waterline) in structures in a marine environment. Also for aesthetic reasons constructions joints are avoided at certain places.

Normally the contractor plans the casting. One parameter here is the casting rate, which influences the location of construction joints. If there are areas in the structure where construction joints must not be placed for reasons of durability or aesthetics, such areas may be specified in the tender material. In this case the contractor decides on the optimum location of construction joints and at the same time takes durability and aesthetics into account.

3.3 Crack width / minimum reinforcement

Reinforcement does not prevent crack formation, but distributes the cracks so that the result is more but smaller cracks.

Crack width and distance between cracks depend on the amount of reinforcement and its dimension as well as influences from:

- strains from temperature and shrinkage during the hardening process
- strains from temperature and shrinkage during service
- dead load, live load, etc.
- differential settlement of foundations.

[BS 8007], [DIN 1045] and [Beton-Teknik, 1981] give calculation methods in which the above influences are included. It should however be stressed that the stipulation of crack width by means of calculations is connected with great uncertainty.

4.0 Concrete properties

The behaviour of concrete during the hardening process can be described by considering a number of distinct properties determined independently by experience, calculation or testing. When these properties are put together in a calculation of a temperature and stress process the behaviour of any concrete structure can be described in principle. The 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 state to a fully hardened material, the thermal and mechanical properties change. In order to be able to plan the hardening process by calculations of the temperature and stress changes, the relevant properties must be described from the mixing time and throughout the whole hardening process, cf. Section 2.3.

The following concrete properties should be used as data when concrete works are planned:

Thermal properties
- Adiabatic heat generation
- Specific heat
- Density
- Thermal conductivity

Mechanical properties
- Coefficient of thermal expansion
- E-modulus
- Tensile strength (for instance splitting tensile strength)
- Poisson's ratio
- Autogenous shrinkage
- Creep

A calculation of the temperature only requires that the thermal properties of the concrete are known, but a calculation of stresses requires both the thermal and the mechanical properties.

It is important that all relevant properties be determined on the concrete that is used so that the same reaction sequences form the basis of all properties. In [Spange and Pedersen, 1996] and in [Riis et al, 1997] two complete sets of material properties for concrete mixes are given.

In connection with calculations of hardening it is not possible (as in the case of calculations of load capacity) to determine whether a given property should be given a high or a low characteristic value in order to be on the safe side.

Therefore the properties are described only by average values so that the calculated process as far as possible corresponds to the real properties.

As the properties tend to vary there will also be a certain variation of the calculated stresses. If all properties, including the tensile strength, were not correlated cracks would be generated in 50% of all cases if the tensile strength were utilized 100%. But the properties are correlated; for example, if the E-modulus is high the strength is also high. The safety factor against crack formation is reflected in the permissible coefficient of utilization of the tensile strength. A high coefficient of utilization will result in a certain frequency of damage, while a low coefficient of utilization will require unnecessary hardening measures.

Apart from testing in relation to control of crack formation it must be shown that the chosen concrete mix satisfies the specifications stipulated for workability, casting properties, stability in the fresh state and also the requirement for properties found in the finished structure.

4.1 Maturity

The rate of the chemical reaction between water and cement depends on the temperature. The higher the temperature, the higher is the rate of reaction. In a hardening concrete structure each location has its own temperature history (cf. Figure 2.3) and thus its own development. It is therefore necessary to define a parameter that indicates how far the chemical reaction has proceeded.

Example 4.1

A measure for how far the chemical reaction has proceeded can be expressed by "maturity". Maturity corresponds to the level in the reaction where the process would be if the reaction had taken place at 20 °C.

In practice the maturity at a point is found by dividing the temperature process at the point into time-intervals. The average temperature in a time-interval, Δt, determines the rate factor, H, of the time-interval and the contribution of the time-interval to the total maturity is the product of the length of the time-interval and the rate factor. The total maturity M is found as the sum of the maturity contributions of all intervals:

$$M = \Sigma \Delta t \cdot H \quad \text{[hours at 20 °C]} \qquad (4.1)$$

where Δt *is the time-interval considered*
H *is the rate factor in the time-interval*

The rate factor for the calculation of maturity is given by the following formula::

$$H = \exp E/R\ [\ 1/293 - 1/(273 - \theta)] \qquad (4.2)$$

where $E = 33500 \text{ for } \theta \geq 20\,°C$ *[J/mol]*
$E = 33500 + 1470(20 - \theta) \text{ for } \theta < 20\,°C$ *[J/mol]*
$R = 8.314$ *[J/mol °K]*
$\theta =$ *is the average temperature in the considered time-interval [°C]*

The rate factor is tabulated in table 4.1

The dependence of the rate factor on the temperature is determined on the basis of changes in the compressive strength determined at different temperatures.

In practical calculation it is assumed that the correlation between the state of the reaction and the magnitude of a certain parameter is unambiguous.

The rate of development of the different properties is determined by testing at laboratory temperature and then converting this rate to a function of the reaction process. Now the properties can be utilized at any temperatures observed in real structures.

°C	+0	+1	+2	+3	+4	+5	+6	+7	+8	+9
-10	0.03	0.03	0.04	0.05	0.06	0.07	0.08	0.10	0.11	0.13
0	0.15	0.17	0.20	0.23	0.26	0.29	0.33	0.37	0.41	0.45
10	0.50	0.54	0.59	0.64	0.70	0.75	0.80	0.85	0.90	0.95
20	1.0	1.0	1.1	1.1	1.2	1.3	1.3	1.4	1.4	1.5
30	1.6	1.6	1.7	1.8	1.9	2.0	2.0	2.1	2.2	2.3
40	2.4	2.5	2.6	2.7	2.8	2.9	3.1	3.2	3.3	3.5
50	3.6	3.7	3.9	4.0	4.2	4.3	4.5	4.7	4.8	5.0
60	5.2	5.4	5.6	5.8	6.0	6.2	6.5	6.7	6.9	7.2

Table 4.1: The rate factor at different temperatures. Example: The rate factor at 15 °C can be read to 0.75.

4.2 Adiabatic heat of hydration

The heat of hydration is the result of the hydration process and is determined as adiabatic heat generation, i.e. the generated heat results in a temperature rise in the concrete. As the heat development in concrete reflects the reaction between cement and water the process will depend both on the chosen type of cement and on the quantities of cement and water. The heat of hydration is proportional to the total content of powder (cement + fly ash + silica fume) and also on the amount of water. If all the water is used the reaction and consequently the heat development stop.

The cement type determines the rate of heat development and the total amount of heat that is generated. Normally these properties are controlled by the cement fineness and the cement composition, and it will therefore be possible to choose between rapid hardening cements with a high heat of hydration and ordinary or slow hardening cements with a moderate to low heat of hydration.

Another way to control the heat development than by choice of cement type is to influence the process by retarding or accelerating additives. The use of plasticizing additives often has a retarding side effect.

The heat of hydration can be determined by testing according to [NT BUILD 388 "Heat development"]. The generated heat is proportional to the cement content or the total content of powder if silica fume and fly ash have been added, which will appear from the concrete mix. It has to be checked that the calculation of the temperature development is based on the content of powder that is assumed in the adiabatic heat development used.

Example 4.2

Figure 4.1 shows a measured heat development, Q [kJ/kg cement or powder] of an actual concrete. A regression curve is marked on the figure:

$$Q = Q_\infty \exp[-(\tau_e / M)^\alpha] \tag{4.3}$$

Q_∞	=	*is the total heat development*
τ_e	=	*is a gradient that determines the curve in time*
α	=	*is a curvature parameter that describes how sharply the property curve bends in the S-shape*
M	=	*is the maturity*

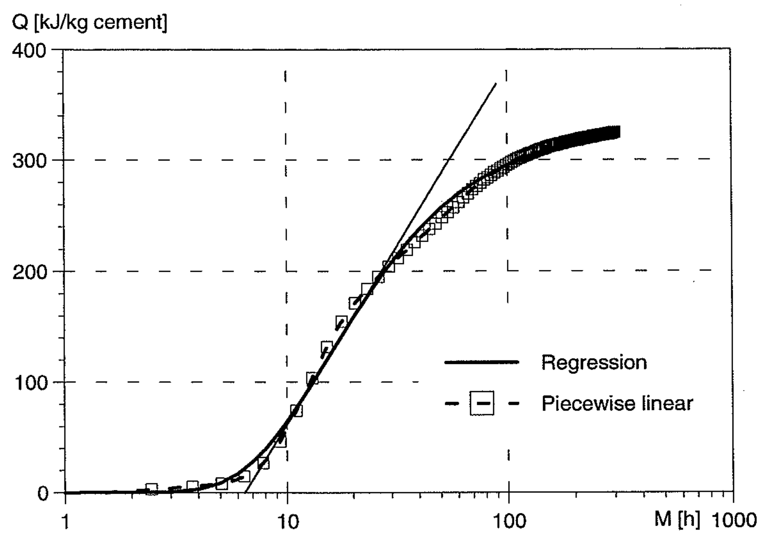

Figure 4.1: Example of heat development.
Q_∞ = 332 kJ/kg cement, τ_e = 14.43 h, α = 1.15, τ_0 = 6.46

The three parameters can be determined by regression. The figure shows the tangent at the point with no curvature of the curve. The tangent intersects the x-axis at the time τ_0 The time corresponds approximately to the setting time which is determined according to [DS 423.17]. In the case of concrete that contains both silica fume and fly ash it might be difficult to get good correlation between the measurement and this regression curve. If it is impossible to describe the development satisfactorily by means of the regression curve a curve of linear segments may be used - this is made by connecting the individual observations, cf. Figure 4.1.

4.3 Specific heat and density

The density of the concrete and its specific heat determine how much energy is needed to raise the temperature of the concrete.

The density very much depends on the density of the aggregate materials and on the content of air in the fresh concrete. Typical values of the density are in the range 2250 - 2400 kg/m^3.

The specific heat is influenced by the content of water and will decline during the hardening process. In practice the specific heat is regarded as constant and is calculated on the basis of the concrete composition for instance in connection with the adiabatic heat development. Typical values of the specific heat are in the range 1.0 - 1.1 kJ/kg°C.

The specific heat is calculated from the expression:

$$c_{concrete} = \frac{\sum_{i=1}^{n} m_i \cdot c_i}{\sum_{i=1}^{n} m_i} \tag{4.4}$$

where

$c_{concrete}$	= is the specific heat of concrete [kJ/kg °C]
m_i	= is the mass of a constituent material [kg]
c_i	= is the specific heat of a constituent material [kJ/kg °C]
n	= is the number of constituent materials

4.4 Thermal conductivity

In concrete with normal aggregate materials the thermal conductivity will vary from approximately 8 kJ/m h°C in the fresh state to approximately 6 kJ/m h°C in the hardened state. If lightweight aggregates are used the value declines to approximately 2.5 kJ/m h°C.

When calculating the hardening process, experience shows that a reasonable accuracy is obtained if a constant thermal conductivity of 8 kJ/m h°C is assumed, corresponding to the value for fresh concrete.

4.5 Coefficient of thermal expansion, E-modulus and tensile strength

The coefficient of thermal expansion, the E-modulus and the tensile strength are determined by testing at specific maturities. As stress calculations require that these properties be known at any age, a regression curve is established that is based on the test results. In order to be able to describe changes in development with reasonable accuracy, tests should be made at 0.5, 1, 2, 3, 7, 14 and 28 maturity days. For instance the E-modulus can be determined according to TI-B 102, splitting tensile strength according to DS 423.34 and uniaxial tensile strength according to DS 423.24.

Example 4.3

Figure 4.2 shows the development of coefficient of thermal expansion as a function of maturity.

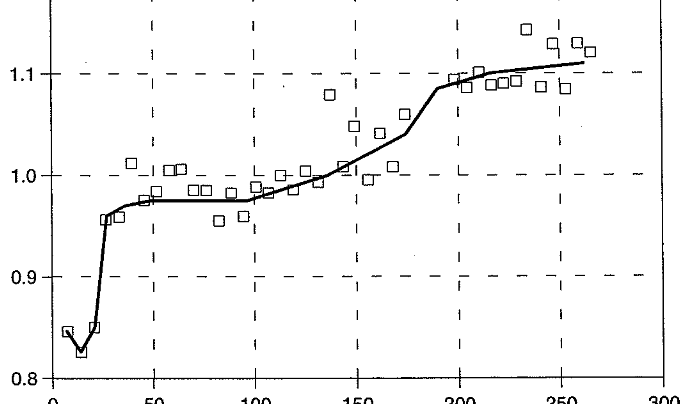

Figure 4.2: Example of development of coefficient of thermal expansion.

Example 4.4

Figures 4.3 and 4.4 show E-modulus and splitting tensile strength, found by testing at 7 different maturities. At each maturity a 3-fold determination was made and the fact that the E-modulus and the tensile strength are zero at the setting time (cf. example 4.2) was taken into account. The figures show the following function:

$$v = v_\infty \exp[-(\tau_e / M)^\alpha] \qquad (4.5)$$

where the values:

v_∞	=	*property for time approaching infinity*
τ_e	=	*gradient parameter that determines the curve in time*
α	=	*curvature parameter that describes how sharply the curve bends in the S-shape*

are determined by regression. M is the maturity.

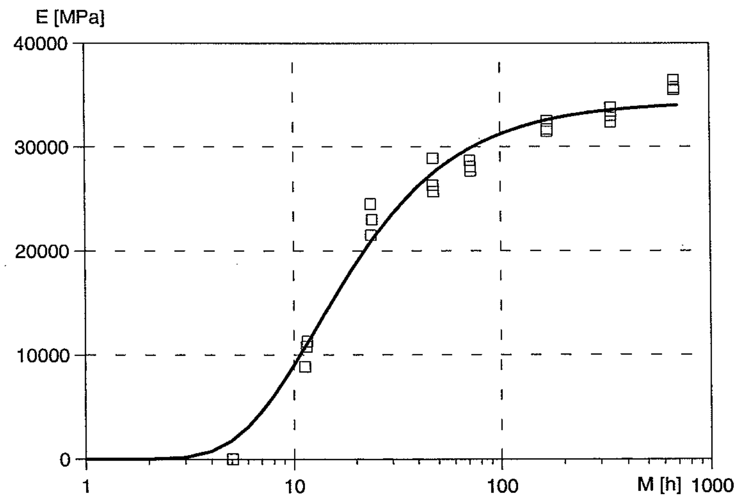

Figure 4.3: Example of development of E-Modulus E_∞ = 34330 MPa, τ_e = 12.89 h, α = 1.15

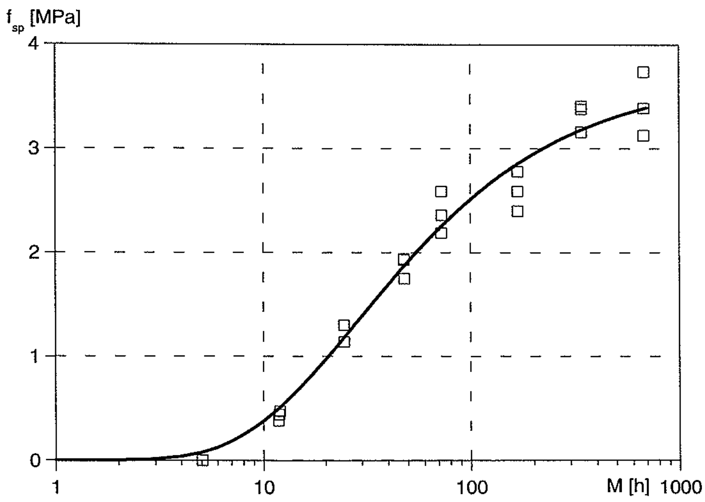

Figure 4.4: Example of development of splitting tensile strength

4.6 Poisson's ratio

Theoretically the elastic Poisson's ratio decreases from a value of 0.5 (for a liquid) to approximately 0.17 during the hardening process. Normally, tests are not carried out and the value of 0.17 is used for the whole hardening period.

4.7 Autogenous shrinkage

Concrete shrinks during the hardening process even if measures against drying and thereby against drying shrinkage have been taken. This shrinkage is designated autogenous shrinkage. The extent of this shrinkage is dependent on the concrete composition.

During the reaction between cement and water new solids are generated, the volume of which is smaller than the original volumes of cement and water. The result is a shrinkage - the so-called chemical shrinkage - at an early stage during the hardening process.

In concrete with a low w/c-ratio the capillary water is eventually used up by the hydration process and this gradually generates another form of shrinkage, the so-called self-desiccation shrinkage.

The chemical shrinkage is highest for high w/c-ratios while the self-desiccation shrinkage is highest for low w/c-ratios. A high content of fine-grained materials such as silica fume will increase the self-desiccation shrinkage.

Measurements made before the setting time (cf. example 4.2) show extreme shrinkage strains. These, however, have no significance in relation of the hardening strains because the concrete is still a liquid and cannot build up elastic stresses.

Shrinkage is normally determined by continuous measurement, so that its development during the hardening process is registered.

Example 4.5

Figure 4.5 shows the change of shrinkage measured according to [TI-B 102]. The shrinkage is given as a function of maturity.

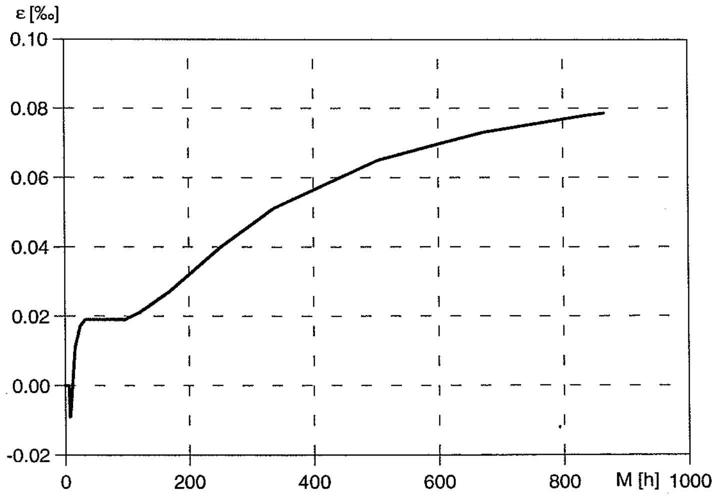

Figure 4.5: Example of autogenous shrinkage.

4.8 Creep

When a young concrete is exposed to a load, both permanent and reversible creep strains (cf. Figure 4.6) are observed. It is usually assumed that compressive and tensile creep are equal at the stresses that normally arise in hardening concrete structures. Linear visco-elastic creep models can normally be used.

The creep capacity of concrete is related to visco-elastic deformations in the cement paste and therefore decreases during hardening. Each point in a hardening structure goes through a stress process that starts in compression and finishes in tension or vice versa (cf. Figure 2.1) It is therefore necessary to register the behaviour of the concrete as regards permanent and reversible creep throughout the hardening period. In practice the creep properties are assumed to develop as a function of the hardening process (e.g. the maturity).

Poisson's ratio for creep is assumed to have the same value as the elastic Poisson's ratio.

Example 4.6

The test method [TI-B 102] recommends load histories for 3 concrete specimens as shown in Figure 4.6. The load histories 2 and 3 ensure that information on "reversible creep properties" are registered at appropriate times. Load history 1 describes a slowly increasing load corresponding to the change of stresses in a real concrete structure after the stress has passed zero.

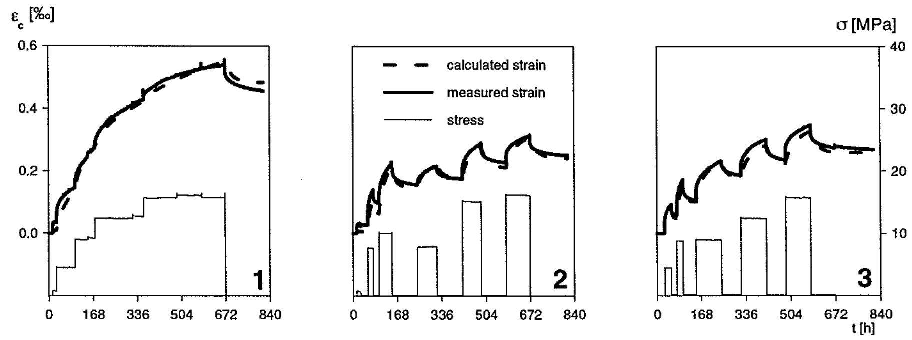

Figure 4.6: Registered and calculated creep strains and stress histories

The changes of strains shown in Figure 4.6 are described in Figure 4.7 by the spring and dashpot model. The permanent creep strains are described by the isolated dashpot while the reversible strains are described by the parallel-coupling.

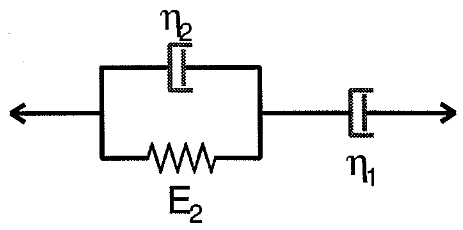

Figure 4.7 Model for creep strains

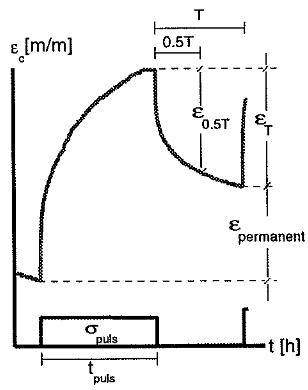

Figure 4.8 Terms used in (4.6) - (4.8)

By measuring the permanent deformation one load-pulse produces in the load histories 2 and 3, the viscosity of the isolated dashpot can be determined at a number of times to be:

$$\eta_1 = \frac{\sigma_{puls} \cdot t_{puls}}{\varepsilon_{permanent}} \tag{4.6}$$

where σ_{puls}, t_{puls} *og* $\varepsilon_{permanent}$ *are shown in Figure 4.8. Correspondingly, estimates for* E_2 *and* η_2 *can be determined by:*

$$\eta_2 = \frac{-E_2 \cdot T}{\ln\left(1 - \dfrac{\varepsilon_T \cdot E_2}{\sigma_{puls}}\right)} \tag{4.7}$$

$$E_2 = \frac{2 \cdot \varepsilon_{0.5T} - \varepsilon_T}{\varepsilon^2_{0.5T}} \cdot \sigma_{puls} \tag{4.8}$$

where T, ε_T, $\varepsilon_{0.5T}$ *og* σ_{puls} *are shown in Figure 4.8.*

Figure 4.9 shows the observed values of η_1, η_2 *og* E_2. *The curves represent the function:*

$$v = v_0 + v_\infty \exp[-(\tau_e / M)^\alpha] \tag{4.9}$$

where the values:

v_0	*the property at time zero*
v_∞	*the property at time approaching infinity*
τ_e	*the gradient parameter that determines the curve in time*
α	*the curvature parameter that describes how sharply the curve bends in the S-shape*

are determined by regression. M is the maturity.

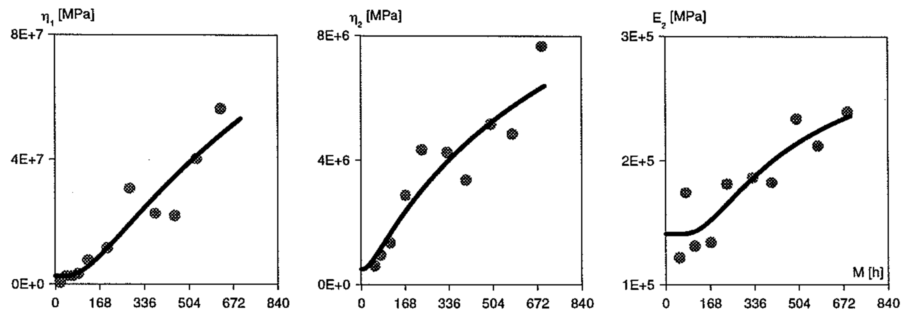

Figure 4.9: Development of creep properties.

Figure 4.6 shows the correlation between test and calculation model. The registration in load history 1 is not included in the establishment of the creep parameters and can therefore be used to assess the accuracy of the creep model.

A number of creep models are available. A common feature of the models is that the controlling parameters are established on the basis of testing. The quality of a creep model depends on the correlation obtained between observed and calculated changes of creep strains.

Attention should be given to the fact that a creep model may comprise so many parameters that it will straight away fit the registered process. This i however no guarantee that the creep model can simulate a change in creep strains which is <u>not</u> included in the stipulated control parameters.

5.0 Planning the execution

When a project is planned an execution method must be chosen that lives up to the requirements for hardening. The requirements can cf. Chapter 7 be based on:

- maximum temperature differences
- maximum utilization of tensile strength
- maximum crack widths.

During the planning stage calculations can be carried out, based on the concrete properties and a chosen strategy, that indicate whether it will be possible to satisfy these requirements. The strategy includes:

- location of construction joints
- casting schedule
- choice of form materials
- times at which formwork is removed
- use of insulation
- use of cooling pipes/heating cables
- use of cold/warm concrete
- etc.

Any given requirement may be satisfied in many ways, so the contractor has the freedom to choose the method that is most appropriate for the given project.

The above list is not arranged in order of priority.

5.1 Strategies for control of crack formation

Cracks may be generated:

- when the difference in thermal strains between adjacent structural components is too large

- when the difference in thermal strains within a structure is too large

- when a support form is removed and the structure shall carry its own weight.

Example 5.1

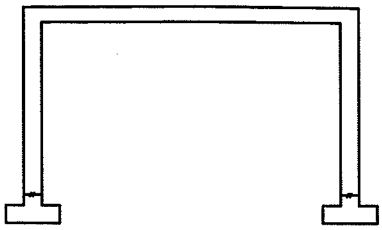

Figure 5.1: Tunnel unit.

The tunnel unit shown in Figure 5.1 may present all the above-mentioned types of crack. Temporary surface cracks may form in the thick structural components when the concrete reaches its maximum temperature and/or when the formwork is removed from a warm structure that is momentarily cooled at the surfaces (Figure 5.2). The crack pattern can be more randomly distributed than shown in the Figure.

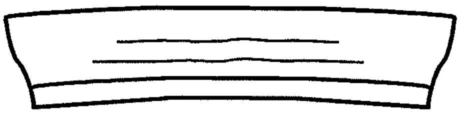

Figure 5.2: Surface cracks

Figure 5.3: Through-going cracks at the top of the wall

Temporary through-going cracks can form at the top of the wall (and in the deck if it is cast at the same time) when the concrete temperature is a maximum. This crack type is observed primarily in the end zones as shown in Figure 5.3.

During the cooling stage permanent through-going cracks can be formed in the wall as well as in the deck, cf. Figure 5.4.

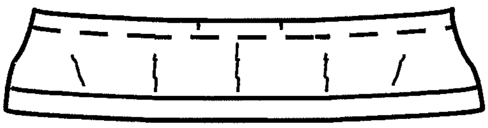

Figure 5.4: Through-going cracks in walls and deck

Also thermal strains in the plane of the cross-section may cause crack formation. Because the deck expands transversely, cracks as shown in Figure 5.5 may form.

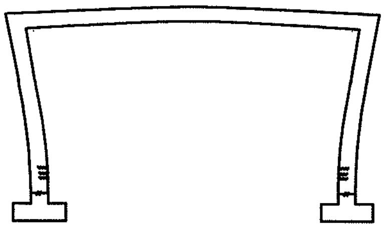

Figure 5.5: Cracks caused by transverse strains in the deck.

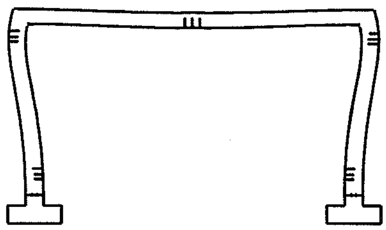

Figure 5.6: Cracks caused by dead load.

When the shuttering at the underside of the deck is removed, the dead load is taken up by frame action in the tunnel cross-section. If the tensile strength is too small, the result will be crack formation (Figure 5.6).

Also the previously cast structural components may be damaged: A foundation that is slender in relation to the cast-on wall may be pulled apart on account of the thermal expansion of the wall.

If the difference in material thicknesses is large the result may be temperature differences between components in the same casting and cracks may form.

5.1.1 Reduction of risk of through-going cracks

Cracks generated on account of different thermal strains in adjacent structural components can be prevented by:

- location of construction joints
- choice of casting schedule
- control of temperature.

It is normally an advantage to cast the structures with the fewest possible construction joints (and thereby casting sections), because the number of adjacent castings is hereby reduced. The location of construction joints can be specified by the owner.

Example 5.1

Figure 5.7: Walls and deck cast in one casting

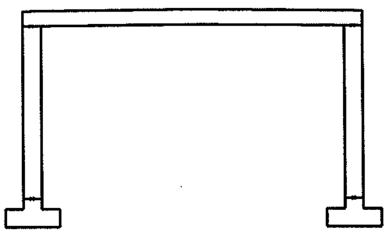

Figure 5.8: Walls and deck cast in two castings

The walls and deck for a tunnel structure may be cast in one or two castings (Figures 5.7-5.8).

Casting without construction joints between walls and deck may reduce the risk of crack formation in the deck because walls and deck are more likely to cool at the same rate.

By calculating the effects of different locations of construction joints it can be decided how the casting can be planned most appropriately, i.e. with a minimum of measures to avoid formation of hardening cracks.

It must however be carefully considered if the chosen arrangement of casting sections is appropriate for execution. The following should be considered:

- Do the casting sections have a suitable size?
- Is the wall so high that a correct casting of the lower part is impossible?
- Will the work schedule for removal of formwork be appropriate?

Control of temperature is carried out i.a. by the use of form and insulation materials which in combination with suitable form periods will result in appropriate temperature developments. If necessary, the following measures can be taken:

- Heating of adjacent structural components, for instance by means of electrical heating cables, cf. Section 5.3.5
- Cold concrete or cooling pipes during casting, cf. Section 5.3.6
- Retarding the first part of a casting by means of additives.

5.1.2 Reduction of risk of surface cracking

The risk of formation of surface cracks can be reduced by insulating the surfaces. This reduces the temperature difference between the core of the structure and the surfaces but increases the temperature generally. The higher temperature level will mean:

- the stripping must be postponed until the concrete has reached a temperature at which the risk of surface cracking is eliminated
- the thermal strains in the structural component will be greater in relation to adjacent structures which increases the risk of formation of other types of crack

The use of cold concrete or cooling by means of pipes placed in the form can also reduce the risk of surface cracking. Such a solution retards the hardening process and thereby postpones stripping, removal of drying protection and cable tensioning.

5.2 Testing of concrete properties

During the planning stage calculations are carried out which involve the concrete properties. Two different test programmes, for temperature and stress respectively, are available (cf. Chapter 4.1) Normally the project specifications state the tests to be carried out. The testing is usually performed by the supplier of the concrete, but shrinkage and creep tests are typically made by external laboratories. The testing including report takes approximately 3 weeks for temperature calculations and approximately 6 weeks for stress calculations. The cost of the two types of tests are DKK 5,000 and DKK 100-150,000 respectively (1996 prices).

For major projects where the costs for control of hardening are correspondingly high it might be advantageous to examine several concrete mixes with a view to selecting a concrete with the appropriate characteristics.

5.3 Planning based on calculation of the hardening process

5.3.1 Planning the execution

In connection with the planning of the execution of concrete casting the following can be decided upon:

- location of construction joints
- casting schedule
- choice of form
- time for stripping
- use of insulation
- use of cold/warm concrete
- use of cooling pipes/heating cables

The planning shall also include:

- concrete properties
- the expected weather conditions during casting and hardening
- support conditions
- requirements for drying-/frost-protection

The result of the planning shall be:

- a detailed specification of the work as regards the casting
- the specification of the extent of documentation of the casting (for instance number and location of thermo-sensors)

5.3.2 Casting form

The choice of casting form depends on the requirements for insulation properties, surface structure and geometry (cf. Table 5.1) all of which may be specified by the owner. The possibility of reusing the form is also important for the choice of form material. Steel forms can be reused more than wooden forms.

The most commonly used form materials for construction work are wood, casting plates (plywood or surface-treated casting plates), steel or a combination of these materials.

Form material	Aesthetics	Insulating capacity	Re-use
Steel	Smooth surface	No insulation	High
Plywood/casting plates	Smooth surface	Moderate insulation	Moderate
Casting boards	Rustic surface	High insulation	Low

Table 5.1

5.3.2.1 Surface / aesthetics

Requirements for the appearance of the finished surface determine the choice of form material. If a smooth surface is wanted the choice will be plywood or steel, and if for example the finished surface is to have a "plank" structure, casting boards are chosen.

5.3.2.2 Geometry

The casting pressure, vertical or horizontal, must be taken into account when the form is built, so that the stiffness of the form ensures a structure with the planned geometry.

5.3.2.3 Insulating capacity

The form materials have different insulating capacities, cf. Figure 5.9 and Appendix A.

Casting plate / plywood:

Casting plates / plywood are used for both vertical and horizontal casting and can be acquired as standard form systems. The material thickness ranges typically from 18 -21 mm and the insulating capacity is moderate. The dissipation of heat to the surroundings is appropriate, so that the temperature differences and levels are normally moderate. It can be necessary to insulate the form in winter in order to control the hardening process.

Casting boards on plywood:

Casting boards on plywood are typically used for vertical castings of considerable height and the total thickness is approximately 50 mm with a relatively high insulating capacity. The form is often symmetrically built and therefore the dissipation of heat to the surroundings is hampered, which means that the temperature profile is evenly distributed over the cross-section of the structure as well as a higher maximum temperature.

Casting boards:

Casting boards are normally used for the underside of horizontal castings, for instance the underside of a bridge. The form structure resembles a floor construction in a house, i.e. boards on joists. The normal thickness is approximately 30 mm and the insulation capacity is relatively good. It is difficult for the hydration heat to dissipate to the surroundings which i.a. means an evenly distributed temperature profile over the cross-section of the structure and a higher maximum temperature.

Steel forms:
Steel are typically used in connection with casting of "standard" structures. The form construction usually corresponds to the construction of standard systems of ordinary casting plates. The standard thickness is between 6 and 8 mm. The thickness is however of secondary importance because the insulating capacity is practically zero.

The poor insulating capacity of the steel form means that the hydration heat easily escapes which especially in the case of thick structures creates an inappropriate temperature profile over the cross-section of the structure (high temperature difference between the core and the surfaces) and a reduction of the maximum temperature. The steel form provides great flexibility for control of the hardening process, because a suitable amount of insulation can be placed in order to obtain the most appropriate temperature profile. In practice it is however difficult to insulate the forms.

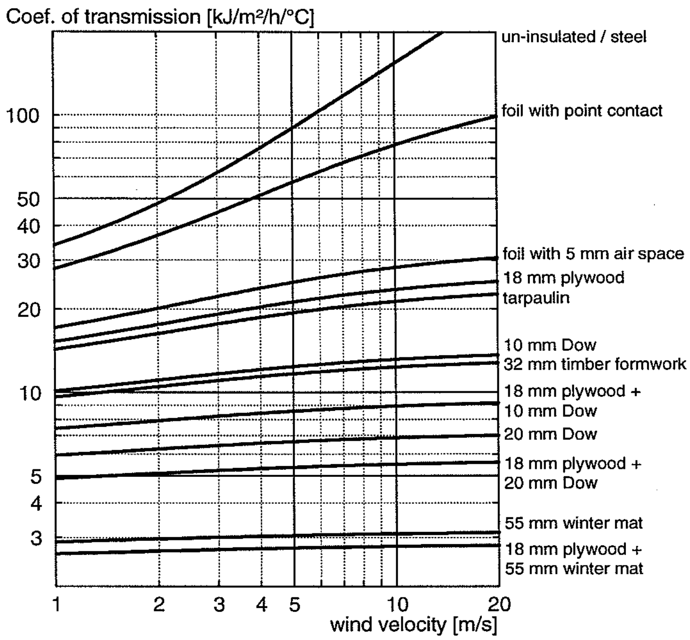

Figure 5.9: Coefficient of transmission as a function of wind velocity.

5.3.2.4 The colour of the form
The colour of the form materials influences the heat by radiation during the hardening process. A dark form absorbs more heat from radiation than a light form. During the day the sun supplies heat to the form by radiation; the energy supply depends on the season, cloudiness, the position of the sun and the orientation of the form. During the night the opposite takes place - heat radiates from the concrete/form. Even if the effect from radiation can be measured on a hardening concrete surface, it is difficult to determine the extent of such radiation in advance.

If calculations show that only small changes in the hardening process can be accepted before the requirements are exceeded, the creation of shadow for the structure should be considered, or the use of white form materials.

Care should be taken to use light materials for covering freshly cast surfaces (cf. guideline [HETEK-Curing, 1997]).

5.3.3 Insulation

In order to control the hardening process, cast concrete surfaces, casting forms or stripped surfaces can be insulated. The most commonly used insulating materials are:

- Mineral wool mats
- Expanded/extruded polystyrene
- Plastic film or tarpaulins

The insulating capacity of the materials differ, cf. Figure 5.9 and Appendix A. The figure shows that the insulating capacity of insulated forms is not significantly affected by the wind velocity.

5.3.3.1 Insulation of free concrete surfaces

Freshly cast concrete cannot carry winter mats without their leaving an imprint on the surface. Therefore a lighter insulation, e.g. expanded polystyrene, is placed during the first day or two and winter mats can be used later.

The contractor can insulate free surfaces in order to optimize the time for stripping or tensioning of a structure even if the hardening process would meet the requirements without an insulation.

The insulation will also protect against evaporation from the surface (cf. guideline [HETEK-Curing, 1997]).

5.3.3.2 Insulation of forms

Insulation of forms shall be made on the outside of the bracing system in order to get the best result. Insulation is often sufficient in the form of tarpaulins placed on the outside of the form so that a cavity of static air is created. This solution is easy to perform and strong. It is **important** to make sure that draught cannot make its way under the tarpaulins.

If winter mats are used care should be taken that they fit close to the form by building a frame system.

5.3.3.3 Insulation of stripped surfaces

Stripped surfaces are insulated:

- to get an even temperature profile over the cross section, se that the requirement for D_{int} is met

- to optimize the use of the forms and at the same time obtain an even temperature profile so that the requirement for D_{int} is met. A good insulation typically complements the effect of the form

- when the form is removed prior to fulfilment of the hardening requirements. In the case of thick structures an insulation with a lower insulating capacity than the form may be used to reduce the maximum temperature and at the same time ensure the fulfilment of the requirement for D_{int}. The method is a combination of the above-mentioned

- to retain the hydration heat in order to minimize the risk of formation of through going cracks in the subsequent castings. In the event of postponements in the casting programme the temperature will continue to drop in the hardening structure, which unfavourably affects the possibility of meeting the requirement for D_{ext}. This should be considered before this method is applied.

Example 5.2

If relatively well insulating forms (for instance 50 mm wood) are used the stripping can be carried out early to reduce the maximum temperature and then an insulation is placed with a lower insulating capacity than the form, for example one or two layers of tarpaulins. This procedure will meet the requirement for D_{ext} *as well as the requirement for* D_{int}.

5.3.4 Casting temperature of concrete

A significant parameter for the temperature level is the casting temperature of the concrete. It is typically assumed that a 1 °C rise in the casting temperature results in a maximum temperature that is 1 °C higher.

The casting temperature varies with the air temperature and the time of day at which the concrete is delivered. The concrete temperature also depends on whether it was mixed in one batch or mixed successively in a warm concrete mixer. A rule-of-thumb is that the concrete temperature is approximately 2-5 °C higher than the average 24-hour temperature except for concrete heated during mixing.

Summer
Concrete delivered from a factory on a hot day may reach a temperature of 25-30 °C. The casting temperature can be reduced by adding liquid nitrogen into the mixing drum, but this is costly.

Winter
Without initiation of special measures the concrete temperature will normally not drop below approximately 7-8 °C. Concrete can be delivered hot with temperatures of typically 13-18 °C depending on the transportation distance. The hot concrete is produced by addition of hot water which may raise the temperature by 5 °C. The use of aggregate from heated silos may also contribute to a rise in temperature, but the

production capacity is limited by the silo capacity. Steam can also be used to raise the temperature but it is difficult to control and document the amount of water used.

Spring and autumn
The concrete temperature is typically in the range 11-18°C.

5.3.5 Heating

The risk of formation of through-going cracks when casting against a previously cast structural component can be reduced by heating. If the previously cast structural component is heated prior to casting and is then cooled concurrently with the casting, the difference between the temperature strains is reduced.

<u>The structural component to be heated must not be prevented from expanding or contracting by adjacent structures or the foundation. Foundations such as solid rock or limestone and structures under water are very difficult to heat.</u>

Calculations should be performed in order to check that the heating does not give cause for crack formations in the structural component to be heated.

Heat can be supplied externally or internally and can be initiated exactly when required and for exactly the necessary period of time during the hardening process. Normally the heating is started days or hours before the casting and is continued until the hardening structure has reached a suitable temperature. The total heating period is typically from 2 to 4 days depending on the structure.

<u>Heating shall be stopped when the freshly cast structural component has reached its maximum average temperature. Otherwise the heating is pointless.</u>

External heating is typically used as an emergency solution in cases where heating has not been planned. External heating takes place from the surface of the concrete for instance in the form of infrared light or ordinary light (for instance halogen), heaters or electric heating mats. The method is less efficient than internal heating. The best way of guarding against damage to the concrete surface is to place a thermo-sensor at the surface.

Internal heating is a planned hardening procedure scheduled to be carried out with the casting. It is carried out either in the form of cast-in electrical heating cables/wires or by using cast-in pipes to be heated by means of hot water. At the same time the external surfaces should be insulated so that the cross-section of the structure is evenly heated. The heating process is best controlled by means of thermo-sensors in the concrete.

Heating can also be used when:

- the hardening process shall be accelerated, e.g. on account of strength development and curing
- construction joints shall be heated to be frost-proof

- there is a risk of freezing of fresh concrete
- a hardened structure shall be heated to +5 °C before prestressing cable ducts are injected.

5.3.5.1 Heating cables/wires

Internal heating can be carried out by means of heating cables (220V/380V) or heating wires (40-50V). Heating cables are connected directly to mains while transformers are needed for the heating wires. The effect of heating cables depends on the type of cable used, while the effect of heating wires can be adjusted by means of the transformers. Both types yield (and consume) normally 10 - 50 W per meter cable/wire. Depending on the applied voltage U and the resistance R the output P can be determined as follows:

$$P = \frac{U^2}{R} \quad [\mathrm{W}] \qquad (5.1)$$

The resistance R increases as the temperature rises. If the resistance at 15 °C is R_{15} the resistance at temperature T can be estimated at:

$$R_T = R_{15} \cdot (1 + 0.004 \cdot (T - 15)) \quad [\Omega] \qquad (5.2)$$

Normally heating cables/wires do not stand temperatures over 60 °C. Cf. formula (5.2) the resistance at 60 °C is increased by 18% compared with the resistance at 15 °C. According to formula (5.1) a correspondingly smaller output is obtained in the concrete. Furthermore, heating wires lose a certain output to coupling-boxes, switches, etc.

Heating cables/wires are typically placed at intervals of 20 - 40 cm depending on the heating requirement. Very often it will be possible to place cables/wires directly on the reinforcement bars by means of plastic binders. Cables/wires must not be in contact with each other or with insulating materials. Most types require complete encasement in order not to become too hot and thereby destroyed. A possible surplus can be covered with wet sand. If the cables/wires are shortened the resistance is reduced and the output increased correspondingly. After installation the cables/wires should be turned on for a few seconds, and afterwards the resistance registered with an ohmmeter. The actual output can then be determined according to formula (5.1).

The total amount of heat to be supplied depends on:

- the amount of concrete to be heated
- the desired rise in temperature
- the heat loss to the surrounding air
- the heat loss to the adjacent structures including the sub-base

The necessary output depends on the total quantity of heat to be supplied and the duration of the heating period. If the heating takes place too quickly there is a risk of generation of cracks in the heated structural component. And the fewer the number of heating wires, the greater the risk of crack formation. This risk can be assessed by stress calculations.

When casting directly on soil the calculation shall include the amount of soil affected by the heating. By choosing the initial and boundary conditions it should be ensured that the average temperature in the heated structural component is not lower at the termination of the heating phase than at the start.

Example 5.3

A foundation on soil shall be heated so that the average temperature is raised by 10 °C. The cross-section appears from Figure 5.10. During summer the air temperature is assumed to be 25 °C and the soil temperature 11 °C. In order to establish a "correct" temperature distribution between the air and the soil the temperature calculation is started before the heating cables are turned on. The initial temperature of the foundation (25 °C) is hereby reduced to 22 °C after 60 hours (cf. Figure 5.11) corresponding to reaching a quasi-stationary state. At this time the foundation is insulated with winter mats and 12 heating wires with a total output of 396 W per meter foundation are turned on. After 3 days the average temperature has risen by 10 °C and the heating wires are turned off. The time for casting is planned so that the two structural components reach the maximum temperature at the same time. The cooling process is controlled by reducing the amount of insulation so that the thermal strains follow those in the cast-on wall.

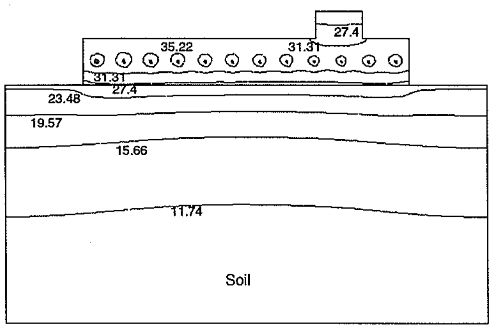

Figure 5.10: Temperature distribution in foundation and soil at time t=132 hours [°C]

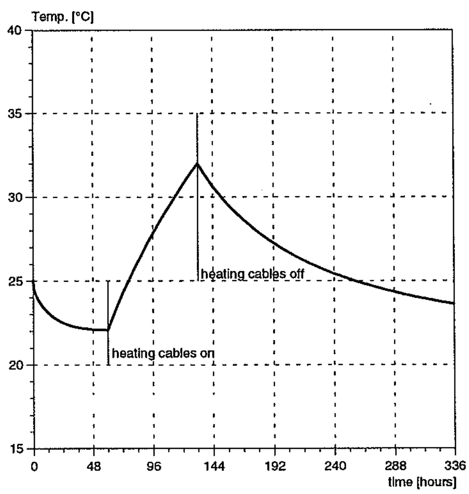

Figure 5.11: Average temperature in foundation

5.3.6 Cooling

In thick-walled structures the risk of generation of surface cracks during the heating stage can be reduced by cooling.

Cooling can also be used to reduce the thermal strains of a structural component in relation to adjacent structures, thereby reducing the risk of through-going cracks.

Cooling of concrete is typically carried out by embedding pipes in which water with a lower temperature than the concrete is circulated. The embedded pipes are usually of plastic or metal and are placed at intervals of 0.3 - 0.6 m depending on the desired cooling effect. A doubling of the distance reduces the effect to a quarter.

It is generally assumed that the risk of surface cracking during the heating phase is reduced when the pipes are placed in the areas of the structure where the temperature will be highest in the uncooled structure. Correspondingly, the risk of cracks in the cooling phase is reduced when the pipes are placed in the areas where the risk of cracking is highest in the uncooled structure. An optimum location of the cooling pipes is determined on the basis of a stress calculation.

Heat is developed at the highest rate at the beginning of the hardening process, and the cooling shall take place during this period. Cooling is started from the time in the casting procedure when the concrete has covered the cooling pipes. During the process the flow of water and/or the temperature of the cooling water can be adjusted so as to control the cooling process. This control can be carried out according to temperature registrations in the concrete, the inlet and outlet temperatures of the cooling water and the flow. It is advisable to build in alarm systems to register failure of the cooling system.

Normally the cooling can be stopped when the concrete temperature has fallen 2-3 °C below the maximum. It is usually appropriate to continue the cooling until the time when the maximum temperature would have been reached if no cooling had been performed. The termination of the cooling at an early stage will reduce the local eigen-strains around the cooling pipes.

The planning of the temperature development normally requires 2-dimensional calculation models.

Cooling water can be:

- groundwater, water from lakes and rivers and seawater. In order to prevent a blocking of the pipes by impurities filters should be used. The cooling can be controlled by the flow velocity. It is assumed that the groundwater is at 8 °C, but the temperature should be checked well in advance of casting. The groundwater may be considerably warmer depending on the groundwater level and how close to the coast the water is pumped.

- water cooled by refrigerating plant. The cooling is controlled by a combination of the inlet temperature and the flow velocity.

If water is transported over long distances it must be ensured that the water temperature is not significantly affected by insolation or frost. Frost may cause both interruption of the cooling and cracking in the concrete. If saline water is used, cooling pipes of steel shall be rinsed with fresh water in order to minimize the risk of corrosion. In structures exposed to frost the pipes shall be blown through with a frost-resistant liquid before injection. Alternatively the cooling pipes can be provided with drainage tubes to prevent filling with water.

The tigthness of the cooling system should be tested by pressure testing before casting.

Appendix A shows how the coefficient of transmission of cooling pipes is determined.

Example 5.4

The maximum average temperature in the bottom half of a 0.5 m thick wall shall be lowered by approximately 10 °C in order to reduce the thermal strains in relation to the foundation (cf. Figure 5.12). Plastic cooling pipes are placed at intervals of 0.5 m in the centre line of the wall. The length of the cooling pipe is approximately 120 m and the flow is 2.4 m^3/h. When the water flows through the pipe the temperature of the cooling water rises. It has been assessed that the average temperature of the cooling water is 15 °C. Figure 5.13 shows the development of the average temperatures in wall areas with and without cooling, and it appears that the maximum is reduced from 48 °C to 38.5 °C in the cooled area. Figure 5.13 shows that the temperature of the concrete around the cooling pipe is approximately 20 °C.

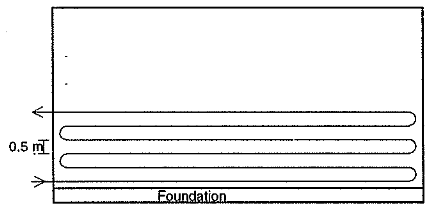

Figure 5.12: Sketch of wall with cooling pipes

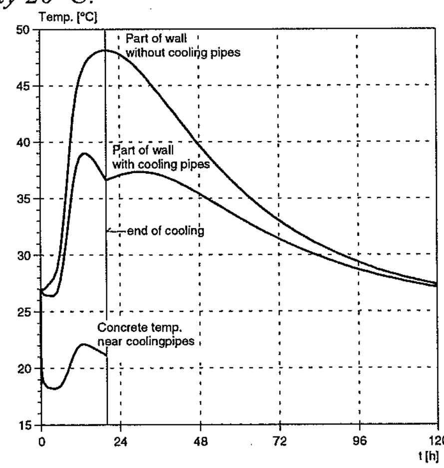

Figure 5.13: Average temperature in wall with and without cooling pipes and concrete temperature around cooling pipes. The cooling pipes are turned off 20 hours after casting.

The necessary inlet temperature can be determined from:

$$T_{outlet} = T_{concrete} - \frac{(T_{concrete} - T_{average}) \cdot k}{(1 - e^{-k})} = 13.3°C \tag{5.3}$$

where

$$k = \frac{\pi \cdot \alpha \cdot d \cdot s}{c_{water} \cdot Q} = 0.604 \tag{5.4}$$

$T_{concrete}$	*is the concrete temperature (20 °C)*
$T_{average}$	*is the average temperature of cooling water (15 °C)*
α	*is the coefficient of transmission for the wall with cooling pipes, t=2mm (576 kJ/m²/ °C/h)*
d	*is the internal diameter of cooling pipe (0.028m)*
s	*is the length of cooling pipe (120m)*
c_{water}	*is the specific heat of water (4192kJ/m³/ °C)*
Q	*is the flow (2.4 m³/h)*

2.4 m3 of water at a temperature of 13.3 °C or less shall thus be supplied per hour. If the water is delivered from a refrigeration plant the necessary output can be determined as:

$$P = (T_{outlet} - T_{inlet}) \cdot c_{water} \cdot Q = 30182 \text{ kJ/h} \tag{5.5}$$

where

$$T_{outlet} = T_{concrete} + (T_{inlet} - T_{concrete}) \cdot e^{-k} = 16.3°C \tag{5.6}$$

5.3.7 Weather conditions

During the planning of the concrete casting it shall be decided on which weather conditions should be investigated in the temperature and possible stress calculations. The definition of weather conditions should include outdoor temperature, wind velocity and possibly insolation. And generally the weather conditions influence greatly the temperature of the fresh concrete, cf. Section 5.3.4.

Information on weather conditions in Denmark is given in Appendix B.

When casting at a specific time of the year, it will normally be necessary only to investigate two climatic situations, i.e. a cold and a warm situation for the relevant season.

When casting several identical structural components over a long period of time, it will in most cases be appropriate to investigate two climatic situations corresponding to a winter and a summer situation. Possibly a weather condition in between corresponding to autumn/spring should be investigated, too.

The planning should result in acceptable threshold values for the relevant parameters, for instance the temperature of the fresh concrete, the outdoor temperature, the cooling water temperatures, etc. cf. Section 6.1.4.

It appears from the above that conservative assumptions for weather conditions provide a wider margin for the different parameters, which again means better security for the performance of the casting as planned. On the other hand the costs are increased.

It should be noted that the safety factor as regards the choice of weather conditions should be increased for structures without cooling, because the possibility of influencing the temperature development in the structure is very small once casting has started.

5.3.8 Modelling of support conditions

The choice of support conditions is - as in a traditional structural calculation - important for stress calculations of hardening concrete structures.

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, foundations.

Figure 5.14 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.

In order to calculate the reaction distributions a 3-dimensional analysis is needed in which the sub-base E-modulus, Poisson's ratio, angle of friction and cohesion are taken into account. As it is very complicated to carry out such calculations, assumptions on the support conditions on the safe side must be made in practical applications. The involves that an assessment shall be made of whether:

- the structure can move perpendicularly to the plane of the support (can bend)
- the structure can move in the plane of the support

Long structures include cross-sections that can bend freely and cross-sections that are fixed in a rectilinear geometry on account of the dead load, cf. Figure 5.14. It cannot be decided in advance which situation is the most dangerous for a given point in the structure at a given time, and therefore both situations should be assessed.

Short structures do not include cross-sections with a fixed curvature, cf. Figure 5.14, because the dead load is of less importance.

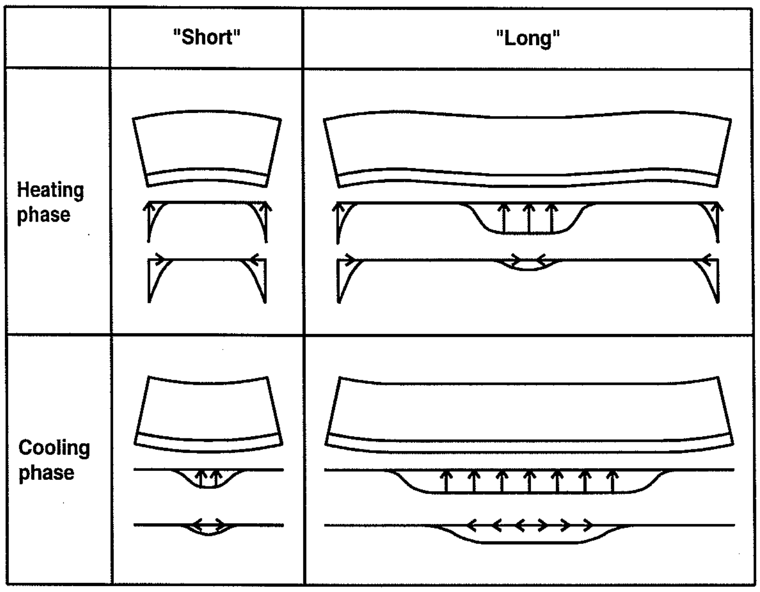

Figure 5.14: Principal reaction distributions in a tunnel seen in elevation

It is however a problem to determine whether a structure is "short" or "long", but it is possible to indicate a distinction between the two. The transition between short and long is somewhere around a length/height ratio of 3. Structures with a length/height ratio greater than 3 are normally assumed to be long. The ratio 3 is not the final value and cases may arise where the ratio is greater or smaller, cf. [Andersen et al,1977]. When casting directly on limestone or solid rock, also short structures will be locked against the sub-base. This can be accounted for by assuming that the support is a structural component to be cast on, and by including it in the stress calculation.

The support conditions concerning movements in the plane of the support surface are established by balancing the forces that are produced by the support in relation to the forces that the hardening structure can exert on the support.

Example 5.5

Normally tunnel structures are divided into sections separated by expansion joints. Such structures are usually considered not to be fixed to the support against horizontal movements perpendicular to the plane of the cross-section caused by thermal and shrinkage strains. This is due to the fact that the friction forces that can build up within the section do not reach a magnitude where they contribute to the stresses in the tunnel cross-section. This is not the case when casting is carried out directly on limestone or solid rock, just as the influence of friction forces should be included for very long structures.

In the case of support conditions that are important to the stresses in the plane of a tunnel cross section matters are different. As the tunnel walls are relatively flexible (cf. Figure 5.5) it is normally not possible to establish forces in the sub-base surface strong enough to bring about a significant movement. Therefore the sub-base can be assumed to be fixed against horizontal movement in the plane of the cross-section.

5.4 Trial testing of the execution

It is recommended to carry out a trial casting. As concerns the hardening such trial casting is used to

- test whether the planned measures can be carried out
- perform temperature measurements
- observe crack formation
- verify the tools used during the planning to predict temperature and crack formation

After the trial casting an assessment can be made, cf Chapter 6.3 and if necessary the planning is revised.

6.0 Documentation

This chapter treats the documentation that is part of the planning and execution of the concrete works and that should be carried out if control of cracking is required. The following should be documented:

- Calculated planning of the hardening process
- Detailed description of the work
- Registrations and observations during casting
- Registrations and observations during and after hardening
- Assessment in relation to the specified requirements.

6.1 Planning

Calculations shall demonstrate that a planned method for the execution will result in a hardening process that meets the requirements that have been stipulated for temperatures, maturity (on account of drying protection, stripping strength) utilization of tensile strength and/or crack widths. If requirements have been set for the utilization of the tensile strength and/or cracks widths the contractor shall carry out calculations that indicate the temperature differences the structure can stand without exceeding the requirements.

The task is then to plan an appropriate execution that will result in a temperature development in the structure that satisfies the stipulated requirements for maximum temperature differences. Furthermore it should be estimated to what extent a change in the planning assumptions will reduce the possibility of meeting the requirements.

6.1.1 Temperature calculation

When calculated temperatures are presented the points stated in the list below should be included:

Calculation basis:

- geometrical model (sketches with dimensions marked)
- models for thermal properties of concrete (adiabatic heat development, thermal conductivity, specific heat and density)
- weather conditions
- temperature of fresh concrete
- boundary conditions (for instance coefficients of transmission and their variation in time)
- systems for control of temperature (cooling or heating systems)

Calculation results:

- the change in average temperature of the various structural components as a function of time
- development of the external temperature difference (D_{ext}) as a function of time
- development of the internal temperature difference (D_{int}) as a function of time
- isoquants for the time of maximum internal temperature difference (D_{int})
- isoquants for the time of maximum external temperature difference (D_{ext})
- isoquants that show where and when the maximum temperature is registered
- isoquants that show where low temperatures occur at the surface in connection with a possible risk of early frost
- development of the maturity at the surface as a function of time

6.1.2 Stress calculation

The execution method that satisfies the requirement for maximum utilization of the tensile strength shall be supplemented by a temperature calculation that describes how the temperature at every point is expected the change during the hardening process. On this basis temperature requirements are specified that shall apply to the actual casting. These temperature requirements shall replace the requirement for maximum utilization of the tensile strength.

Description of the basis of calculations apart from what has been stated for the temperature calculations:

- model of the support conditions
- model of dead load and external load
- models of the mechanical properties of the concrete (coefficient of thermal expansion, E-modulus, Poisson's ratio, creep, autogenous shrinkage, tensile strength)

Description of the result of stress calculations:

- development of stress and utilization of tensile strength at critical points as a function of time
- isoquants for stress and utilization of tensile strength at times when the utilization of the tensile strength is extreme, e.g. at the time of maximum utilization of the tensile strength at the surface (heating phase and stripping) and at the time of maximum risk of formation of through-going cracks (cooling phase)

6.1.3 Calculation of crack width

Crack widths are calculated on the basis given in the specifications. A calculation of the crack width includes i.a. calculation of the hardening temperature.

The execution that satisfies the requirement for maximum crack width shall be supplemented by a temperature calculation where the expected temperature development at every point during the hardening process is described. On this basis the temperature requirements are specified that shall apply to the actual casting. These temperature requirements shall replace the requirements for crack width.

6.1.4 Detailed working description

On the basis of the planning the following shall be specified:

- assumed casting temperature and weather conditions
- form types, insulation
- stripping times
- location of cooling pipes/heating cables
- start-/stop-times of cooling/heating
- drying protection
- location of thermo-sensors
- temperature requirements

The working description shall allow for deviations from the assumptions made during the planning. Deviations in the casting temperature, weather conditions, etc. may result in temperature developments that do not correspond with the planning. During the hardening process the registered temperatures are used to assess frost resistance, maturity and stripping strength. On this basis it might be necessary to change the time for stripping, drying protection and start/stop of cooling/heating. Normal procedure is described in the literature and shall not be treated in further detail. Reference is made to [Vinterstøbning af beton, 1985].

The weather forecast should be followed closely in the days before a casting in order to decide on possible additional calculations. On the day of the casting the following assumptions should be checked before the casting is started:

- temperature of fresh concrete
- outdoor temperature (expected average, minimum and maximum)
- temperature of cooling water
- wind velocity

The casting should not be started if the assumptions for the planning are not fulfilled. The assumptions should be continuously monitored during casting and hardening.

6.1.5 Documentation of temperatures during execution

Irrespective of specified requirements for temperatures, utilization of tensile strength or crack width the documentation shall be carried out by measuring temperatures in the hardening structure. In the case of castings between new and old concrete structures, the temperature of the old concrete is also registered.

It is not possible to set up guidelines that cover all situations for location of thermo-sensors as this depends very much on the complexity of the structures. Normally the thermo-sensors are placed at the points where extreme temperatures are expected. These will typically be the core of the structural component and the surfaces. The thermo-sensors are normally placed 10 mm from the surface, the location should be based on temperature calculations that have been carried out during the planning.

The thermo-sensors might be damaged during casting and in order to minimize that risk the thermo-sensor can be placed on a spacer block. It should be placed on the side of the spacer block to avoid it lying in an air pocket under the block, which would result in inaccurate registrations. It is especially important to know the location of thermo-sensors in the cover; as the highest temperature gradients are normally observed in the surface area, even a minor misplacing of the thermo-sensors will result in a significant misregistration. If the thermo-sensor is placed by means of the reinforcement bar the sensor should not be placed in direct contact with the mesh, as this might cause electrical disturbances. The thermo-sensors might for instance be placed on a non-conductive bar fixed in the form.

All sensors shall be numbered on a drawing that shows the locations. The sensors are numbered distinctly at both ends of the wire so that they cannot be exchanged by mistake with the result that the temperatures cannot be documented, cf. Figure 6.1. Embedded thermo-sensors shall be protected with e.g. shrink film to prevent transmission of current and subsequent misregistration.

One or more thermo-sensors should register the outdoor temperature at the structure during the hardening process.

Figure 6.1: The thermo-sensors are numbered at both ends [Vinterstøbning, 1985]

6.1.5.1 Thermo-sensors in structures without cooling pipes/heating wires

Figure 6.2 shows a typical location of thermo-sensors in a structure without cooling pipes. Locations 1-6 make it possible to document the requirements for temperature differences between the core of the cross-section and the surfaces. Sensor No. 2 is also used to document the maximum temperature of the cross-section.

Sensors 1, 3, 4, 5 and 6 may be used to determine the maturity of the concrete surface and thus the time for stripping.

The documentation of temperature differences between adjacent structural components requires that thermo-sensors be placed in both structures. Heat transmission from the new concrete to the old by construction joints causes a local heating of the old concrete in the area under the construction joint. If only one sensor is placed in the old structure it should be placed at a point where the temperature is not affected by the casting. If several sensors are placed it will be possible to determine how the average temperature in the old structure varies according to the hardening process in the new structure. Very often thermo-sensors will have been installed in the old structure for documentation purposes.

If requirements are made for a limited temperature difference between the old and new concrete and there are no thermo-sensors in the old concrete, these can be installed in drilled holes at the desired depth. The drilled hole should be injected to get a reliable temperature registration. As the injection mortar also develops heat during hardening, the drilled-in thermo-sensors shall be inserted well in advance of the casting so that the heat of hydration of the injection mortar is not measured, which would result in an underestimation of the temperature differences.

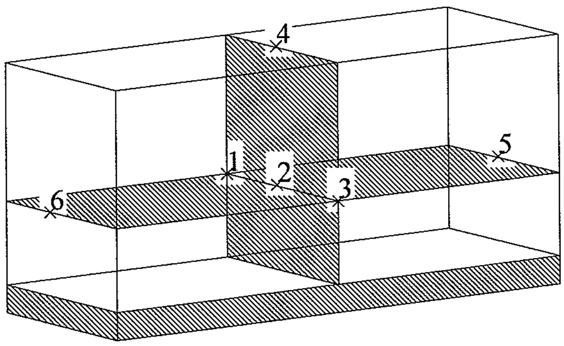

Figure 6.2: Typical location of temperature measuring points in a structural component.

6.1.5.2 Thermo-sensors in structures with cooling pipes/heating wires
Structures that are cooled by cooling pipes or heated by heating wires often present complicated temperature profiles over the cross-section and therefore the number of thermo-sensors has to be increased to assess/document the hardening process.

Figure 6.3 shows an example of location of thermo-sensors in a cooled/heated cross-section. In cooled structures sensor 4 registers the temperature at the surface of a cooling pipe. As the temperature gradient close to the cooling pipe/heating wire is high the exact location of the sensor should be known.

If structures cast on soil are to be heated, a thermo-sensor should be placed for instance 0.5 m under the sub-base surface to register if the values assumed for the specific heat and the thermal conductivity of the soil are suitable.

On account of the electric field that exists around a heating wire, measurements from sensors placed on heating wires can be incorrect.

The other thermo-sensors 1, 2, 3 and 5 register the temperature at different points around cooling pipes/heating wires. These measurements can be used to calculate an average temperature on the basis of a previously defined calculation procedure, cf. Section 6.3.1.1-6.3.1.2.

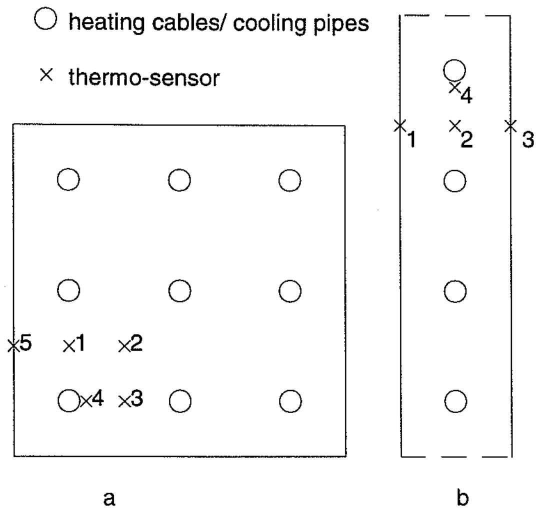

Figure 6.3: Location of thermo-sensors in structures with cooling pipes/heating wires

6.2 Registrations during the hardening process (logbook)

In connection with casting, a logbook should be made which includes the following:

- Reference to detailed description of the work
- Reference to calculation (if completed) on which the description is based
- Reference to planning document on location of thermo-sensors
- Registration of temperature development in soil at relevant depth
- When is heating/cooling started?
- Registration of inlet- and outlet-temperature of cooling water and flow
- When is casting started?
- The temperatures of concrete on delivery and casting ?
- The temperature of any adjacent structures ?
- Weather conditions before, during and after casting (temperature, wind, rainfall, cloudiness)
- When is the casting terminated ?
- When is the cooling/heating stopped ?
- When is concrete covered and possibly insulated and by what?
- When is stripping carried out (possible re-insulation)?
- Maturity and temperature at stripping

- Removal of insulation
- Check for cracks at stripping (temperature, where and how big)
- Curing at stripping (curing membrane, how much, which and for how long)
- Check for cracks after cooling (temperature, where and how big)
- Has post-calculation been carried out ?

6. 3 Assessment

6.3.1 Measured temperatures

By means of the criteria given in Sections 6.3.1.1-6.3.1.3 it can be decided whether the temperature requirements have been met. The criteria include the formulas for calculation of average temperatures based on temperatures registered by thermo-sensors. The applicability of these formulas should be demonstrated for the relevant concrete geometry by means of a temperature calculation.

If it cannot be documented that the temperature requirements were fulfilled the cause of the deviation of the planned temperature development in the sensors should be investigated. In Section 6.3.3 a number of factors are mentioned that might cause deviations and an updated temperature calculation can be made that will better correspond to the observations made.

This does not mean that the requirements for temperature differences are met, but a stress calculation based on the updated temperature calculation may prove to fulfil a possible requirement for maximum utilization of tensile strength.

If the casting is one of many, deviations shall always be investigated whether they are favourable or unfavourable. If a casting has been carried out according to plan and deviations still occur, the basis of the planning is inadequate. An uncertain basis may result in both damage and excessive hardening precautions for subsequent castings.

6.3.1.1 Temperature difference in hardening structures without cooling pipes/heating wires

The average temperature T_m in a cross-section shown in Figure 6.2 can as a guideline be defined as:

$$T_m = 2/3 \cdot T_2 + 1/6 \cdot (T_1 + T_3) \qquad (6.1)$$

This formula is based on the assumption that the temperature profile is approximately parabolic, which implies that T_1 and T_3 do not differ significantly.

A requirement to the effect that the difference between average temperature and that at the surface of the cross-section must not exceed a stipulated limit, $D_{int,req}$, is met when the following applies during the entire hardening period:

$$T_m - T_1 < D_{int,req} \quad ; \qquad T_m - T_3 < D_{int,req} \qquad (6.2)$$

6.3.1.2 Temperature difference in hardening structures with cooling pipes/heating wires

The average temperature T_m in a cross-section shown in Figure 6.3a can as a guideline be defined as:

$$T_m = 2/3 \cdot (2/3 \cdot (T_1 + T_2 + T_3)/3 + 1/3 \cdot T_4) + 1/3 \cdot T_5 \qquad (6.3)$$

A requirement to the effect that the difference between average temperature and that at the surface of the cross-section must not exceed a stipulated limit, $D_{int,req}$, is met when the following applies during the entire hardening period:

$$T_m - T_5 < D_{int,req} \qquad (6.4)$$

The average temperature T_m in a cross-section shown in Figure 6.3b can as a guideline be defined as:

$$T_m = 1/6 \cdot T_1 + 1/6 \cdot T_3 + 2/3 \cdot (4/5 \cdot T_2 + 1/5 \cdot T_4) \qquad (6.5)$$

A requirement to the effect that the difference between average temperature and that at the surface of the cross-section must not exceed a stipulated limit, $D_{int,req}$, is met when the following applies during the entire hardening period:

$$T_m - T_1 < D_{int,req} \quad ; \qquad T_m - T_2 < D_{int,req} \qquad (6.6)$$

6.3.1.3 Temperature difference between adjacent structural components

The variation in the average temperature of the structural component can be used as a measure of the total amount of temperature movements that take place in the component. When structures are cast together, the temperature movements in the individual structural components during the cooling phase shall be assessed and compared. Figure 6.4 shows the development of average temperatures of a previously cast component and a freshly cast component. The rise in average temperature in the previously cast component may originate from the heat flow through the construction joint from the hardening component, directly from heating by wires or perhaps from the residual hydration heat. According to the figure the difference between the temperature movements of the two structures at a given time t after t_{max} can be determined as:

$$\alpha \cdot (\Delta_{NEW} - \Delta_{OLD}) \qquad (6.7)$$

where

Δ_{NEW} is the difference between maximum temperature (observed at time t_{max}) and the actual temperature in the structural component for which the risk of cracking is assessed (cf Figure 6.4)

t_{max} is the time of maximum average temperature in the structural component for which the risk of cracking is assessed

Δ_{OLD} is the difference between the temperature at time t_{max} and the actual temperature in the previously cast concrete (cf. Figure 6.4)

α is the coefficient of thermal expansion of the concrete

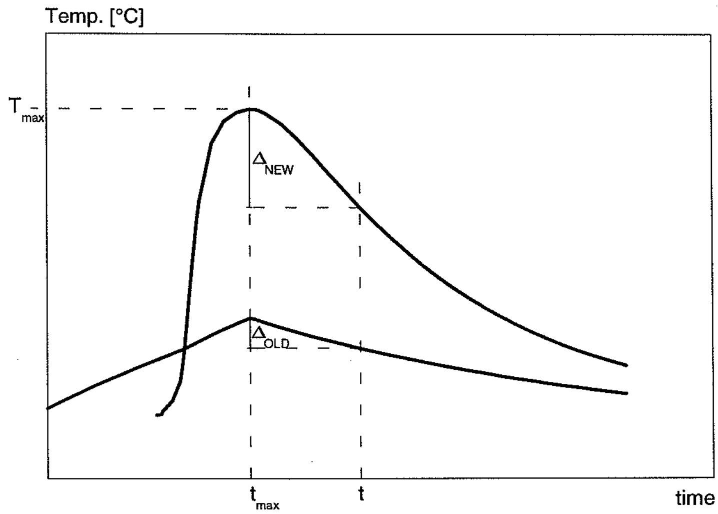

Figure 6.4: The development of the average temperature in adjacent structures

In practice the requirement for the maximum permissible difference in temperature movements is expressed by a temperature difference. Consequently the coefficient of thermal expansion is eliminated.

A requirement for D_{ext} is satisfied if

$$D_{ext} = \Delta_{NEW} - \Delta_{OLD} < D_{ext,req} \tag{6.8}$$

The requirement shall be met at any time after the hardening structural component has reached the maximum temperature, i.e. at all times later than t_{max}.

Similarly it can be shown that the requirement for maximum permissible difference in temperature movement when structural components are cast simultaneously is met.

Example 6.1

A foundation slab is heated 10 °C in 72 hours by electric heating cables. After 48 hours of heating a wall is cast on the foundation slab, which reaches its maximum average temperature after 20 hours, cf. Figure 6.5.

Figure 6.5 shows D_{ext} as a function of time. After 144 hours the maximum value of D_{ext} is determined as

$D_{ext} = \Delta_{NEW} - \Delta_{OLD} = (45.5 - 25.6) - (32.0 - 25.6) = 13.5\,°C$

which meets a requirement of 15 °C.

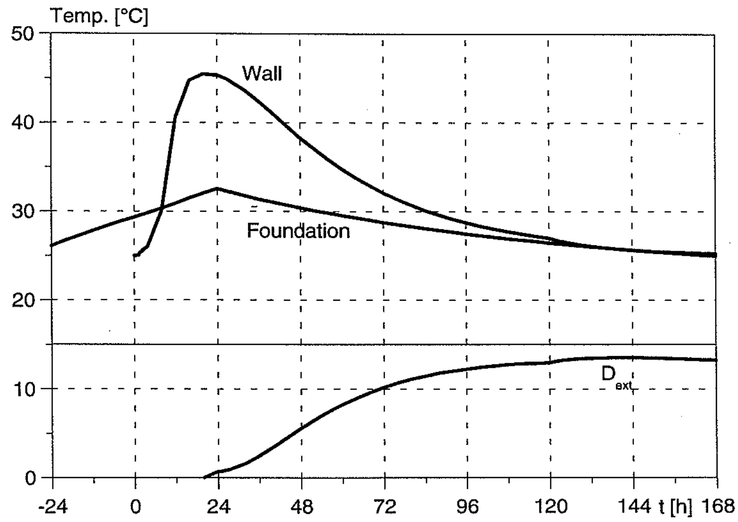

Figure 6.5: Average temperatures and D_{ext} as a function of time. Time t = 0 corresponds to the casting of the wall.

If the heating is stopped too late the result will be the development of the average temperatures and D_{ext} shown in Figure 6.6. In this case the maximum value of D_{ext} is determined after 84 hours as

$D_{ext} = \Delta_{NEW} - \Delta_{OLD} = (45.5 - 30.2) - (32.0 - 38.0) = 21.3\,°C$

which does not meet a requirement of 15 °C.

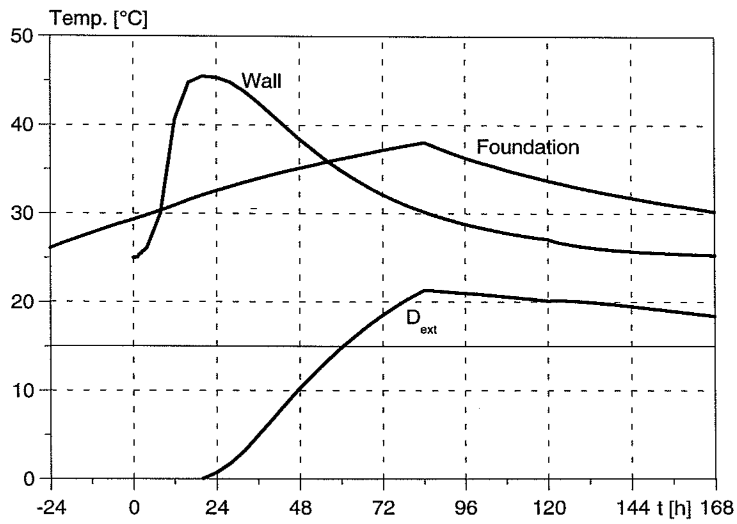

Figure 6.6: Average temperatures and D_{ext} as a function of time. Time t = 0 corresponds to the casting of the wall.

6.3.2 Crack registration

In practice the only usable method to check for cracks in structures cast in-situ is by visual inspection of the concrete surface.

Surface cracks that are generated on account of internal fixation are formed during the heating phase and close again during the cooling phase, while through-going cracks that are generated on account of external fixation are formed during the cooling phase and do not normally close again. It is therefore difficult to choose one time as suited for the detection of both surface cracks and through-going cracks. Experience does however show that it is easier to perceive cracks immediately after stripping than later.

It is recommended to carry out two crack inspections. One immediately after stripping and one when the structure is in thermal equilibrium with the surroundings, but not before the requirement for duration of drying protection has been met.

Measurement/registration of jcracks should include the drawing of sketches that show the position and development of the cracks and measurements of crack widths and lengths.

6.3.3 Deviations

After casting, a comparison of calculations and measurements indicates whether there are deviations between the planned and the actual hardening process. The deviations can be:

- Differences between calculated and measured temperature developments
- Cracks are observed even if calculations show no cracks
- Crack widths greater than planned are observed

The accuracy of thermometers in concrete is typically ±2 °C. For thermo-sensors placed in areas with high temperature variations (e.g. close to a surface or near cooling pipes/heating wires), uncertainty regarding the location can result in great inaccuracy.

Deviations in the form of crack formation may be due to deviations in the temperature development, but also incorrect assumptions for the support conditions and material properties may be the cause.

Increased crack width may be due to deviations in the temperature development or that the crack width formulas used underestimate the effect of the distribution reinforcement.

When the casting has been carried out, it is possible to investigate if the following assumptions may have caused the deviations:

Execution
It is investigated if the work has been carried out as planned. Such investigation is based on the logbook and the calculations carried out.

The casting temperature of the concrete
If there is no requirement for a specific concrete temperature on delivery, the temperature may vary. This is easy to check by measuring the temperature of each concrete delivery and comparing it with the assumed casting temperature.

Mix design
It is checked if the mix design of the delivered concrete corresponds to the mix assumed in planning.

Weather conditions
Weather conditions on the casting day will seldom be as assumed. During the planning a typical weather situation with typical deviations during the relevant casting period can be considered. If the weather conditions deviate significantly from the assumed this may be the cause of deviations between measured and calculated values of the temperature.

Soil temperature, specific heat and thermal conductivity
If the sub-base (soil) is included in the temperature calculations the change in temperature registered by the buried thermo-sensor can be compared to the calculated change. The degree of water saturation and the composition of the soil do however greatly influence the thermal properties and therefore the actual properties can deviate significantly from the assumed. In a calculation the thermal properties of the soil can be changed and it can then be shown if this accounts for the observed deviation.

Temperature of cooling water / effect of heating wires
It is investigated if the registered temperatures of the cooling water (inlet and outlet) and the flow correspond to the planned. The actual resistance of the heating wires can be measured by an ohmmeter and the effect is calculated and compared to the value used in the planning.

Support conditions
The static support conditions greatly influence the calculated stresses. In each case it shall be assessed if reasonable assumptions for support conditions have been made. If cracks are generated in areas where calculations showed no crack formation, it might be possible to assess - based on the crack pattern - if the support conditions have been appropriate. If the calculation is carried out 2-dimensionally, it should be assessed whether the problem is actually 3-dimensional.

General
If deviations are observed in one or more of the above areas it will be necessary to carry out a reassessment by means of a new calculation based on the revised assumptions. If an updated calculation does not result in conformity between measurements and calculations an explanation may be found in:

Inaccuracy in calculations
For the Finite Element Method (FEM), the accuracy of the calculated solution depends on the fineness of the element mesh. The means for both temperature and stress calculations that a "correct" solution is obtained only when a closer mesh does not give another result. It should be investigated if the element mesh used in the calculation is fine enough.

Inaccuracy in material models
It should be checked whether the material models used are adequate, cf. Example 4.1.

Coefficients of surface transmission
The coefficients of transmission of concrete surfaces are normally assessed on the basis of empirical values where the wind velocity around the form or free surfaces are taken into account. In the casting situation some concrete surfaces are sheltered and others exposed to considerably higher wind velocities caused by turbulence or a stronger wind than assumed. As for the soil properties, it shall in each case be assessed if the properties of the concrete surfaces can account for the observed deviation.

Insolation
Higher temperatures in a casting than expected can in some cases be the result of insolation. It shall be assessed in each case if the insolation can account for the deviation observed. The orientation of the structure in relation to the sun, and the angle of the surfaces in relation to the position of the sun both vertically and horizontally influence the contribution of the sun to the heating of the concrete. The insolation also depends on the time of year and the structure and colour of the surface. The insolation also depends on the cloudiness during the relevant period.

6.4 Measuring equipment

In principle, measurements of the temperature in the hardening concrete can be carried out at 3 levels:

- Manual measurement with frequent registrations
- Continuous measurement by means of datalogger

- On-line continuous measurement by means of datalogger and PC

These guidelines will primarily describe automatic measurement with datalogger by cast-in thermo-sensors.

A more detailed description of the different types of temperature sensors is given in the literature, for instance [Betonteknik, 1975].

Normally the measurement of the concrete temperature is carried out by means of a so-called thermo-sensor where the principle is that two different metals electrically connected will create a potential difference that depends on the temperature of the surroundings. The potential difference can be measured by a millivoltmeter or a datalogger to which wires from the structure are connected.

If the requirement is only a subsequent documentation of the temperature development, entry of data from the datalogger can be made at the termination of the hardening process, but this procedure does not allow the temperature development to be changed during the hardening process.

If it is necessary to carry out corrective measures during the casting by using the temperature measurements, on-line registration is required. The datalogger is therefore connected to a PC which by a special programme continuously receives data from the datalogger and simultaneously presents the temperature development graphically. The programme can be extended to carry out maturity calculations, relay control of e.g. cooling water, etc.

Most computer programmes used for registration of changes in temperature during a hardening process store data continuously on the harddisk of the PC. The operator in charge of the registration of temperature data should before each start of the programme make sure that the capacity on the harddisk is large enough to store the expected amount of data plus a certain stand-by capacity.

It should be noted that the thermo-sensors can be sensitive to magnetic fields originating from electric current or radio signals. An appropriate wiring system and test of the system in advance of casting should, however, trace and minimize such disturbance. The planning of the wiring system should take other site work into account in order to avoid damage to the cables and thereby interruptions of the measurements.

Computers that are used as dataloggers are normally connected to the mains supply set up at the site. Such temporary electrical installations are, however, subject to power failures of short duration. A power failure will for most PCs mean that the temperature registration is interrupted, and at worst that the data stored prior to the power failure are lost. By installing a back-up battery that is continuously charged by the mains supply, the PC will operate for a while after a power failure.

7.0 Requirements for cracks formed during hardening

This Chapter is concerned with the requirements that should be specified in order to limit or prevent formation of cracks during the hardening period. For same period requirements should be stipulated to protect the concrete against early drying and frost. These requirements are described in the guidelines [HETEK - Curing, 1997] and [Vinterstøbning af beton, 1985].

It shall be stressed that the purpose of this Chapter is **not** to provide a complete proposal for a specification. It is to describe some of the possible requirements for crack formation, so that readers can obtain a better knowledge of the relevant problems. The final choice of requirements should always be based on technical, time schedule and economic considerations for the project.

7.1 In which structures do hydration cracks form ?

Cracks can appear during the hardening process as:

- surface cracks caused by too great differences in the temperature movements between the core of the concrete and the surfaces. These cracks are generated when the hardening temperature is a maximum, but they close again during the cooling phase.
- through-going cracks caused by too great differences in the temperature movements in adjacent structural components. The cracks are typically formed during the cooling phase and do not close again.

Surface cracking may occur in structures with large dimensions, e.g. public works. In structures with smaller dimensions, e.g. beams and columns for houses, the hydration heat is transferred quickly and surface cracks are rarely observed.

Through-going cracks appear most frequently in large structures, but can also appear in smaller ones if temperature movement is hindered by adjacent structures.

7.2 Which requirements should be specificed ?

If there is a risk that a structure may generate cracks during the hardening phase, it shall be decided if cracks are permissible. This is done on the following basis:

- environmental influence combined with planned service life
- function (typically tigthness)
- possibility of maintenance (repair)
- appearance

Example 7.1

Cracks should not occur in:
- *horizontal surfaces that are water-saturated, e.g. by rain*
- *structural components that can absorb water from the sub-base*
- *structural components located in the splash-zone (water line)*
- *walls and decks under water pressure*

Cracks are normally permissible in structures in passive and moderate environmental classes, if there are no special requirements for service life or appearance.

If cracks are not permissible, the requirements should be based on maximum utilization of tensile strength. The requirement is described in detail in Section 7.3.

If cracks are permissible, a requirement is specified for the maximum crack width. The crack width depends on the reinforcement system and the load in the service limit state. Furthermore, the crack width depends on the temperature and shrinkage development which the structure undergoes during the hardening phase. For a given minimum reinforcement the requirement can be a maximum permissible temperature movement expressed by D_{ext}.

In practice the crack width calculation is often omitted because it is subject to considerable inaccuracy. Instead the requirement for D_{ext} is specified on the basis of an empirical assessment of the stiffness ratio between the adjacent structural components. The requirements for temperature difference and crack width are further described in Sections 7.4-7.5.

7.3 Requirements for maximum utilization of tensile strength

The requirement for maximum utilization of tensile strength can be stated directly by a given numerical quantity or indirectly by specifying the maximum temperature differences that are permissible.

7.3.1 Direct requirement for maximum utilization of tensile strength

By means of stress calculations the risk of crack formation is assessed on the basis of the ratio between a calculated tensile stress in the concrete and the tensile strength of the concrete (utilization of tensile strength). Formally it is assumed that cracks will not be generated as long as the calculated tensile stresses do not exceed the tensile strength of the concrete. As calculations of the tensile stress and the tensile strength are subject to some uncertainty, a decision must be made on the safety factor required for prevention or minimization of the number of cracks.

On the basis of observations of crack formation in concrete structures, the report [Pedersen, E.J., 1997] concludes that normally very few cracks are formed if the tensile stresses during the cooling phase are less than 80% of the splitting tensile strength (according to [DS423.34]). If no cracks at all are permissible, it can be decided to specify a maximum utilization of tensile strength of 65 - 70%.

Specifying more than one limit for the utilization of tensile strength can be considered. Requirements can e.g. be stipulated for stresses caused by casting on old concrete (external fixing) and for stresses caused by temperature differences between the surface and the core of the cross-section (internal fixing).

When assessing the risk of crack formation in casting on old structures it must be remembered that external fixing results in permanent tensile stresses that to a certain extent are reduced in the course of time because of creep. When the structure is in service, the permanent stresses are overlaid by contributions from live loads, and also by seasonal temperature variations and drying shrinkage. Thus the consequence of an insufficient safety factor against crack formation during external fixing may be that the structure is accepted and delivered free from cracks, but that cracks are suddenly formed after a cold winter or a warm summer. Therefore a certain amount of crack distributing reinforcement should always be placed even if stress calculations have been carried out.

Example 7.2

The water tigthness of a structure is estimated not to be threatened by temporary surface cracks, but by through-going permanent cracks (cf Figure 2.2). In this case requirements are specified only for determination of the stress component that directly causes the risk of generation of cracks (cf. Figure 2.7).

7.3.2 Requirement for stress calculation

As stress calculations for hardening concrete structures are based on a previous temperature calculation, all remarks about temperature calculations also apply to stress calculations, cf. Section 7.4.3.

It should be required that a calculation model has been documented to be suited for the particular project. The documentation may be in the form of references to previous projects, technical specifications of algorithms, manuals, etc.

Furthermore, the mechanical properties to be included in the calculation (cf. Chapter 4) should be specified. And finally it should be specified how the mechanical properties are determined or possibly the values they can be given.

In connection with the preparation of specifications based on utilization of tensile strength, the support conditions to be assumed should be considered. The specification can e.g. require that different support conditions be assessed by means of calculations. In the case of very high castings there should be a limit to the number of previous castings to be included or determined by means of a calculation.

7.3.3 Requirement for documentation

Temperature and stress calculations shall be available that correspond to the chosen execution method, cf Sections 6.1.1-6.1.2. The fulfilment of the requirement for utilization of tensile strength shall be documented, but because it is not feasible to determine the stresses in a hardening concrete structure the fulfilment of the requirement will in practice be documented by measuring the temperature development (cf. Section 6.1.2) The documentation therefore corresponds to the documentation of the temperature requirement described in Section 7.4.4.

7.3.4 Requirements in connection with deviations

It should appear from the specification that the calculations carried out and the given assumptions shall be analysed. This analysis may result in a change in the execution method, in order to prevent recurrences. If the requirements have been met, but cracks are nevertheless formed, the stipulated requirements shall be reconsidered.

7.3.5 Requirements for maximum utilization of tensile strength in the form of temperature requirements

Guidelines for temperature requirements on the basis of stress calculations are given in Sections 6.1.2 and 6.3.1. Regarding the formulation of the temperature requirements, reference is made to Section 7.4.

7.4 Formulation of temperature requirements

Requirements for maximum temperature differences can be derived from the requirement for maximum utilization of tensile strength (Section 7.3), maximum crack width (Section 7.5) or based directly on empirical values. Temperature requirements are expressed as maximum permissible values of D_{ext} and D_{int}, cf. Sections 2.1-2.2.

7.4.1 Requirements for external temperature difference, D_{ext}

The requirement applies to the maximum difference that is allowed in the average temperatures of the adjacent structural components. The requirement should apply to structural components that are cast at different times. If there are significant differences in material thickness in one casting, the latter should be divided into segments, and requirements should be specified for the maximum temperature movements between these segments, too.

It is recommended to define the structural components to which the requirement for D_{ext} shall apply, i.e. the structural components that are assumed to be fixed by adjacent structural components, cast simultaneously or earlier.

In the case of extensive construction works where several types of concrete and/or several different types of structural components are cast it should be considered to define different maximum permissible temperature differences for the individual structural components. In practice this requirement often necessitates the use of complicated and costly measures for control of temperature, e.g. cooling systems. For structural components with a high degree of fixation, e.g. long components or components that are cast between other components (cf. Section 3.1) the requirement for D_{ext} should be made more rigorous.

Example 7.3

DS411 recommends the maximum permissible D_{ext} to be 15 °C when casting is carried out on to a previously cast structure. The requirement is often made more rigorous - or eased - within the interval 12 to 20 °C depending on the stiffness and support conditions as well as the amount of distribution reinforcement.

Example 7.4

The temperature level will differ significantly:

- *in the massive slab and edge-beams when a bridge cross-section is cast*
- *in walls and deck with substantially different material thicknesses*

7.4.2 Requirements for internal temperature difference, D_{int}

The requirement applies to the maximum difference that is allowed between the average temperature and the surface temperature of a structural component. In practice the temperature deformations in the core and at the surface are different for all components because of the heat exchange to the surroundings. The requirement for D_{int} therefore applies to all structural components. The amount of distribution reinforcement does not normally influence the requirement for D_{int}.

Example 7.5

[DS411] recommends that D_{int} is 15 °C. Practical experience from the site indicates that a maximum D_{int} of 15 °C normally prevents the formation of surface cracks.

7.4.3 Requirements for temperature calculations

It should be required that a calculation model has been documented to be suited for the particular project. The documentation may be in the form of references to previous projects, technical specifications of algorithms, manuals, etc.

The specification should define the thermal properties that should be included in the calculation (cf. Chapter 4). Furthermore it should be specified how the thermal properties are determined or possibly the values they can be given.

Temperature calculations shall be carried out in order to ensure that the requirements for weather conditions (temperature, wind, etc) that can be expected during the casting are met. Normally it is the contractor who defines the appropriate weather conditions and who then checks whether these existed before and during the casting. It can, however, be required that for every casting a specified number of calculations are carried out with different weather conditions (e.g. 2 or 3 situations).

7.4.4 Requirement for documentation

Primarily it must be required that the documentation of temperature calculations include all data so that an independent calculation can be carried out, cf. Section 6.1.1. Then it should be considered how long before casting the calculation should be made in order to provide time for the detailed planning of the execution of the concrete work.

For specifications based on temperature requirements, the following should be documented:

1. Planning of execution method cf. Section 6.1.1

2. Temperature measurement
   The specification should include requirements for:
   - the structural components where the temperature shall be measured. The temperature should be measured in all components with different geometry, boundary conditions or method of temperature control
   - duration of measurements. The measurement should be carried out from the initiation of the casting until it has been documented that all requirements concerning the temperature or maturity have been met, and at least until the temperature of the component is in equilibrium with the temperature of the surroundings.
   - the location of the thermo-sensors and how the average temperatures are calculated. The principle of weighting of thermo-sensors can be given in the specification or the contractor may choose a suitable system for weighting the temperature measurements. If the latter method is chosen it should be required that the contractor by means of temperature calculation verifies the chosen principle for calculation of the average temperature.
   - continuous measurement and monitoring of the temperature during hardening
   - accuracy of the measuring equipment

3. Logbook for the hardening process
   A hardening logbook should be prepared with a content as described in Section 6.2.

4. Crack inspection
   Requirements should stipulate that the structure be inspected for cracks, cf Section 6.3.2.

7.4.5 Requirements in connection with deviations

The specification should require an analysis of the calculations and the assumptions. This analysis may result in the execution method being changed, in order to prevent recurrences. If the temperature requirements have been met, but cracks are nevertheless formed, the stipulated requirements shall be reconsidered.

7.5 Requirements for crack width

Requirements for maximum crack width can be expressed directly as a given numerical quantity or indirectly by stating the maximum temperature differences, expressed as D_{ext}, that are permissible for a given amount of distribution reinforcement.

7.5.1 Direct requirement for crack width

According to [DS411] there is no clear correlation between crack width and risk of corrosion. For structures in which the crack width can affect the durability, DS 411 recommends crack widths under 0.2-0.3 mm in aggressive environment classes.

7.5.2 Requirements for calculation of crack width

A relevant calculation procedure should be recommended. Many crack width formulas are available that enable a determination of crack widths for permanent load and of shrinkage for a given reinforcement percentage. Hardening cracks correspond to shrinkage cracks and therefore the calculation formulas for shrinkage can to a certain extent be applied to hardening cracks (cf. Section 3.3).

The chosen calculation procedure should include reinforcement percentage, reinforcement diameter, geometry as well as shrinkage and temperature deformations. The deformations shall include contributions from the hardening process as well as from the temperature variation through the year.

It should be stressed that calculation of crack width is subject to great uncertainty.

7.5.3 Requirement for documentation

The necessary documentation in connection with fulfilment of the requirements for crack width should comprise:

- dimensioning of reinforcement in relation to the planned hardening process
- inspection of the structure for cracks, cf. Section 6.3.2
- documentation as described regarding temperature requirements, cf. Section 7.4.4

7.5.4 Requirements in connection with deviations

It should appear from the specification that the calculations carried out and the given assumptions shall be analysed. This analysis may result in the execution method being changed, in order to prevent recurrences.

7.5.5 Requirements for crack width stipulated as temperature requirements

If temperature requirements have been stipulated they shall include a sufficient amount of distribution reinforcement. As concerns the stipulation of temperature requirements reference is made to Section 7.4.

7.6 Economy

When preparing specifications efforts should be made to clearly define the services that the tender shall include. It is recommended to fix a number of unit prices for the different measures for control and measurement of temperature (e.g. cooling pipes, heating wires, winter mats, thermo-sensors, etc.) so that the contractor and owner have a definite basis for settling the price of possible additional services. The price of such measures should be fixed so that payment is effected according to actual consumption (including operation and installation). The pricing of alternative measures in connection with the submission of tenders increases the possibility of reaching technical and financial solutions that are optimal.

It may also be considered to include a price for calculations of temperature and stress in the tender so that calculations in excess of the number given in the tender are paid for separately.

It is important that the tender material is unambiguous so that the owner is not tempted to have the contractor pay for services that are not specified in the contract, and the contractor is not tempted to get extra payment for taking unnecessary measures.

If the specifications are based on well-defined temperature requirements the preparation of the tender is normally straightforward; it should include all measures for control and measurement of temperature.

The introduction of stress calculation means that the costs of measures for control of temperature can be adjusted to the casting in question. In certain cases the costs are reduced in relation to ordinary temperature requirements, but in other cases they are increased.

Irrespective of a reduction of the execution costs by an introduction of requirements on utilization of tensile strength, the planning costs will be increased in relation to the requirement on temperature for the following reasons:

- The specification is often more detailed/complicated and it might be necessary to carry out analyses during the design phase
- The calculation work is more comprehensive, because calculations of both temperature and stress shall be carried out
- The preparation and evaluation of reports on the calculations and associated working descriptions are often more time-consuming
- Extended initial period of the building project on account of determination of additional concrete properties.

For projects of a certain size, the total cost of establishment, maintenance and possible repairs is however assumed to be smaller when stress calculations have been carried out because individual requirements have been specified for the various types of structure and general temperature requirements have not been specified.

8.0 Example of a calculation of the hardening process

A tunnel structure as shown in Figure 8.1 is considered. The height of the wall is approximately 7 m and the width of the deck is approximately 11 m. The thickness of wall and deck is 0.5 and 0.6 m, respectively. The tunnel is supported by strip-foundations. The tunnel is cast in 20 m long sections separated by expansion joints in both foundations and superstructure. The wall and deck are cast simultaneously, i.e. without construction joints. The foundations were cast long before the superstructure. The structure is to be cast in the summer.

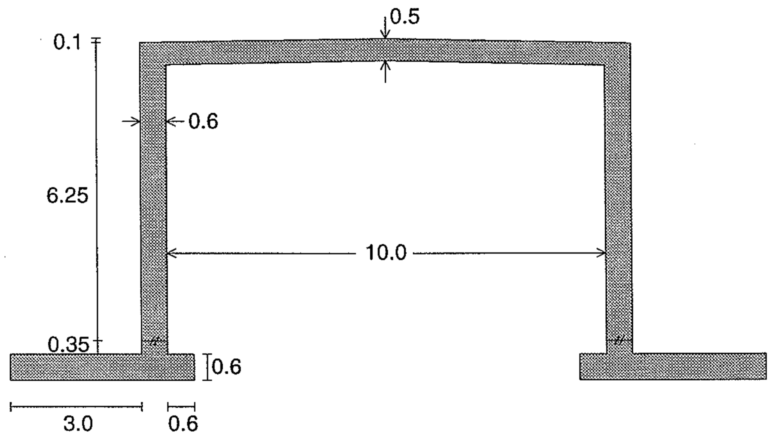

Figure 8.1: Cross-section of tunnel

The tunnel is planned to be cast in an 18 mm plywood form. The form type is the same for the inside and the outside of the walls and the underside of the deck. The form shall be removed 120 hours after casting on account of the preparations for the casting of subsequent sections. The upper side of the deck shall be protected against drying by application of a curing membrane according to the principles given in [HETEK-Curing, 1997].

It is a requirement that the ratio between the tensile stress and the splitting tensile strength does not exceed 80% at any time. The structure shall remain protected against drying for at least 120 maturity hours.

8.1 Calculation assumptions

The calculations were carried out by means of the programme CIMS 2D [DTI Byggeri, 1995], which is based on the principles described in Chapter 2.

Geometry
As the structure is symmetrical as concerns both geometry and stresses, the calculations are carried out for only one half of the cross-section shown in Figure 8.1. This means a reduction of the calculation time. In the symmetry line there is no exchange of energy with the surroundings.

Concrete properties
The chosen concrete has a w/c-ratio of 0.40. The cement is a low alkaline sulfate resistant Portland cement with fly ash and silica fume. The properties of the concrete are determined by testing (cf. Chapter 4). The properties are described in [Spange and Pedersen, 1996].

Casting schedule
As the foundations are old when the superstructure is cast, it is therefore assumed that the E-modulus is fully developed. Furthermore, creep and autogenous shrinkage of the foundations are neglected.

Wall and deck are cast in one and the same process. In practice a period of time will of course elapse from the initiation of the casting of the walls to the completion of the deck. In the calculations the walls are assumed to be cast at time t = 0 and the deck to be cast at time t = 6 hours. In the calculation it is assumed that the upper sides of the walls are without insulation until the deck is cast.

Form
The thermal conductivity of the form is taken as 0.5 kJ/m/h/°C. The form is removed at t = 120 hours.

Weather conditions
The situation is a warm summer with

air temperature	= 25°C
wind velocity	= 5 m/s
concrete temperature	= 27°C

The initial temperature of the foundation is estimated to be equal to the air temperature. As the temperature at the underside of the foundation is assumed not to be affected significantly by the casting of the wall it is assumed that there is no exchange of energy between foundation and soil.

Support conditions
It is assumed that the structure is on the boundary between long and short, cf. Section 5.3.8, as

$$L/H = 20/7 \approx 3$$

Calculations are consequently carried out on two situations, one where the cross-section is allowed to bend freely about the horizontal axis in the plane of the cross-section and one where the cross-section is prevented from bending. This corresponds to the situation in a section placed at the end of and in the middle of the structure, respectively (cf. Figure 5.14). As the calculations are carried out on one half of the structure, bending of the cross-section about the vertical axis must be prevented for reasons of symmetry.

The underside of the foundation is fixed in order to prevent movement in the vertical and horizontal directions. In the symmetry line of the deck, horizontal movement is prevented.

Dead load
When the deck is stripped the dead load is no longer carried by the form, and is taken up by the structure. This is important for the stresses in both deck and walls (cf. Figure 5.6). In the calculations the dead load is included in the form of a downward line load of 0.012 MN/m applied along the upper side of the deck. The line load corresponds to the dead load of the deck.

8.2 Preliminary calculation without hardening measures

8.2.1 Temperature calculation

Figure 8.2 shows maximum temperatures as a function of time for wall and deck, respectively. The maximum temperatures in wall and deck are 53.5 °C and 51.5 °C respectively and they are observed about 24 hours after the casting of the wall. At the stripping time the maximum temperature has dropped to about 29.5 °C in the wall and about 28 °C in the deck. The superstructure has cooled to the temperature of the surroundings after approximately 168 hours.

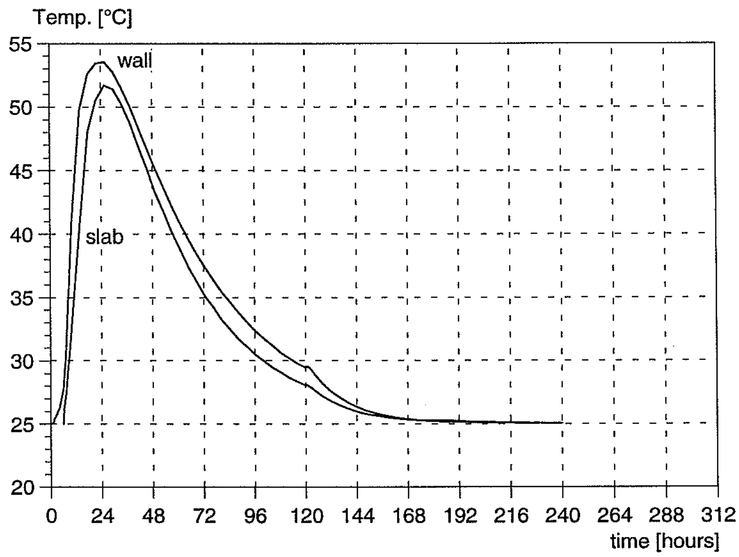

Figure 8.2: Maximum temperatures

Figure 8.3 shows the temperature distribution over the cross-section 24 hours after casting of the wall. The temperature distribution over the wall is symmetrical. The surface temperature of the wall is about 45 °C. The temperature distribution over the deck is oblique with a temperature of about 42 °C at the underside and about 33 °C at the upper side. The maximum temperature of the deck occurs in the area over the wall.

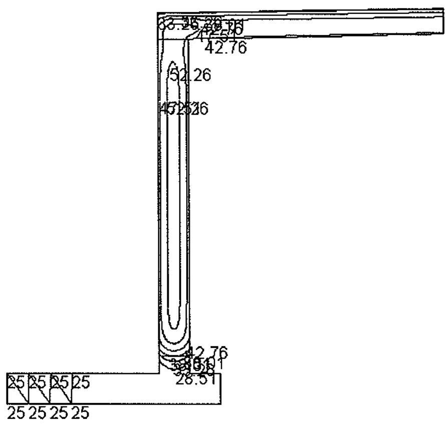

Figure 8.3: Temperature distribution at t = 24 hours.

Figure 8.4 shows the average temperature as a function of time for foundation, wall and deck. The maximum average temperatures in wall and deck are about 50 °C and 43 °C respectively. The average temperature of the foundation rises less than 1 °C on account of the casting of the superstructure. It is seen that the average temperature of the foundation corresponds to the initial temperature when the superstructure has cooled to the temperature of the surroundings (cf. Section 5.3.5.1).

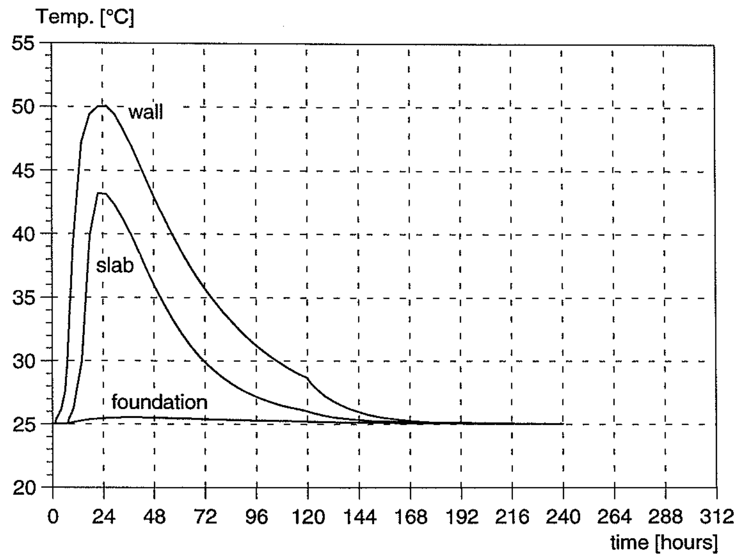

Figure 8.4: Average temperatures

Figure 8.5 shows the minimum maturity at the surface of both wall and deck as a function of time. It is seen that the maturity at stripping (t = 120 hours) is approximately 160 hours. The requirement for drying protection has been met.

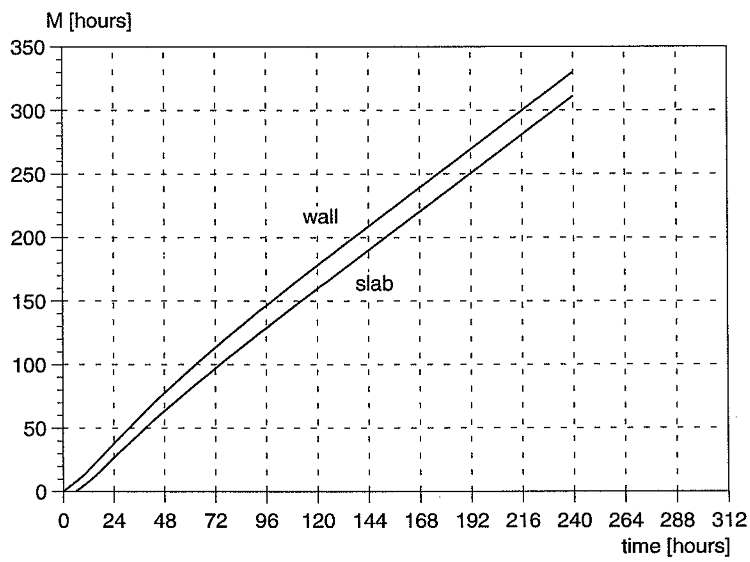

Figure 8.5: Minimum maturity in wall and deck.

8.2.2 Stress calculation - cross-section allowed to bend freely

Figure 8.6 shows the maximum tensile stress in relation to the splitting tensile strength for wall and deck as a function of time when the cross-section is allowed to bend freely. It is seen that the requirement for a maximum 80% utilization is exceeded both during the heating phase and the cooling phase.

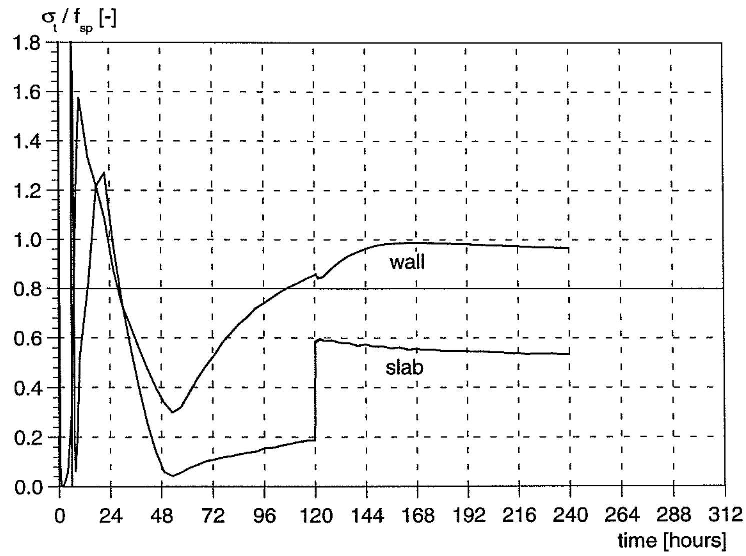

Figure 8.6: Maximum utilization of splitting tensile strength. Free flexure.

Apparently the utilization of the tensile strength is very high immediately after the casting of the wall (t = 0) and also the casting of the deck (t = 6 hours). Immediately after casting the E-modulus and thus the calculated stresses are close to zero. The tensile strength is also close to zero at this moment. Therefore the very high utilization immediately after casting does not constitute a problem, because the concrete is still a liquid.

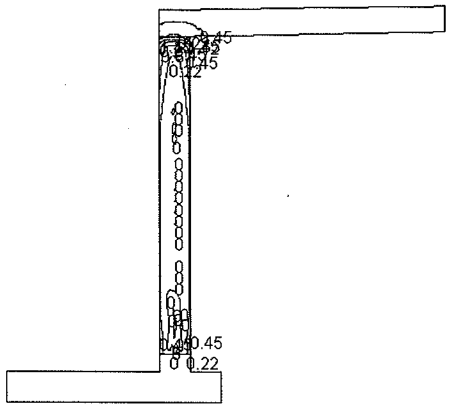

Figure 8.7: Utilization of splitting tensile strength at time t = 10 hours. Free flexure.

Figure 8.7 shows the distribution of tensile strength utilization over the cross-section at t = 10 hours, corresponding to the time of maximum utilization in the wall during the heating phase. The maximum utilization of the tensile strength is observed at the top corners of the wall. The problem originates from the stress component perpendicular to the cross-section (σ_z) and is caused by the cooling of the top of the wall on account of the free surface before the deck is cast. In the calculation - as mentioned in Section 8.1.1 - it is assumed that the deck is cast 6 hours after the wall. In practice the casting of the deck will take place in continuation of the casting of the wall, which means that the top of the wall will not be cooled. Consequently the high utilization does not constitute a problem.

Figure 8.8 shows the utilization of tensile strength at t = 22 hours corresponding to the time of maximum utilization in the deck during the heating phase. A high utilization of the tensile strength is observed in the upper side of the deck with a maximum in the area over the wall. The problem originates from the horizontal component in the plane of the cross-section (σ_x), but also the stress component perpendicular to the cross-section (σ_z) is high.

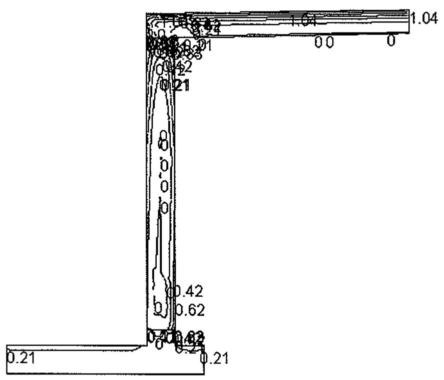

Figure 8.8: Utilization of splitting tensile strength at t = 22 hours. Free flexure.

The distribution of σ_x at t = 22 hours is shown in Figure 8.9. The tensile stresses in the upper side of the deck are caused by the cooling of the free surface in relation to the centre. The maximum tensile stress is observed in the surface over the wall. One reason is that the temperature in the deck is highest in this area (cf. Figure 8.3). Another reason is that the deck expands in the plane of the cross-section on account of the heating. As the structure acts like a frame, this expansion generates tensile stresses in the upper side of the frame corner (cf. Figure 5.5). The high σ_x involves a risk of longitudinal temporary cracks in the top side of the deck.

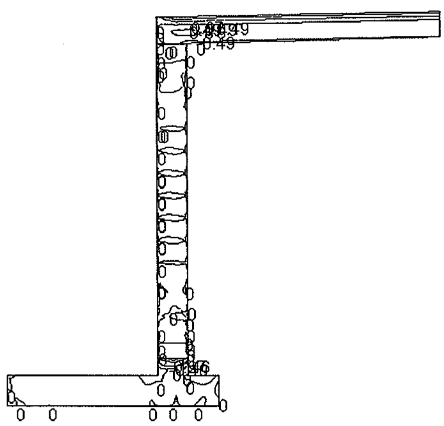

Figure 8.9: Horizontal stress component σ_x at t = 22 hours.

The distribution of σ_z at t = 22 hours is shown in Figure 8.10. The tensile stresses at the top of the deck are caused partly by the temperature profile across the deck and partly by the expansion of deck and wall in relation to the foundation (cf. Figure 5.3). The cross-section is allowed to bend freely and this generates the tensile stresses observed at the bottom of the wall. The high σ_z at the top of the deck involves a risk of surface cracks across the deck.

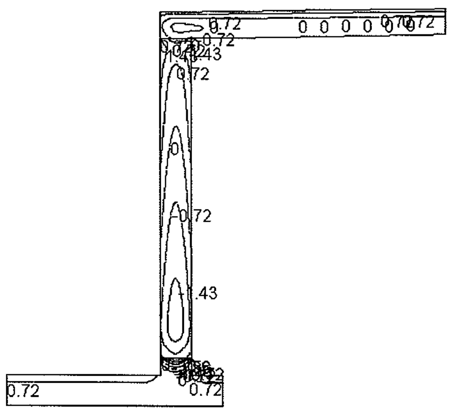

Figure 8.10: Stress component perpendicular to the cross-section σ_z at t = 22 hours. Free flexure.

At the top of the wall a high σ_z is observed in the surfaces at this time which originates from the cooling of the wall before the casting of the deck. As previously mentioned this cooling does not take place in practice. Therefore the high utilization at the top of the surfaces of the wall (cf. Figure 8.8) does not constitute a problem.

Figure 8.11 shows the distribution of the vertical stress component σ_y at t = 22 hours. The variation across the wall thickness is due to the temperature difference between centre and boundary. Furthermore, the transverse expansion of the deck causes flexure in the wall which can be seen from the oblique iso-quants. In the given situation there is no risk of crack formation, but if the deck had been broader the risk of cracks would have been higher.

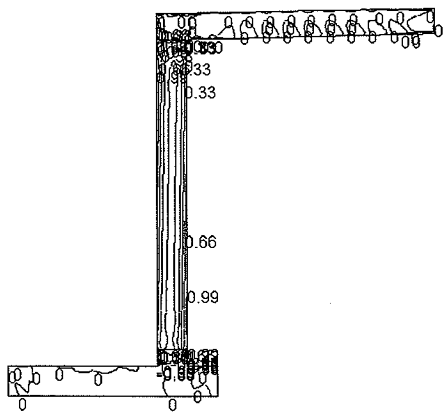

Figure 8.11: Vertical stress distribution σ_y at t = 22 hours.

Figure 8.12 shows the utilization of tensile strength at t = 121 hours corresponding to immediately after stripping. The highest utilization is observed at the bottom of the wall and arises from the stress component perpendicular to the cross-section (σ_z). The problem is due to the wall and deck cooling and subsequently contracting in relation to the foundation (cf. Figure 5.4). As the cross-section is allowed to bend freely, the highest tensile stresses are observed at the bottom of the wall, while there are compressive stresses in the deck as shown in Figure 8.13. The reason why the highest tensile stresses do not occur at the construction joint is that the bottom part is cooled by the foundation (cf. Figure 8.3) so that the changes in temperature are not so great in this area. The high σ_z means a risk of vertical through-going cracks in the wall.

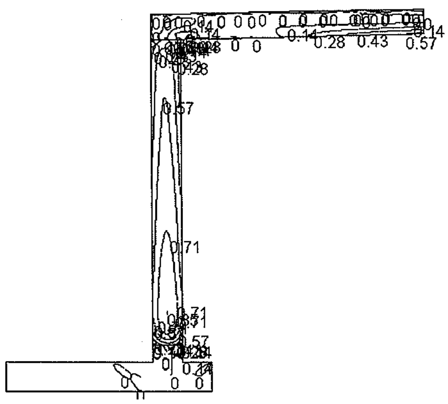

Figure 8.12: Utilization of the tensile strength development immediately after stripping (t = 12 hours). Free flexure.

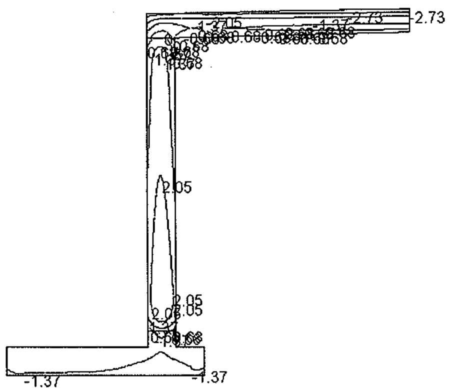

Figure 8.13: Stress component perpendicular to the cross-section, σ_z

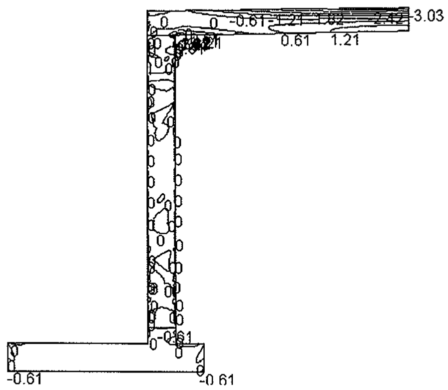

Figure 8.14: Horizontal stress component σ_x immediately after stripping (t = 121 hours).

Figure 8.12 shows that the utilization at the underside of the deck in the area close to the centre of the span is high compared to the rest of the deck. The utilization originates from the horizontal component in the plane of the cross-section σ_x. Figure 8.14 shows the distribution of σ_x at t = 121 hours. Compressive stresses are observed at the upper side of the deck at the centre of the span and tensile stresses in the underside. This is due to the dead load which is released when the form is removed. As the structure acts as a frame, the dead load also causes tensile stresses in the upper side of the frame corner. In this situation the dead load involves no risk of crack formation in the deck.

Figure 8.15 shows the distribution of the vertical stress component σ_y at t = 121 hours. The variation across the wall thickness is due to the highest temperature being developed at the centre of the wall. The release of the dead load results in tensile stresses at the top of the outside of the wall and at the bottom of the inside on account of the frame effect. Furthermore, the transverse contraction of the deck caused by the cooling results in tensile stresses at the bottom of the outside of the wall. In this situation there is no risk of crack formation, but a larger span (deck width) would increase the risk.

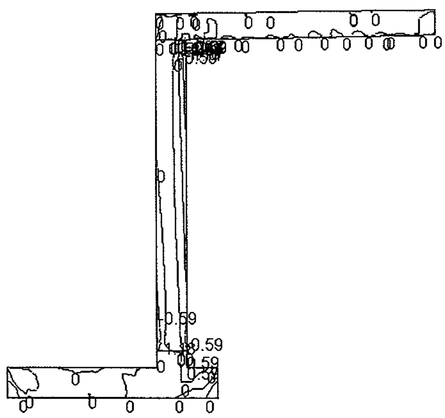

Figure 8.15: Vertical stress component σ_y immediately after stripping (t = 121 hours).

The cooling of the surfaces on account of stripping results in tensile stresses at the surfaces. In this case, however, the contributions are small, because the temperature at the stripping time is close to the temperature of the surroundings (cf. Figure 8.2).

The maximum utilization of the tensile strength is observed (cf. Figure 8.6) at t = 168 hours. The highest utilization is seen at the bottom of the wall and originates from the stress component perpendicular to the cross-section σ_z. The stress distribution corresponds in principle to the distribution at t = 121 hours (cf. Figure 8.13). This means the there is a risk of through-going permanent cracks in the wall.

8.2.3 Stress calculation - cross-section is fixed against flexure

Figure 8.16 shows the maximum tensile stress in relation to the splitting tensile strength for wall and deck as a function of time when the cross-section is fixed against flexure. It is seen that the requirement for maximum 80% utilization is exceeded in the deck during the heating phase. As in the situation with free flexure described in Section 8.2.2, the very high utilization immediately after casting does not present a problem.

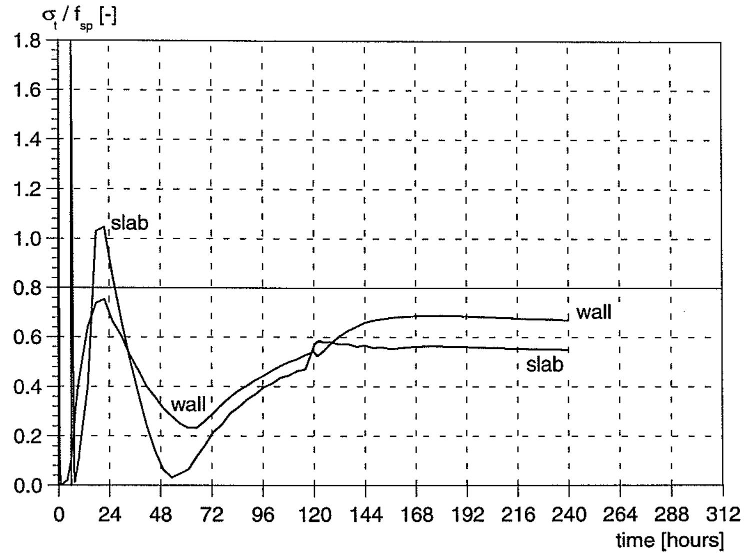

Figure 8.16: Maximum utilization of splitting tensile strength. Flexure prevented.

The stresses in the plane of the cross-section (σ_x and σ_y) are practically independent of whether the cross-section is allowed to bend or not. This means that the conditions concerning the dead load, transverse temperature movements of the deck and the temperature differences between the centre and the surface correspond to the situation with free flexure (cf. Section 8.2.2). In the following sections the stresses perpendicular to the cross- section (σ_z) will be treated.

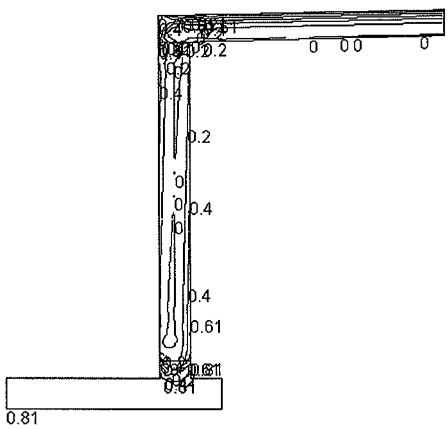

Figure 8.17: Utilization of tensile strength at t = 22 hours. Flexure prevented.

Figure 8.17 shows the utilization of tensile strength at t = 22 hours corresponding to the time of maximum utilization in the deck during the heating phase. There is a high utilization in the top side of the deck with a maximum in the area over the wall corresponding to the situation where the cross-section is allowed to bend freely (cf. Figure 8.8). In the situation with free flexure this was due to the horizontal component in the plane of the cross-section (σ_x) as well as the component perpendicular to the cross-section (σ_z) as described in Section 8.2.2. In the situation where the flexure was fixed, σ_z creates no risk of crack formation. The distribution of σ_z at t = 22 hours is shown in Figure 8.18. The stress distribution in the wall does not vary over the wall height.

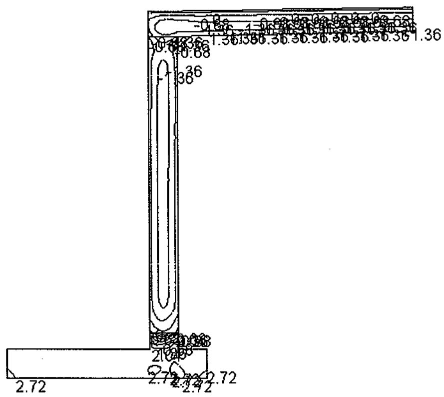

Figure 8.18: Stress component perpendicular to the cross-section σ_z at t = 22 hours. Flexure prevented.

Figure 8.19 shows the distribution of σ_z at t = 121 hours. It is seen that the cooling of the superstructure results in tensile stresses over the whole of the wall height, contrary to the situation with free flexure, where the tensile stress is highest at the bottom of the wall (cf. Figure 8.13). It is observed that the tensile stress in the wall is highest in the situation where the cross-section can bend. On the other hand, the tensile stress in the deck is highest where flexure is prevented.

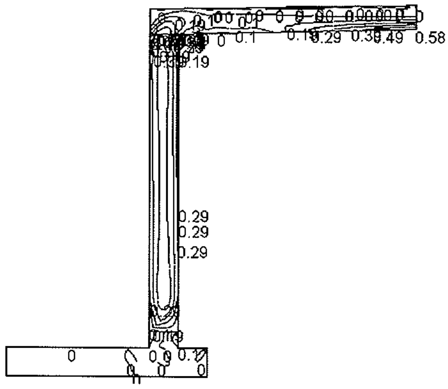

Figure 8.19: Utilization of tensile strength immediately after stripping (t = 121 hours). Flexure prevented.

The maximum utilization of tensile strength is observed cf. Figure 8.16 at t = 168 hours and results from the stress component perpendicular to the cross-section σ_z. The stress distribution corresponds in principle to the distribution at t = 121 hours (cf. Figure 8.19). In this situation there is no risk of crack formation in the wall or the deck during the cooling phase when flexure is prevented. It is, however, difficult to determine in advance which of the two situations will be the most hazardous at every point in the structure at a given time.

8.3 Calculation with hardening measures

8.3.1 Hardening measures

In order to eliminate the risk of longitudinal cracks in the top side of the deck during the heating phase the top side of the deck is insulated with tarpaulins. These also protect against drying out and replace the application of a curing membrane. In the calculation it is assumed that the tarpaulins are placed 1 hour after the casting of the deck.

In order to eliminate the risk of transverse cracks in the top side of the deck during the heating phase and through-going cracks in the walls during the cooling phase the foundations are heated by means of heating wires. 12 heating wires are placed in each foundation at intervals of 0.35 m. The output is 33 W/m per wire. The foundations are insulated by 55 mm winter mats at the time when the heating wires are turned on. The geometry of the foundation, the insulation and the heating wires correspond to Example 5.3. It appears from this example that the chosen measures can heat the foundation by about 10 °C in the course of 72 hours. This is assumed to be appropriate. As described in Section 5.3.5 the foundations and superstructure shall be cooled simultaneously. The maximum temperature of the superstructure is observed about 24 hours after the casting of the wall (cf. Figure 8.2). The heating wires are therefore turned on 48 hours before the casting of the wall and remain on for 72 hours.

Example 5.3 shows that in this case it is relevant to include the soil up to 1 m outside the foundation at each side and to a depth of 3 m (cf. Figure 5.10). The example shows that it will be necessary to calculate for about 60 hours in order to reach a realistic temperature distribution in soil and foundation before the heating wires are turned on when the initial temperature is estimated at 11 °C in the soil and 25 °C in the foundation. The thermal conductivity of the soil is estimated at 6 kJ/m/h/°C, specific heat at 1.1 kJ/kg/°C and density at 1700 kg/m^3.

8.3.2 Results

Figure 8.20 shows the average temperature as a function of time for foundation, wall and deck. The maximum average temperature in wall and deck is about 49 °C and 48 °C respectively. The average temperature of the foundation rises by about 12 °C on account of the heating by heating wires. The foundation and superstructure are cooled at the same time as planned.

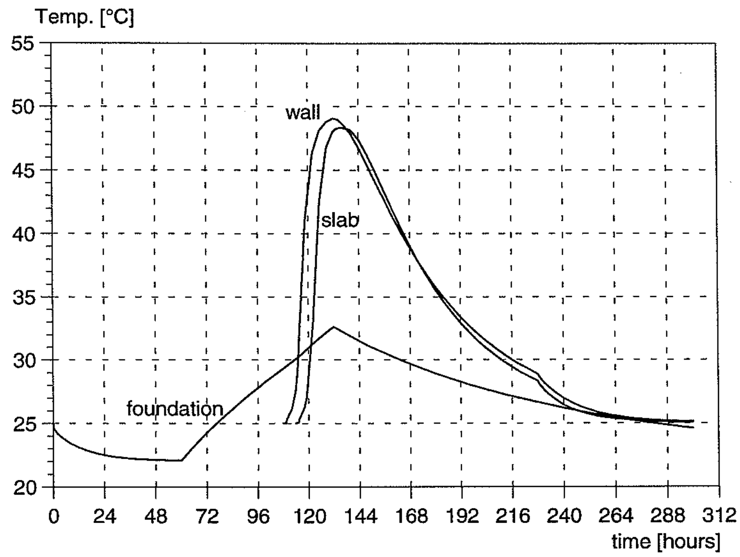

Figure 8.20. Average temperatures. The foundation is heated by heating wires.

Figure 8.21 shows the maximum tensile stress in relation to the splitting tensile strength of wall and deck as a function of time when the cross-section is allowed to bend freely. It is observed that the requirement of maximum 80% utilization has been met for the deck. Apparently the value is exceeded in the wall during the heating phase. The conditions are the same as in the preliminary calculation (cf. Figures 8.7-8.8) because the wall cools before the deck is cast - a situation that does not arise in practice as mentioned in Section 8.2.2. It has been checked that the proposed measures do not result in problems for the situation where the cross-section is prevented from bending. It can therefore be concluded that the chosen solution will satisfy the specified requirements. Figure 8.20 shows that the foundation is not fully cooled at the same time as the superstructure. Therefore the foundation will continue to contract, which will further reduce the risk of through-going cracks later.

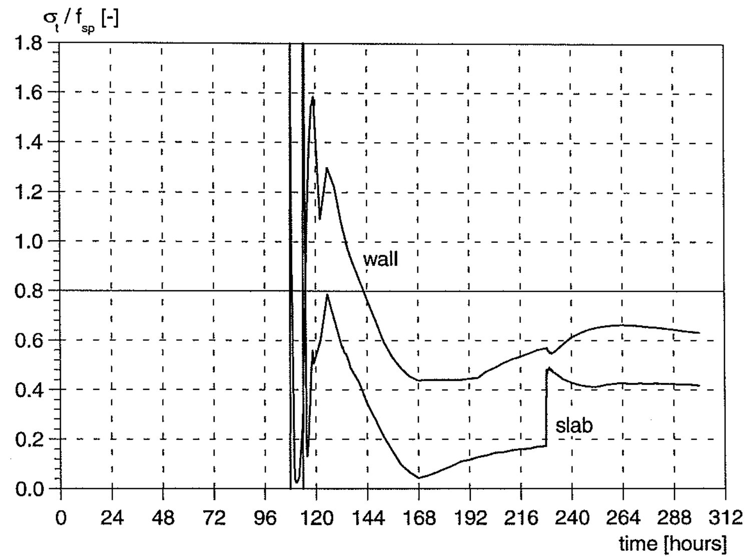

Figure 8.21: Maximum utilization of tensile strength. The foundation is heated by heating wires. Free flexure.

8.3.3 Alternative concrete

In [Riis et al, 1997] the concrete properties of an alternative concrete type are described. This concrete has a w/c-ratio of 0.45. A Portland limestone cement with addition of silica fume and fly ash was used. Figures 8.22 and 8.23 show the average temperatures as well as the utilization of the tensile strength as a function of time for the alternative concrete. Apart from type of concrete the calculation assumptions are the same as those described in Section 8.1. It is assumed that the cross-section bends freely. The hardening measures are the same as described in Section 8.3.1.

Figure 8.22 shows that the maximum average temperatures in wall and deck are about 64°C and 63°C respectively, i.e. about 15°C higher than was the case for the first concrete type (cf. Figure 8.20). The average temperature of the foundation corresponds to the development observed previously (cf. Figure 8.20).

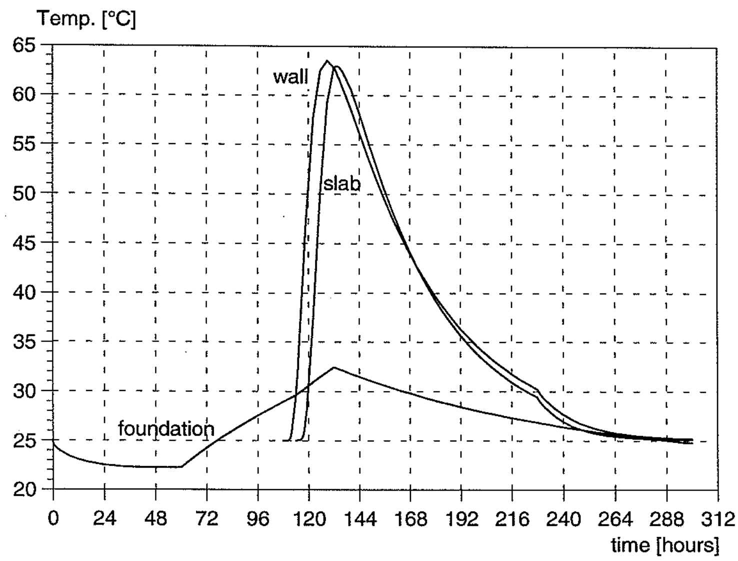

Figure 8.22: Average temperatures using alternative concrete. The foundation is heated by heating wires.

Figure 8.23 shows that the wall exceeds the requirement of maximum 80% utilization over the whole period. The excess during the heating phase is however due to the wall in the calculation cooling in the period prior to the casting. As this cooling - as mentioned in Section 8.2.2 - does not take place in practice, the high utilization in the wall during the heating phase does not constitute a problem. But the high utilization in the wall during the cooling phase is a problem. The maximum utilization of the tensile strength is observed at the bottom of the wall and originates from the stress component perpendicular to the cross- section σ_z. In principle the stress distribution corresponds to the distribution shown in Figure 8.13. This means that there is a risk of through-going permanent cracks in the wall.

It is seen that the utilization of the tensile strength in the deck exceeds 80% during the heating phase. The high utilization is observed in the top side of the deck in the area over the wall. The problem originates from the horizontal component in the plane of the cross-section σ_x and constitutes a risk of temporary longitudinal cracks in the top side of the deck.

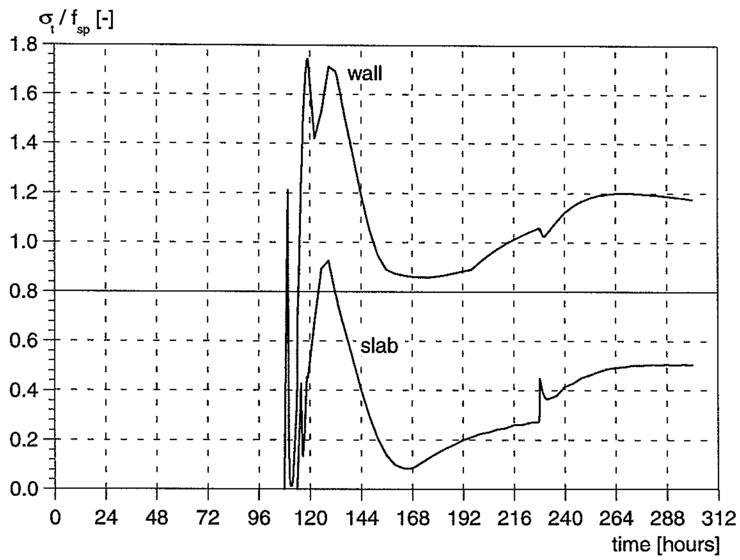

Figure 8.23: Maximum utilization of splitting tensile strength when alternative concrete is used. The foundation is heated by heating wires. Free flexure.

9.0 Literature

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Appendix A

Isolating capacity

Appendix A Isolating capacity

The transmission coefficient of a surface acted upon by wind can be calculated from:

$$\alpha = \left(\frac{1}{\alpha_k} + \left(\frac{e}{\lambda}\right)_i + \left(\frac{e}{\lambda}\right)_f \right)^{-1} \quad [\mathrm{kJ/m^2/h/^\circ C}]$$

with

$$\alpha_k = 20 + 14 \cdot v \qquad v \leq 5 \text{ m/s}$$
$$\alpha_k = 25.6 \cdot v^{0.78} \qquad v > 5 \text{ m/s} \qquad \textbf{(2)}$$

where

e	thickness of insulation and form respectively
λ	thermal conductivity of insulation and form respectively [kJ/m/h/°C]
v	wind velocity [m/s]

For turbulent flow in cooling pipes, the transition resistance between cooling water and pipe-wall is neglected. The transmission coefficient of the cooling pipe can then be determined as:

$$\alpha = \frac{\lambda}{e} \quad [\mathrm{kJ/m^2/h/^\circ C}]$$

where

e	wall thickness of cooling pipe [m]
λ	thermal conductivity of the pipe material [kJ/m/h/°C]

Thermal conductivities and transmission coefficients of a number of materials are given in Table A.1.

Materials	Thermal conductivities [kJ/m/h/°C]	Transmission coefficient [kJ/m²/h/°C]		
		v= 0 m/s	v= 5 m/s	v= 10 m/s
PEL cooling pipe (plastic)	1.152	-	-	-
Tarpaulin (alfa= 24,8)*	*	11.1	19.4	21.4
Winter mat 1 x 55 mm	0.174	2.7	3.1	3.1
Winter mat 2 x 55 mm	0.174	1.5	1.6	1.6
Winter mat 3 x 55 mm	0.174	1.0	1.0	1.0
Dow-mat 1 x 10 mm	0.144	8.3	12.4	13.2
Dow-mat 2 x 10 mm	0.144	5.3	6.7	6.9
Dow-mat 3 x 10 mm	0.144	3.9	4.6	4.7
Plastic foil with point contact	-	28.0	57.4	75.0
Plastic foil with 5 mm air space	-	17.8	25.0	28.0
System formwork, 18 mm plywood	0.5	11.6	21.2	23.5
Plank formwork 32 mm	0.43	10.7	13.3	14.2
System formwork 18 mm plywood + 55 mm winter mat	0.5 + 0.174	2.5	2.8	2.8
10-30 mm water film on the concrete surface	-	50-> 70	-	-
Sprinkled water on the concrete surface	-	200	-	-
6-8 mm steel form	200	20.0	90.0	153.4
Unprotected concrete surface	-	20.0	90.0	153.4

Table A.1: Thermal conductivity and transmission coefficient

Appendix B

Climatic conditions in Denmark

Appendix B Climatic conditions in Denmark

Table B.1 gives climatic conditions in Denmark, the Faeroes and Greenland for each month. Information on specific localities can be obtained from the Danish Meteorological Institute. The Table is taken from SBI Anvisning 125.

Denmark		J	F	M	A	M	J	J	A	S	O	N	D
Absolute max.temperature [1] ...	°C	11.8	15.5	21.2	28.2	32.8	35.5	35.3	36.4	32.3	24.1	18.5	14.5
Max. temperature for month [2] .	°C	6.8	6.7	10.7	16.5	23.6	26.0	26.9	24.8	21.5	16.4	10.9	8.2
Max. temp., 24-hour average [3] .	°C	2.0	2.2	5.0	10.2	15.7	19.0	21.1	20.6	17.2	12.0	7.2	4.1
Average temperature [3]	°C	-0.1	-0.4	1.7	6.2	11.1	14.5	16.6	16.3	13.1	8.7	4.9	2.2
Min. temp., 24-hour average [3] ..	°C	-2.4	-3.0	-1.3	2.4	6.3	9.7	12.2	12.2	9.7	5.9	2.6	0.1
Min. temperature for month [2] ..	°C	-9.9	-10.0	-7.2	-3.0	0.5	4.5	7.3	7.0	2.9	-1.4	-5.2	-8.3
Absolute min.temperature [1] ...	°C	-31.0	-29.0	-27.0	-19.0	-8.0	-3.5	-0.9	-2.0	-5.6	-11.9	-21.3	-24.4
Number of summer days in month, [4], max. > 25°C		-	-	-	-	0.6	2.1	3.8	3.4	0.3	-	-	-
Number of ice days in month, [4], max. < 0 °C		8.9	8.5	2.8	0.0	-	-	-	-	-	-	0.1	2.8
Number of frost days in month, [3], min. < 0 °C		21.0	19.0	19.0	6.0	1.0	0.0	-	-	0.1	2.0	6.1	14.0
Number of windy days [4], wind velocity ≤6m/s		5.0	3.9	4.6	4.0	3.0	2.6	2.2	2.5	2.8	3.4	4.2	4.3
Average wind force [5], 0-12		4.0	3.7	3.8	3.4	3.0	3.2	3.3	3.5	4.4	3.8	4.5	4.2
Faeroes		**J**	**F**	**M**	**A**	**M**	**J**	**J**	**A**	**S**	**O**	**N**	**D**
Average temperature [6]	°C	3.4	2.2	2.8	4.6	6.5	9.7	10.3	10.7	9.5	7.0	4.2	3.6
Greenland		**J**	**F**	**M**	**A**	**M**	**J**	**J**	**A**	**S**	**O**	**N**	**D**
Average temp. [7], Angmagssalik .	°C	-7.6	-8.5	-10.9	-5.3	0.9	4.5	5.8	5.6	3.4	-2.2	-4.6	-7.7
Average temp. [7], Prins Christians Sound	°C	-2.7	-4.1	-4.4	-1.7	1.9	3.8	5.9	6.3	4.2	0.8	-1.1	-2.4
Average temp. [7], Egedesminde .	°C	-12.5	-13.4	-17.0	-9.6	-1.3	2.5	5.8	5.2	2.3	-2.3	-5.9	-9.5

Table B.1: Climatic conditions in Denmark, the Faeroes and Greenland, from the Statistical Yearbook of Denmark, 1971.

1) The period 1874-1971, 2) Average for the period 1886-1925, 3) Average for the period 1931-1960, 4) Average for the period 1938-1960, 5) 1978, 6) The period 1966-1970, 7) The period 1967-1970.