

Temperature Matched Curing Systems

by A. Williams

Synopsis: Early age concrete is subject to a temperature rise due to the hydration of the cement. Cube or cylinder reference specimens stored alongside a structure will not exhibit this temperature rise to the same degree and therefore will not have the same maturity as the concrete in the structure. Temperature matched curing is a method whereby cubes or cylinders of a similar maturity to the structure are produced, and is an extremely efficient method of determining minimum formwork striking and prestressing time.

This paper gives a brief account of the history behind temperature matched curing in the United Kingdom, where the recent issue of a British Standard Institution document has focussed attention to the subject.

Details are presented of the design and construction of a portable matching system for use on site. The system which is extremely robust, (but light enough to be carried by one man), is able to control cube temperatures to within $\pm 1^{\circ}\text{C}$ of the concrete temperature within a structure.

Results are given which show the advantages and disadvantages of such a system and illustrate the difference of maturity and hence strength between reference cubes stored in the curing bath and reference cubes placed next to the structure itself.

Keywords: age-strength relation; concretes; curing; hydration; portland cements; temperature; tests.

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INTRODUCTION

An estimation of the strength of concrete in a structure is obtained by testing reference specimens cast at the same time as the structure. The strength of concrete is a function of the materials used and their mix proportions, also its moisture state and maturity. It is unlikely that reference cubes or cylinders will have the same strength as the *in-situ* concrete due to variations in mix proportions, compaction and moisture level. However for mature concrete, an allowance for these differences can be made, and this is taken into account in the partial factor for safety for concrete in design codes. For early age concrete, gross differences in maturity can occur between the reference specimens and the *in-situ* concrete. This difference is minimised by producing test cubes of equal maturity to the structure.

Temperature matched cubes, where the reference cubes follow the same temperature as the parent concrete have the same maturity as the parent concrete, and therefore give a more realistic estimation of the strength of the structure.

The object of this paper is to describe the basic elements of a temperature matched curing system, to briefly relate the history of temperature matched curing in the United Kingdom, and finally to give details of typical results obtained on site using a portable temperature matching system.

BACKGROUND INFORMATION

The hydraulic reaction that takes place when cement hydrates is exothermic in nature and usually causes the temperature of the *in-situ* concrete to rise considerably. The larger the mass of concrete, the greater will be the rise in temperature, with a consequent increase in the rate of strength development. Reference test cubes or cylinders stored alongside the structure will also exhibit a rise in temperature. Because of the much lower mass of concrete involved, and the proximity of the steel sides of the mold, the temperature rise exhibited by the reference specimens will be much less than that of the parent concrete. Because of the difference in maturity, the test specimens will underestimate the strength developed by the *in-situ* concrete by as much as 50% during the early age of the structure. Temperature matching is a method whereby cubes of the same maturity as the

structure are actually produced, this is an obvious advantage over using the maturity method, as no assumptions or calculations have to be carried out.

The basic elements of a matched curing system are shown in Figure 1 and consist of:

- a) curing bath fitted with a water circulator and heater;
- b) two temperature measuring devices, one placed in the curing bath and a second used to measure the temperature of the *in-situ* concrete;
- c) a control box which monitors the output from the temperature sensors and switches on the heater as required.

In temperature matched curing, the reference test molds are filled and compacted in the normal manner, but the top open surface is covered by a flat steel or glass plate prior to placing the mold in the water bath. The function of the cover plate is to prevent the concrete from being washed out of the mold by the water circulating in the tank, but it does not form a complete seal. The intention is to allow sufficient water to reach the cube, to give wet curing conditions. Alternatively, by using a steel plate with a rubber seal, the water can be prevented from reaching the concrete if required.

The basic ideas behind temperature matching have been known for a considerable time. Davey (1) published a paper in 1933 which describes in detail an adiabatic curing system (Figure 2), in which the reference specimens were constrained to follow the temperature of an enclosed volume of concrete. In that particular case, the control sample was placed in an insulated flask, which itself was placed in the curing tank. By subjecting this control sample to adiabatic curing conditions, it served to represent the centre of a mass concrete structure.

Since that time, a number of papers have been presented on temperature matched curing (2, 3 and 4) two of these relate to laboratory facilities which are much too large or complex to be used on a construction site. The paper by Blakey (2) described a portable system for use on site. A number of systems based on this design were manufactured and used, but operational problems with the equipment eventually led to them being withdrawn from general use, although several still exist.

In the United Kingdom, interest in temperature matched curing has been stimulated recently by the issuing of a British Standard Draft for Development (DD 92:1984) (5) which covers the subject, and details the specific requirements of a temperature matching system. The portable unit shown in Figure 3 was designed to meet this specification, for use on site. The system consists of a microprocessor based temperature controller mounted in a portable steel cabinet. Temperatures are measured by platinum resistance

thermometers housed in stainless steel probes, one positioned in the bath and the other, an insertion probe (Figure 4) in the *in-situ* concrete. This probe is recoverable after the concrete has set. The controller is capable of maintaining the bath temperature to within $\pm 1^{\circ}\text{C}$ of the temperature measured in the *in-situ* concrete. The temperature probes are connected to the control unit by armoured cables, enabling the unit to be sited at a place of safety at a distance of up to 50 metres from the measuring point. A chart recorder is also fitted in the unit and records the ambient temperature as well as the bath and temperatures. The size of the tank was designed to easily accommodate ten 100 mm cube molds.

SITE RESULTS

One of the arguments in favour of using temperature matched curing is that it is an extremely efficient method of determining minimum formwork striking times. Possibly the main problem associated with temperature matching is that of ensuring the probe measures a representative temperature. A guide to the positioning of the sensor is given in Reference 5.

The following examples illustrate how temperature matched curing systems have been used on different structures.

1. 300mm Wide Wall

As can be seen from Figure 5, a 300 mm wide concrete wall cast in insulated steel formwork showed an increase in temperature of around 17°C some 23 hours after casting. Cubes cured alongside the walls followed the changes in ambient temperature. The difference in strength between these cubes and a second set which were placed in a matched curing tank is readily apparent (Table Figure 5). After some 24 hours, the matched cubes are approximately twice as strong as the 'normal' cubes, and the 3 MPa strength difference is still apparent after 67 hours.

2. 1 Metre Wide Wall

The heavily reinforced blast wall shown in Figure 6 was cast in 10 m long by 3 m high sections (half height sections) in insulated timber-faced forms. For these sections the rise in temperature due to hydration at the centre of the pour was required to be measured, so a thermocouple positioned at the centre was connected to the chart recorder in place of the ambient temperature probe normally used. The difference in temperature between the thermocouple and the probe which was inserted 50 mm into the concrete through one of the side forms would give an indication of the temperature distribution in the wall. The centre of the wall was found to reach a temperature of around 36°C some 27 hours after casting, as compared with a temperature of 27°C near the outer surface (Figure 6).

3. Segmental Box Bridge

For other structures however the position of the insertion probe can be more critical. Figure 7 shows one of the diaphragm units for a glued segmental concrete box bridge which forms part of a single-carriageway road system. The bridge has 8 spans with a maximum span length of 90 m, and grade 52.5 concrete is specified for the precast segments. As the factory is required to produce one segment per day, the soffit forms on the side cantilevers have to be stripped 14 hours after casting, at which time a cube strength of 10 MPa is required. Cubes cast and stored on top of the side cantilevers underneath a thermal insulating blanket reached a strength of 1 MPa at 14 hours despite the fact that warmed water was used when the concrete was mixed. It was anticipated that a concrete temperature of 35°C was required throughout the unit to achieve the specified strength of 10 MPa.

In order to determine typical concrete temperatures, thermocouples were used when the next segment was cast to monitor the temperatures at points in the unit. The concrete temperatures measured in that segment are shown in Figure 8 and it can be seen that whereas the sides and bottom of the box reached temperatures around 35°C, the more exposed cantilevers only reached a temperature some 10°C lower than that value. In this case cubes stored under the thermal blanket had zero strength at 14 hours. Cubes placed in the temperature matched curing bath with the probe positioned in one of the side cantilevers reached a strength of 3.5 MPa.

To carry out a site investigation where the temperature variation throughout the structure is unknown, it is possible to cast 50 mm long stainless steel tubes (closed at the inner end), into the concrete at points where the temperature measurement is required. The ambient temperature measuring probe can be moved from pocket to pocket, allowing a stabilising time of a few minutes in each position. In this way the position in the structure which is giving the lowest temperature increase as recorded on the chart recorder can be determined, and the probe then left in that position.

LABORATORY USAGE

In addition to site testing, temperature matched curing is obviously of use in the laboratory. The use of admixtures and cement replacement materials often change the peak temperature attained due to hydration, also the strength of the concrete. Peak hydrating temperatures which are in excess of 65°C might significantly adversely affect the properties of the concrete.

Typical results from laboratory tests in which the portable matching system was used as shown in Figure 9. For these tests, control specimens were cast in cylindrical plastic moulds encased in very thick thermal insulation. The object was to produce adiabatic curing conditions in a manner similar to that described

by Davey (1). The temperatures measured in the control specimens are shown in the figure, as are the compressive strengths obtained from the reference specimens.

CONCLUSION

Temperature matched curing is a method which can readily be used on site for producing cubes or cylinders which are more representative of the strength of the structure. It must be emphasised that the method does not reproduce *in-situ* conditions, but by exactly matching the temperature changes, it does eliminate one of the main reasons for the difference in properties between reference specimens and site concrete.

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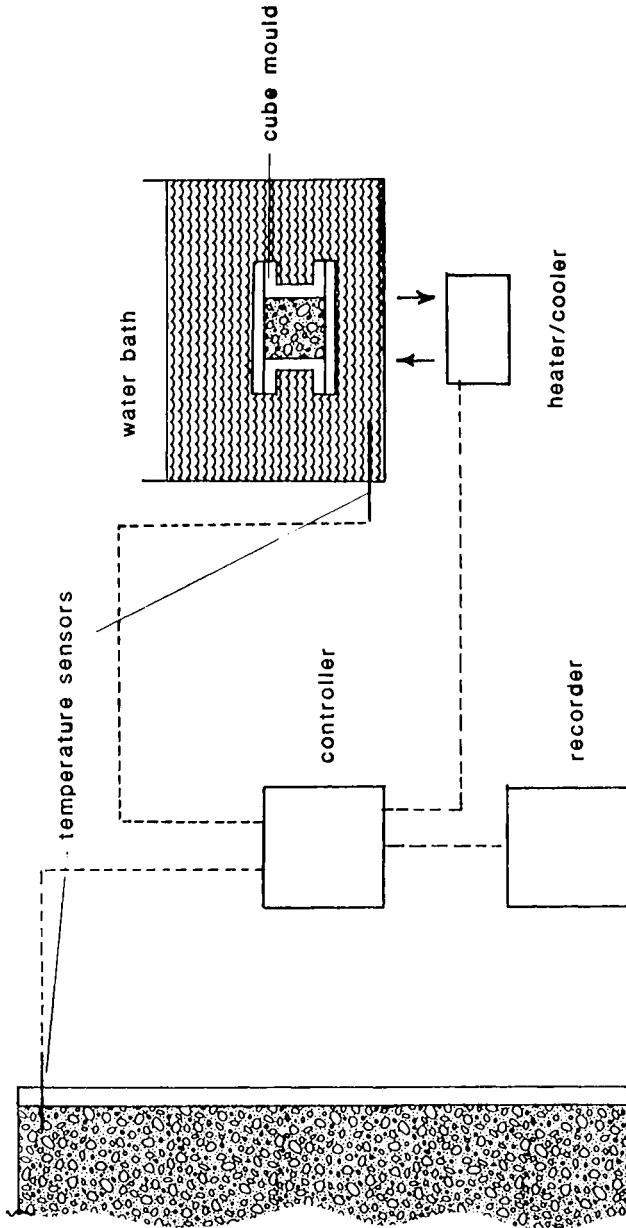


Fig. 1--Temperature matched curing system

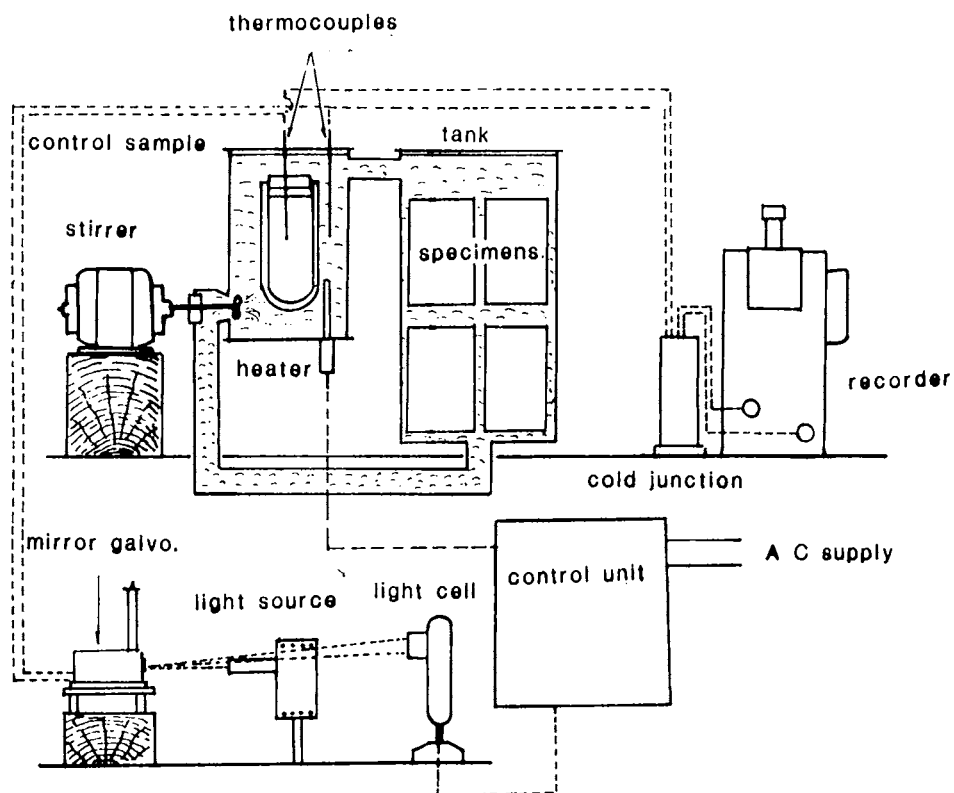


Fig. 2--Davey system circa 1933

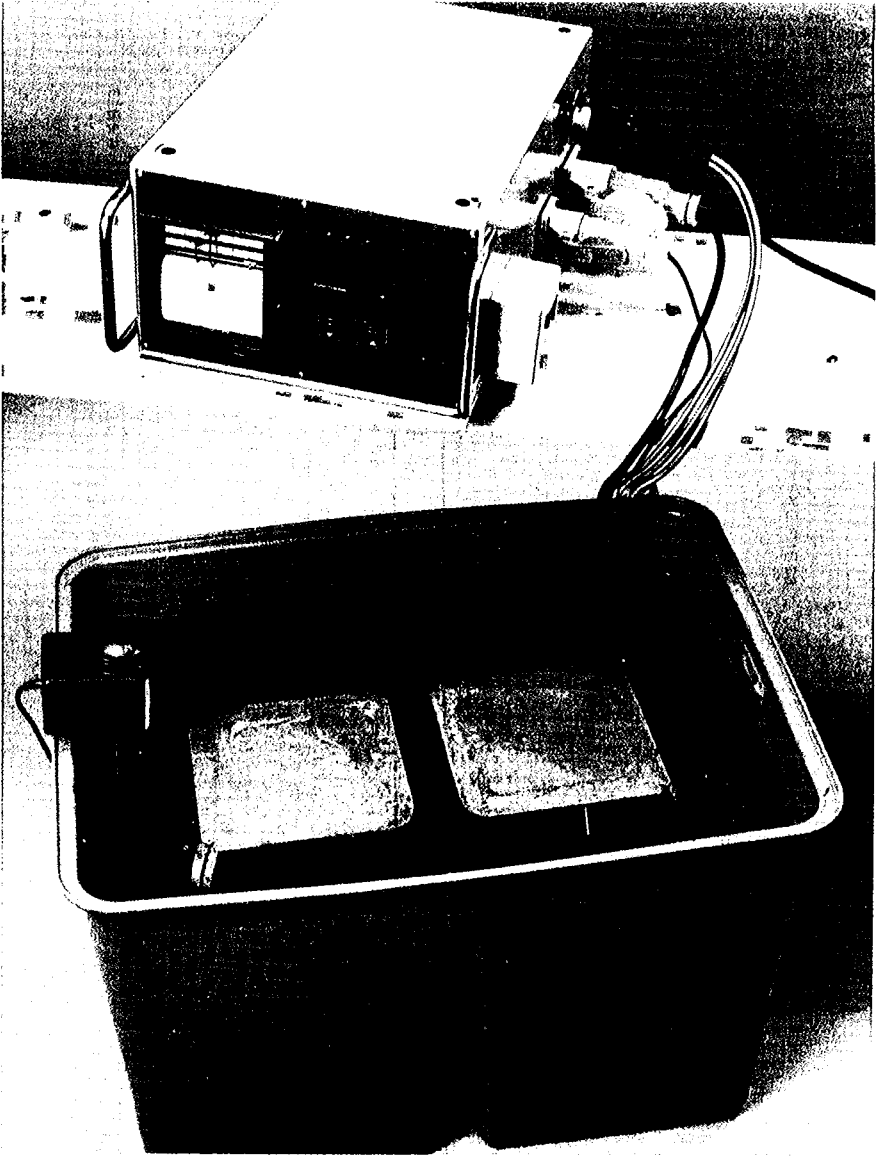


Fig. 3--Portable temperature-matching system

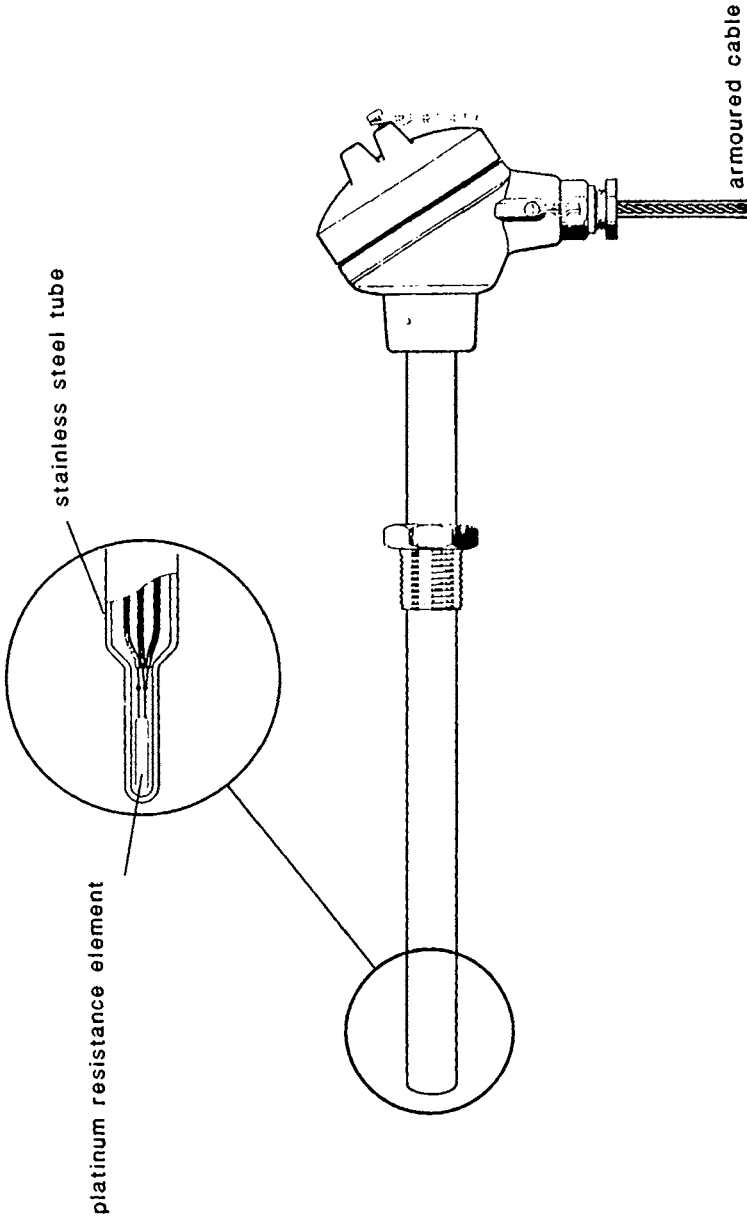
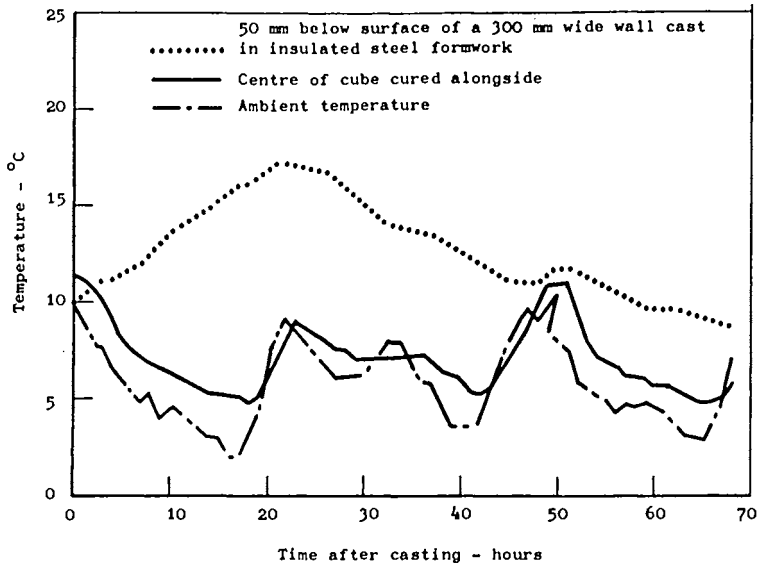


Fig. 4--Insertion probe



Average strength of 3 cubes - N/mm^2

Curing method	Time after casting		
	19 h	26 h	67 h
Cubes cured alongside	1.6*	3.9	13.3
Temperature-matched cubes to a point 50 mm below surface of the wall	3.6	7.1	16.8

* This does not satisfy a 2 N/mm^2 striking criterion for vertical forms.

Fig. 5--Wall results (Harrison)

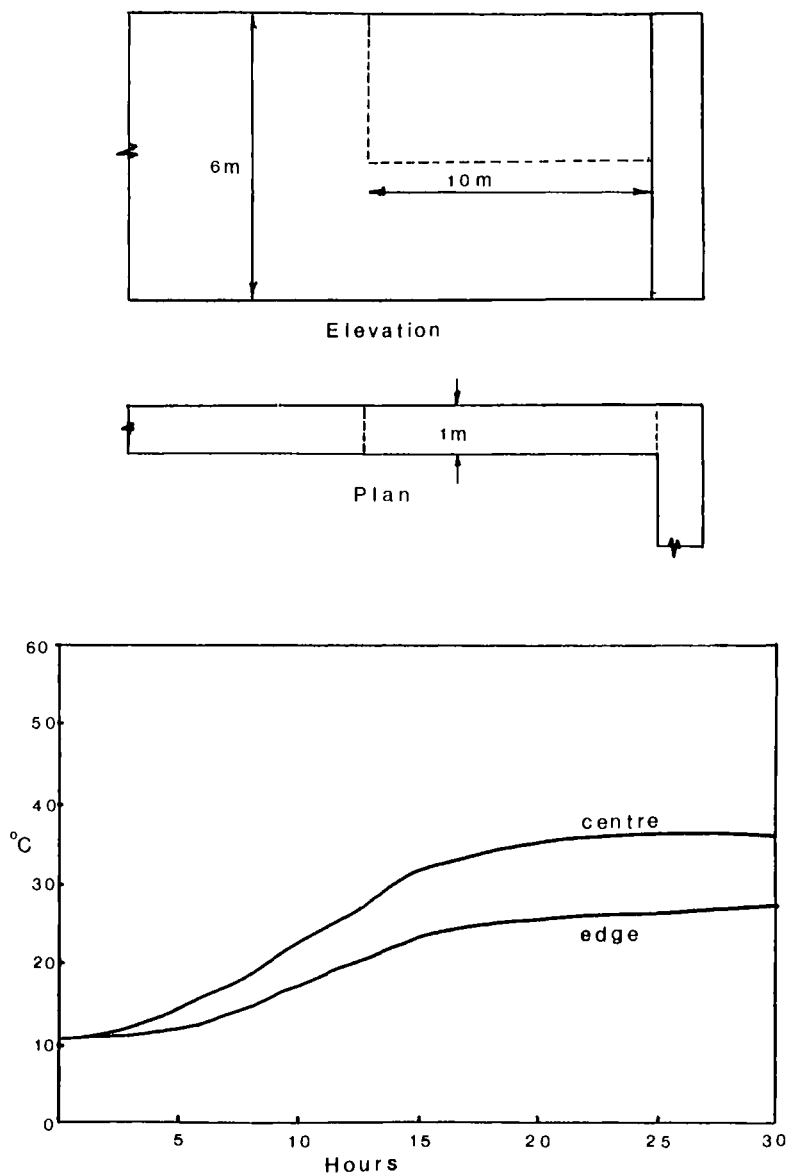


Fig. 6--Blast-wall results

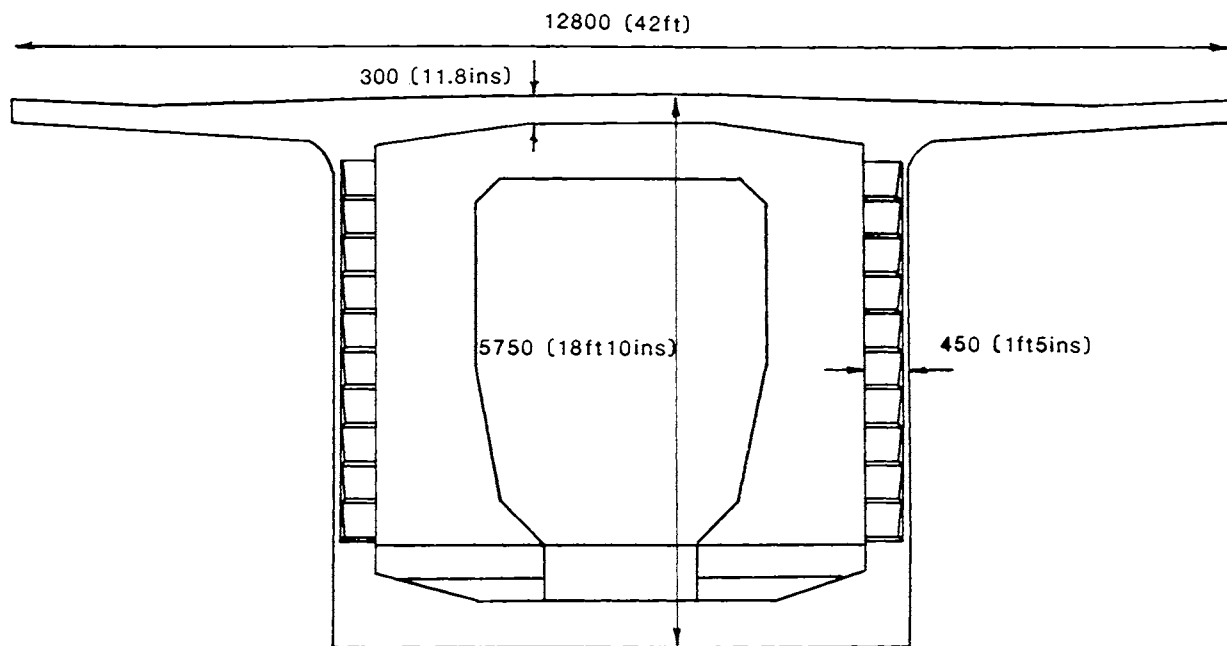


Fig. 7--Segmental bridge unit

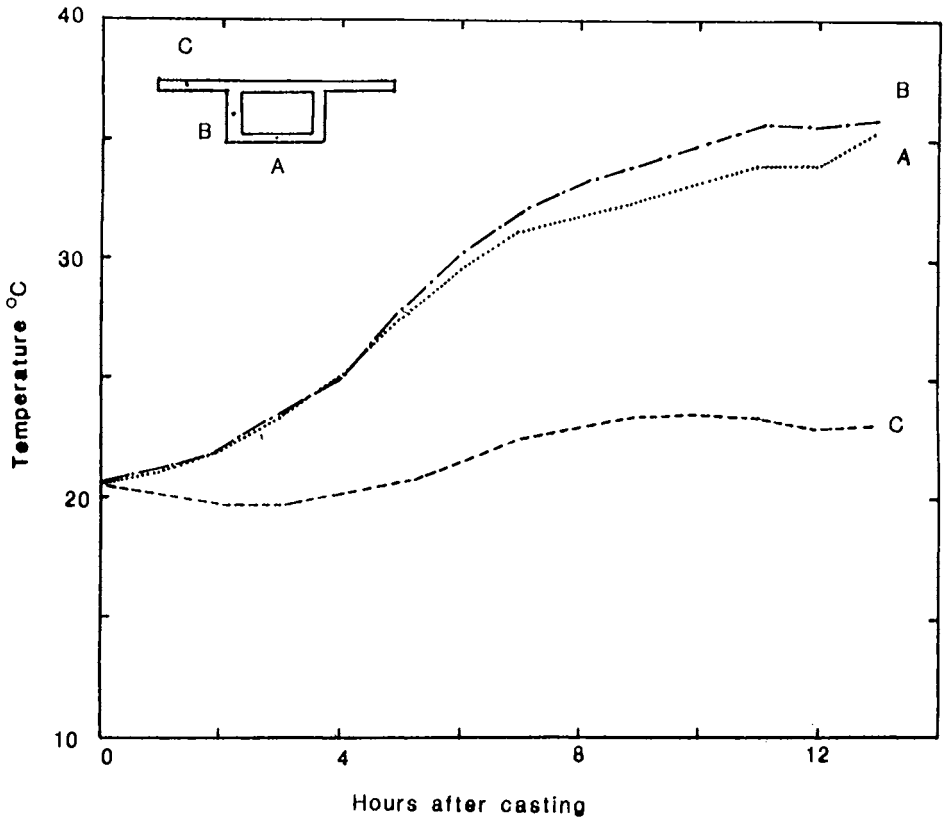


Fig. 8--Temperature changes due to hydration

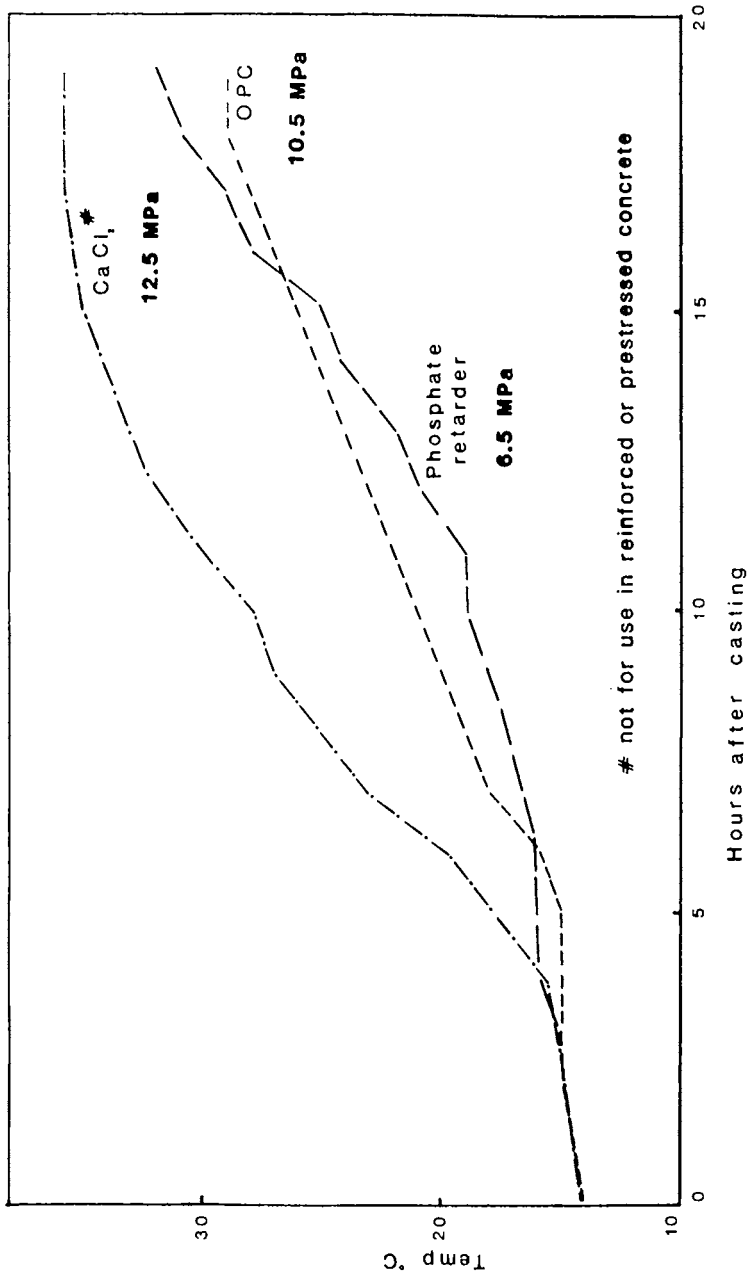


Fig. 9--Effect of admixtures

Curing Technology at the Faroe Bridges, Denmark

by H-H. Gotfredsen and G.M. Idorn

Synopsis: The construction of the Faroe Bridges between Sealand and Falster in Denmark was completed early 1985. The project included the placement of about 42,000 m³ of concrete for the substructures of the bridges. Monitoring the curing of the concrete was an essential part of the efforts invested to assure satisfactory durability of the structures.

The operations included:

- I. Pre-calculations of the heat and strength (maturity) development after placing concrete in critical sections. The requirements to be complied with were (1) maximum temperature, (2) maximum temperature differences in the concrete, and (3) attainment of sufficient maturity before form-removal.
- II. Site-control with continuous recording of the temperature development at essential positions in the concrete, including assessment of the strength development. Means for moderating the predetermined course of the curing, in case of unacceptable deviations, were used.

The expenses for monitoring curing technology were deemed reasonable in view of the directly documented assurance that the required quality of the concrete in the structures was obtained.

Keywords: bridges (structures); concretes; curing; heat of hydration; measurement; quality control; strains; temperature

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INTRODUCTION

The Faroe project comprises two bridges with a total length of 3,322 m (Fig. 1). The Northern bridge between Sealand and Faroe consists of sections with spans of 80 m and a total length of 1,596 m. The Southern bridge connects Faroe with Falster. The central part is constructed as a cable stayed bridge, the span of the central section is 290 m long and the two side spans are 120 m long. The spans of the remaining 15 sections are 80 m long. The total length of the bridge is 1,726 m. The superstructures of both bridges are constructed as closed steel box girders, supported by reinforced concrete pier shafts. The two cable towers (pylons) are of reinforced concrete. The placement of concrete took place from March 1981 to June 1984.

Previous Danish experience has revealed two important factors for the assurance of long-term durability of concrete bridge structures: a high level of concrete quality by selection of materials, concrete composition and processing technology; and the skill and motivation of the professionals and the workmen on the job-site.

High priority was given to preventing deleterious alkali-silica reactions, which recently have proven to be an increasing threat to the durability of many road bridges, as well as to concrete structures, in general, in Denmark. Strict requirements for sand and coarse aggregate were specified, and a special, low-alkali cement was chosen. However, impermeable and crackfree concrete was believed to be the most important requirement for concrete structures, if they are to sustain the wet and harsh Danish climate, the use of de-icing salts, and other possible causes of chemical attack.

Thus, to avoid early cracking, it was necessary to maintain control of the developing temperature strains and the ensuing tensile stresses caused by the constraints of these strains on the one hand, and the developing tensile strength in the concrete during the hardening processes on the other.

From previous bridge constructions it was observed that the available curing technology was applicable as processing control. The interest in curing technology commenced in Denmark in about 1950. Early research then concentrated on curing at low temperatures, aiming at methods for assuring strength development of in-place concrete during the prolonged periods of hardening, and also for attainment of resistance to freezing at early ages. At the end of the 1950's, a major government program for development of new building systems took effect, advancing prefabrication of concrete facade panels, slabs and walls in a sophisticated modular system. This brought attention to accelerated curing of concrete at elevated temperatures, such as by steam-injection and steam-curing. At the end of the 1960's, a 4-year research program with leading Danish prefab-corporations and the Concrete Research Laboratory, Karlstrup, developed a curing technology system including a maturity-meter which is based upon the Arrhenius law for the temperature dependence of strength development.

At the beginning of the 1970's, systematic bridge inspections carried out by the Road Directorate revealed extensive deterioration of newly built concrete bridges in spite of heavy expenditures in the design, specifications, supervision and construction. A 5-year research program into the causes of the many failures resulted in a gradual change in the design and construction specifications of concrete bridges in Denmark. More attention was given to the increased risk of subsequent deterioration resulting from early cracking.

The design and execution of the Faroe bridges represents a culmination of these efforts. The basic principle of the applied curing technology is that the strength and impermeability development of the concrete during its hardening in the structure is monitored by recording the temperature generation in the cast concrete, measured at suitable locations. On the basis of the maturity principle, the actual strength development is estimated. This is in contrast to conventional concrete control which relies on the strength of standard test specimens, stored at a constant temperature of +20°C.

The contract conditions and specifications for the bridge construction required that the contractors present curing technology calculations for each concrete casting. From the start of the work a system was established for monitoring the curing process of each concrete casting. The curing control methodology ensured the required concrete strength in the bridge structures within the fixed time limits, while concurrently reducing the risk of cracks induced by thermal stresses. This was achieved despite some castings in deep water, others above sea level, with some of the structural components heavily reinforced. In addition, the work was executed during calm summer days as well as during windy days in winter.

Requirements

The specifications for the concrete processing required that the contractor should ensure:

1. That the temperature of the concrete did not exceed $+60^{\circ}\text{C}$ ($+70^{\circ}\text{C}$ in large concrete volumes, i.e. foundations, provided approval was obtained).
2. That the concrete remained crackfree throughout the curing phase.
3. That the concrete was not exposed to freezing before adequate frost resistance had been achieved.
4. That the moisture content everywhere in the concrete, and especially at surfaces, was sufficient to obtain the required concrete properties with regard to impermeability and appearance through uninterrupted hardening.

These requirements were adapted to the actual conditions by the choice of cement type, cement content, temperature of fresh concrete, etc. In slabs or walls with a symmetrical temperature profile, and for which thermal expansion was not restrained by adjoining structures, the temperature difference between the surface and the middle of the slab or wall was at no time to exceed 20°C . The mean temperature difference between previously cast slabs and walls and adjoining newly cast slabs and walls of approximately equal thickness was at no time to exceed 20°C .

The concrete became frost resistant when a given maturity age had been achieved. Measures were taken to protect the concrete against drying during the curing until all surfaces had reached a maturity age corresponding to 7 days of continuous storage at 20°C .

When applying the maturity function normally used in Denmark (see also Eq(2)) this equals the following periods:

Concrete temperature at the surface	Period of curing measures
20°C	7 days
15°C	10 days
10°C	14 days
5°C	22 days

In the contractor's planning phase preceding the execution of the works, the curing phase had to be verified through trial testing. Calculations could be made on the following basis, provided that results were not available from experiments or measurements:

- The temperature variations under the expected weather conditions were calculated for typical transverse sections.

- The calculation of the heat exchange between the structure and the surroundings should include contributions from convection, radiation and evaporation.
- The specific heat of the hardening concrete is $1.1 \text{ kJ/kg}^\circ\text{C}$.
- The heat conduction of the hardening concrete is $8 \text{ kJ/m}^\circ\text{C h}$ ($2.2 \text{ Watt/m}^\circ\text{C}$).
- The heat generation of the concrete should be calculated on the basis of results from adiabatic calorimetry, measured on the concrete compositions used.
- The strength development of the concrete should be calculated on the basis of measurements of the strength development of comparable test specimens at a constant temperature of 20°C and 100% RH.
- The calculations of the heat generation and the strength development in the structure should include the effects of temperature, wind and relative humidity on the rate of hydration.

The contractor had to document his ability to comply with the curing requirements even if unfavourable climatic conditions should occur during the hardening period. The documentation had to be presented before to each pour in the form of an "execution manual".

Planning and monitoring of concrete curing process

In order to understand the principles underlying monitoring of the concrete curing processes, it is useful to consider three factors:

1. The rate of the hardening process is controlled mainly by the concrete temperature. An increase in the concrete temperature speeds up the process, whereas a decrease slows it down. At 35°C the hardening rate is approximately twice the rate at 20°C , and at 10°C it is one-half the rate at 20°C , and this is true for almost all types of cement concretes.
2. The concrete hardening generates heat. At complete hydration, 1 kg of portland cement generates from 400,000 to 500,000 Joule. In a typical concrete mixture, this will lead to a temperature increase of 60 to 80°C , provided no heat is lost to the surroundings during hardening.
3. The temperature development in a hardening concrete structure is controlled by the balance between heat generation in the concrete and heat transmission to the surrounding air. Consequently, in massive or heavily insulated structures, the generated heat is not readily transmitted to the surrounding air and the temperature rise is large. In slender, uninsulated structures, the generated heat is easily transmitted to the surrounding air, and the initial temperature increase is therefore small.

It appears from these factors that the most important element in monitoring the curing process is the control of the heat development, and knowing what portion of the heat is retained as a temperature increase in the concrete, and what portion is transmitted to the surrounding air. In practice, a number of options exists for managing this balance between heat generation and heat transmission.

Monitoring Parameters:

The balance between the heat generation and the heat transmission is influenced by a number of factors. Among these factors, construction dimensions and climate cannot be altered, or can be altered only slightly, by the contractor. On the other hand, concrete composition, cement type and cement content can be changed to some degree. However, the casting temperature of fresh concrete, form work type, insulation level and time of form work removal are the major parameters which can be made the prime objectives of planning and execution of concreting operations.

The most expedient construction methods will depend on the given site conditions. The choice of methods will most frequently have economic consequences. Time schedules and depreciation of equipment will, therefore, often be decisive.

Generally, the Faroe Bridges' dimensions are so large that the main problem connected with monitoring the curing period was to obtain a suitable hardening process.

The monitoring took its starting point in a quantification of the basic phenomena:

- The effect of temperature on the hardening rate was established by calculations using appropriate temperature functions.
- The heat generation properties of the concrete were measured and described with suitable mathematical models.
- The heat balance for a cross-section of hardening concrete, with a non-linear heat source, was treated by means of numerical calculations.
- Through appropriate mathematical models for the phase composition of the hardening concrete, criteria for obtaining frost resistance during early curing were established.
- From the knowledge of the temperature distribution in the hardening concrete, including its state during abrupt cooling after form work removal, operational rules for prevention of cracking were established.

Temperature Dependence of The Hardening Process:

The rate of the chemical reactions between cement and water increase with temperature. If the rates at 20°C and at 0°C are compared, the approximate relation will be:

$$H(T) = \exp. \left[\frac{E}{R} \left(\frac{1}{293} - \frac{1}{273 + T} \right) \right] \quad (1)$$

where

$$\begin{aligned} E &= \text{characteristic activation energy} \\ &= \begin{cases} 33500 \text{ J/mol} & \text{for } T \geq 20^\circ\text{C} \\ 33500 + 1470(20 - T) \text{ J/mol} & \text{for } T < 20^\circ\text{C} \end{cases} \\ R &= \text{the gas constant, } 8.314 \text{ J/mol } ^\circ\text{C} \end{aligned}$$

In figure 2 the temperature function is displayed for the temperature interval -10°C to 90°C. The temperature function gives the temperature dependence of the rate of hardening in proportion to the rate at 20°C (1). The reference temperature 20°C has been chosen because it is generally used as a reference for standard testing according to Danish Standard (DS411).

The use of the temperature function makes it possible to compare hardening processes at a temperature T with a previously known hardening process at 20°C. This comparison is carried out by calculating the maturity age, M, which is the equivalent curing age at 20°C. The maturity age is given by:

$$M = \int_0^t H(T) \cdot d\tau \quad (2)$$

Electronic instruments are available to automatically evaluate Eq(2).

The Heat Generation In Concrete:

A central problem in the description of the heat balance in hardening concrete structures is the mathematical modelling of the heat generating properties of concrete under varying temperature conditions. As the heat generation is affected by such factors as chemical and mineral additives and by w/c, key data for the mathematical description must be determined as far as possible by measurements of the concrete in question.

The heat generated by a given concrete is most expediently determined by adiabatic calorimetry.

This measurement records the temperature increase in a concrete sample during hardening without heat transmission to the surroundings. With known concrete composition and known specific heat of concrete, the measured temperature increase can be transformed to a specific heat generation: kJ per kg cement.

By use of the temperature function, the measured heat generation can be established as a function of the maturity age, M . The curve thus obtained is used for calculations of both temperature and maturity age in the structure (2), (3), (4).

If the specific heat generation is plotted as a function of the logarithm maturity age M , the S-shaped curve in figure 3 is obtained.

The Heat Balance In Hardening Concrete Cross Sections:

In the center of massive concrete structures and in strongly insulated structures, the heat transmission of the concrete to the surroundings is insignificant in relation to the heat generated during the early curing. In this period, the temperature increase of the concrete is, therefore, approximately proportional to the adiabatic heat generation.

For slender structures, the loss of heat to the surroundings will often dominate in relation to the heat generated. In order to obtain a reasonable increase of temperature, and thereby a suitably rapid strength development, it is necessary to take controlling measures to counteract the natural cooling of the concrete.

The heat balance to be managed is sensitive to changes in the insulation level selected. This is due to the following interaction:

- The rate of heat generation is reduced, if the concrete temperature is decreased.
- The heat balance will often shift to a state dominated by heat loss to the surroundings, if the rate of heat generation of the concrete is reduced, and thus the temperature drop increases, and vice versa.

This "instability" of the heat balance occurs because the heat transmission is proportional to the temperature difference between concrete and surrounding air, whereas the heat generation changes in accordance with the temperature function. If, for instance, the concrete temperature is 5°C and the air temperature -10°C , a change of 1 degree of the concrete temperature will alter the rate of heat generation by about 11%, whereas the heat transmission will be changed by about 6% only. In principle, a mathematical treatment of this heat balance problem is practicable by means of several numerical methods (5). The extent of the necessary calculations and the possibility of plotting the results in diagrams, will largely depend on the assumed boundary conditions.

Temperature Strain On The Hardening Concrete:

On the basis of present knowledge from completed construction projects, it is not possible to fix the exact limits of temperature differences acceptable in hardening cross sections. Nevertheless, efforts should be made to keep the following limits of temperature differentials:

On cross sections, a maximum difference about 20°C; and on casting joints and in structures with highly variable cross section dimensions, a maximum difference 10 to 20°C.

In the latter case a higher temperature difference than the one stated may be allowed, as the effects of relaxation in the hardening concrete are capable of reducing the effects of temperature induced strain. Consequently, to avoid cracks caused by temperature strains, it is necessary to plan and monitor the curing process.

As discussed above, it may be necessary at short notice to make decisions on removal of form work and or on adding supplementary insulation to previously cast cross sections. Such arrangements must be pre-planned so they can be decided upon on building sites.

In project specifications there will often be requirements on the maximum difference between the temperature measured in the interior of the cross section, T_m , and the one measured at the surfaces, T_r , during the cooling of the concrete. If insulated form work is removed from a hot concrete cross section, an increased temperature difference between interior and surfaces is created. The maximum value of this difference will mainly be determined by the relation between the heat conduction resistance in the concrete cross section and the heat conduction resistance at the boundary. This relationship is established by the dimensionless parameter known as the Biot-number Bi , which is defined by:

$$Bi = \frac{k \cdot \delta}{\lambda} \quad (3)$$

where

- k = surface transmission, $\text{kJ/m}^2 \cdot \text{h} \cdot ^\circ\text{C}$
- δ = characteristic dimension, m
- λ = thermal conductivity of concrete, $\text{kJ/m}^2 \cdot \text{h} \cdot ^\circ\text{C}$

If the Biot-number of the system is low during the cooling process, the interior resistance to heat flow is low in relation to the resistance at the surface. Consequently, the temperature decrease of the concrete, which is proportional to the resistance, will be relatively small.

Conversely, a large Biot-number will indicate that the interior resistance is large in relation to the surface resistance. This means that the temperature decrease at the concrete surface will be large in relation to interior temperature decrease. In figure 4 this effect is expressed graphically. Figure 4 shows the fully developed (peak) temperature distribution as a function of the Biot-number.

Site Control

Knowledge of the temperature development and the temperature distribution in the hardening concrete will enable the contractor to control the hardening process of the concrete with satisfactory quality assurance.

On the basis of temperature measurements, it is possible to make decisions on form work removal, tensioning of cables, supplementary insulation, supplementary curing, etc., and thus ensure that the requirements of the work specifications are met in an economic way.

In practice, it has proved suitable to measure the temperature of the concrete by means of copper/constantan thermo-couples appropriately placed before start of the operations.

The temperature can be recorded, either manually by means of a digital thermometer or automatically by means of a data logger which can transmit data directly to a computer for mathematical treatment and graphical display.

It will always be possible to follow the temperature and maturity development in a number of selected areas in a concrete structure, with the purpose of either proceeding with construction activities when the required maturity has been obtained, or changing the temperature development by changing the degree of insulation.

To some extent, the treatment of the measurement results depends on the purpose of the measurement. In general, it will be a matter of a current control in the curing phase, comprising for instance:

- safe-guarding against early freezing
- compliance with the time of form work removal
- control of maximum temperature
- control of temperature stresses
- choice of time for tensioning of cables
- selection of supplementary insulation
- compliance with the required curing period

Consequently, it is an expedient measure to evaluate the results after each reading, and, if necessary, to adjust the conditions for the future progress of the curing process of the concrete.

Results

The application of the curing technology program meant a further development of the use of the maturity principle and of measures adopted more sporadically at previous major bridge constructions in Denmark. As a result the concrete control process measured and evaluated the actual quality development of the concrete under the curing conditions in the structures themselves.

The supplementary, conventional concrete control by measuring strength of test specimens functioned as reference tests and as control of cement quality, w/c, etc. Supplemented with measurement of the heat development of the concrete by means of adiabatic calorimetry, the strength development of the test cylinders during trial mixing was also used to control the maturity calculations (the maturity functions)

The curing monitoring procedure was superior to conventional control methods, because it was possible to calculate in advance (and follow up by way of the temperature measurements in the concrete):

- * the most appropriate planning of mass pours,
- * time of form work removal,
- * time of maintaining supplementary insulation, if any, as a function of weather conditions, etc.

The time requirements created by this method of planning could be integrated rationally in the working operations. A considerable degree of repetition in the casting process including rationalizing the control function, made the planning expedient.

With the use of the special cement, a high cement content, fly ash and chemical admixtures, the curing technology was also utilized for a better indication of the early development of the concrete properties than practicable by conventional testing. Consequently, it became an instrument for making desirable changes of the concrete composition or construction process during the actual concreting operations. With one exception, occurring at an early date of the work, when leakage in a piece of underwater formwork caused water penetration and disturbance of the calculated temperature development, the process monitoring functioned with such effectiveness that the curing process could be accepted in all control sections. The water penetration incident was detected as a consequence of the temperature measurements, which meant that the process control functioned effectively also with regard to detecting hidden defects that would not have been disclosed by conventional controls.

Altogether, the applied curing technology provided the following experiences:

- Maximum permissible temperatures during the curing can be used as a process governing factor for massive underwater pours.
- Maximum permissible temperature differences can be used as a process governing factor for concreting massive structures on previously hardened concrete.
- Concrete impermeability and uniform appearance of the finished surfaces can be assured by calculating the needed supplementary insulation periods after formwork removal. This experience contains promising aspects to further development of the curing monitoring system, in order to replace the classical method of sprinkling, which is often not practicable, or the use of curing compounds, which are not easy to control, or the specifying of larger reinforcement cover, which has no protecting effect unless adequate maturity of the concrete has been ensured.

On the whole, the trial testing and curing design procedure enabled the contractor and management to become thoroughly acquainted with the prospects of choosing the qualities of concrete most suitable for meeting the requirements, before the production started. These gave them a solid foundation for estimating the consequences of subsequent deviations and uncertainties.

Conclusions

The completion of the project proves that it is possible to obtain crackfree concrete structures in spite of many practical difficulties and to obtain these successful results within reasonable costs. Exact calculations have not been made of the costs incurred by applying the curing technology on the Faroe bridges site, partly because the measures taken by the contractor during the execution of the works were similar to what are regarded as normal winter measures in Denmark. However, the contractor has estimated that the contract costs are about 1 to 2% higher than if the job had been performed on the basis of more conventional practice. This increase also includes measures to prevent alkali-silica reactions, use of special cement, and other special requirements.

The main reason for the success was the contractor's thorough and detailed planning of the curing processes from the time the concrete was cast until the time when the temperature differences in the structures were negligible. This procedure introduced new methods and a different way of thinking with respect to the execution of concrete works for all involved.

The contractor managed to gain so much control of the concrete quality, that the registered variations in the heat development and strength properties were documented to result from the variations in the cement only.

With the hindsight gained from employing the described requirements, we feel the need of change on the following points:

- In order to gain full control of the temperature development, specific requirements for the heat generation of the cement supplied - and especially the variations - must be established and enforced.
- The requirements in the specifications which limit temperature differences are substitute requirements for restricting the developing temperature strains and, as such, have limited validity. Further research into the temperature strain/concrete tensile strength relationship in concrete sections cast together is needed. The establishment of temperature gradient requirements in relation to the relative difference of section dimensions could prove important in securing the required results within acceptable costs.

Acknowledgements

The present paper is based upon an extensive book prepared by The Danish Ministry of Transport, The Road Directorate:

>> THE CONCRETE OF THE FAROE BRIDGES <<.

The prime authors of this book are Mr. F. Meyer of Christiani & Nielsen A/S and Dr. K.R. Henriksen of the Farøekonsortiet. The considerations concerning the impact of heat on the concrete during the curing are based upon Appendix 2 of the book, prepared by Messrs. P. Freiesleben Hansen and E.J. Pedersen of the Concrete and Structural Research Institute (BKI), Denmark. The work of these authors is gratefully acknowledged.

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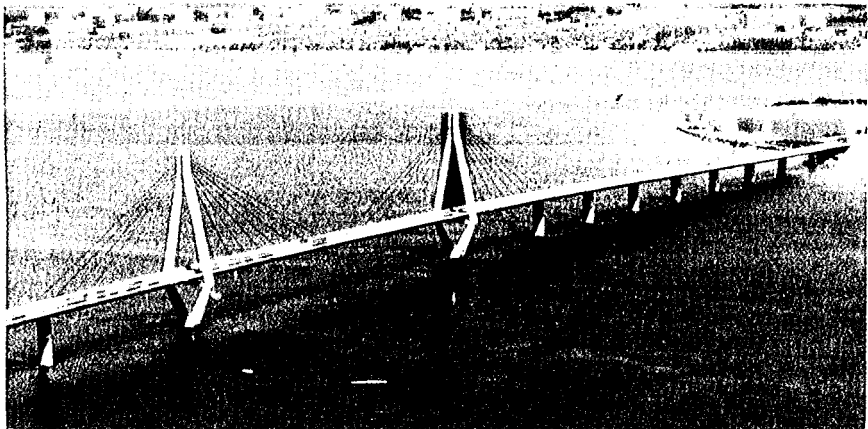


Fig. 1--The South Bridge between Faroe and Falster

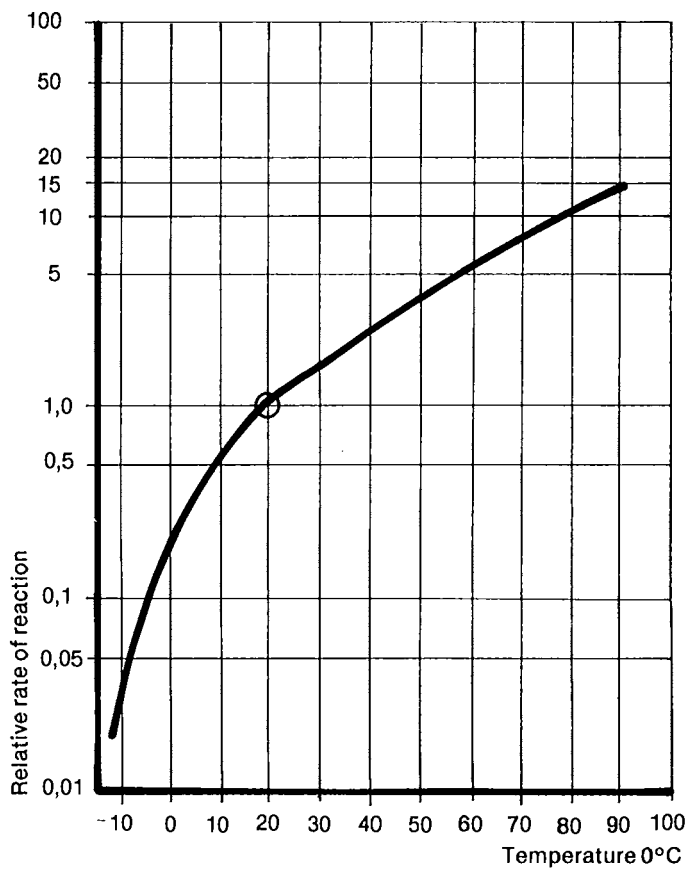


Fig. 2--Effect of temperature on the relative hardening rate

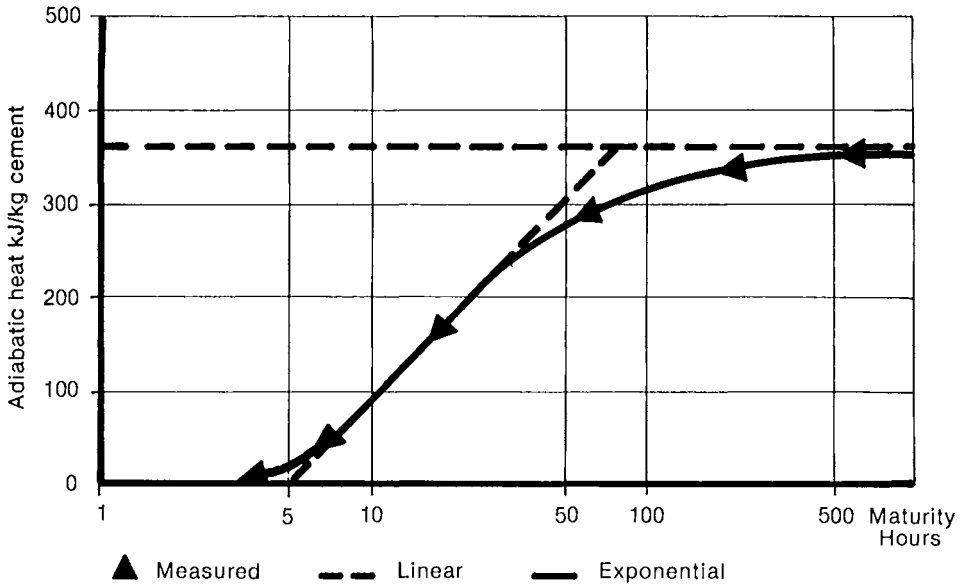


Fig. 3--Generated heat as function of logarithm of maturity

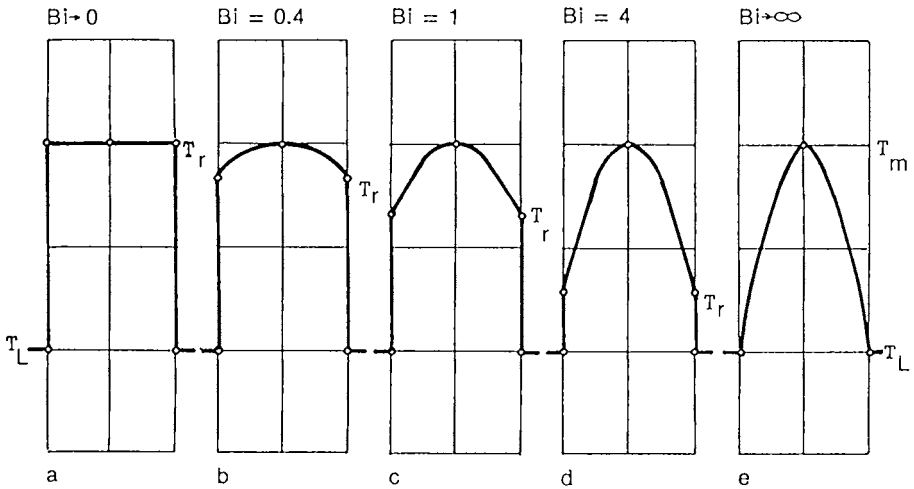


Fig. 4--Temperature distribution as a function of the Biot-number

Temperature-Related Mechanical Effects in Concrete Elements and Optimization of the Manufacturing Process

by P. Acker, C. Foucrier, and Y. Malier

Synopsis: The BITEX computer program was developed to predict the temperature distribution in a concrete structural element during manufacture and to evaluate the resulting stresses. This program takes account of several types of boundary conditions : variable ambient temperatures, and various kinds of formwork and heat treatment.

A simple and inexpensive specific test has been developed to determine the heat of hydration of the cement, the kinetics of which are a function of position. The test consists of recording the temperature during setting of a concrete sample subjected to adiabatic conditions.

These tools have recently been applied to actual cases (bridges, tunnels, pressure vessels, etc.) and have demonstrated the variety of their applications :

- optimization of the temperature cycle, making possible a significant reduction in fuel consumption ;
- better distribution of heating resistors to avoid cracks completely ;
- selection of the type of cement based on quantitative analysis
- a judicious staging of the casting of large structures.

Keywords: computer programs; concretes; heat of hydration; optimization; precast concrete; prestressed concrete; reinforced concrete; setting (hardening); temperature; tests; thermal stresses.

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His works concern constitutive laws and mechanical behavior of concrete, with a particular attention on physical phenomenon. He developed models for the evaluation of losses of prestress, taking into account the thermal treatment of concrete, of temperature-related mechanical effects during the manufacturing process, and of the structural effects due to the drying kinetics.

INTRODUCTION

Grave disorders, sometimes even fractures, that have appeared in concrete elements during or shortly after fabrication have been attributed to stresses resulting from non-uniform temperature fields (1).

These effects may be extensive, especially in the following two cases :

- in thick parts, because of the heat given off by the setting of the cement ;
- in parts of non-uniform thickness (flanged beams, precast segments, etc.) subjected to a heat treatment (2).

To solve these problems, two complementary tools have been developed (3) :

- a test on fresh concrete that yields the curve of heat development under adiabatic conditions ;
- a temperature computing program that takes into account both diffusion of heat, with or without heat treatment, and the internal source of heat, using the curve obtained by the adiabatic test.

Our aim is to present these two tools and to show, on the basis of recent applications, the variety of services they can render (4).

DISCUSSION

Analysis of the thermal phenomena

The temperature fields that develop in concrete parts depend on a large number of parameters, in particular :

- the thermal characteristics of the concrete (diffusivity and specific heat)
- the heat of hydration of the cement, which acts as an internal heat source
- the geometry of the part, in particular its cross section
- the initial temperature of the concrete at the time of placement
- the boundary conditions - the external temperatures (which may result from a heat treatment cycle) imposed at the surface either of the concrete or of interfaces (formwork, insulating tarpaulins), which are then specified by a heat exchange coefficient (5).

The mechanical effects of these temperature fields, for their part, involve :

- the coefficient of expansion of the concrete
- its modulus and its Poisson's ratio
- its tensile strength.

All of these characteristics depend on the maturity of the concrete, i.e., on its age and on the temperatures to which it has been exposed since the start of setting (6).

The importance of the geometry of the parts in these phenomena and the diversity of heat treatments used has naturally led to the use of a finite-element computer program to solve the heat equation.

The simulation program

The BITEX program was developed from an existing program, BIDIF, that makes possible the non-linear transient calculation of heat in plane (or axisymmetric) diffusion and offers great flexibility and a large number of possibilities for the boundary conditions (7).

The main difficulty was introducing the heat of hydration into the calculation : at a given instant, the rates of development of this heat will be different at all points in the part, faster at the hottest points and slower at the coldest.

Recent work has shown that this phenomenon is governed by Arrhenius' law (8).

Thus, at a given instant, the rate of heat development, dQ/dt , depends only on the total quantity of heat Q already developed and the current temperature T . Moreover, the temperature-dependence can be expressed in a very simple and classic manner by an exponential function :

$$dQ/dt = f(Q) \cdot \exp (-K/T) \quad (\text{where } K \text{ is a constant}).$$

The very interesting consequence of this is that it is sufficient to carry out a single test and record the temperature $T(t)$ and the heat developed $Q(t)$ for the whole duration of the test to be able to predict the rate of heat development dQ/dt under different conditions.

Better still : such a test calls for an homogeneous test specimen and a uniform temperature field, and the best way of obtaining this is to carry out an adiabatic or quasi-adiabatic test. And, assuming that the specific heat C of the concrete does not vary with time, it is then possible to determine the quantity of heat developed $Q(t)$ from the recording of temperature $T(t)$ alone.

It is then sufficient to say that, in each element, and during each time interval $(t, t+dt)$, the variation of the mean temperature of the element in question is the algebraic sum of the variation of internal origin, calculated as indicated above, and exchanges with neighbouring elements resulting from diffusion. If the sum of the first terms is stored (this sum represents the cumulative exothermal development up to time t), it is then possible to calculate the temperature increment of internal origin in the next time interval by the following equation

$$\Delta T_{n+1} = \frac{\exp(-K/T_n)}{\exp(-K/T_a)} \cdot [T_a(t^*+t) - T_a(t^*)]$$

in which t^* , the equivalent time, is obtained by solving the following equation :

$$Q_a(t^*) = C \cdot T_a(t^*) = \sum C \cdot \Delta T_i$$

This method of calculation is no more than a generalization of the one given in the new French regulations for the design of prestressed concrete structures (B.P.F.L.), to take into account the influence of the heat treatment on the evolution of the properties of the concrete (9).

The modification of the BIDIF program therefore consisted essentially of three points (10) :

- processing the results of an adiabatic test-reading of data ($T_a(t)$ curve) in the form of a temperature vs. time table and an interpolation function ;
- adding a subroutine to solve the equation giving the equivalent time and calculate the temperature increment of internal origin at each point ;
- constituting a table containing, for each point of the lattice, the cumulative heat evolution incremented at each time interval.

To obtain the master curve, $T_a(t)$, we were led to develop a "quasi-adiabatic" test making it possible to record the heat of hydration developed by the setting of the cement in the concrete ("Q.A.B." test, figure 1).

The prototype developed includes a microprocessor that prints directly the table of values to be entered as program data. Each instrument must be calibrated, and the calibration constants, defined like those of Langavant apparatus used for cements, are entered with the calculation data (11).

The boundary conditions were investigated using a device described in figure 2.

In this way, it was possible to establish a table giving, for the various types of boundary conditions (wooden or metal formwork, insulating tarp, exposed, ventilated, or sheltered surface, etc.), the values of a heat exchange coefficient (table 3).

The program makes it possible to print out the results in the form of tables (temperatures at the nodes of the lattice, time interval by time interval), together with a drawing of the cross-section, with isothermals for selected time intervals (figure 5) and temperature vs. time curves ("profiles", figure 6) for selected special points. The data are entered using a conversational pre-

processor that automatically checks the consistency of the set of data.

APPLICATIONS

The methodology based on the complementary use of the program and the quasi-adiabatic test of the concrete has been operational for nearly two years, and we have been able to apply it to a dozen cases. We have selected four of these applications we consider representative of the various services these tools can render, in particular, in the examples we shall be considering :

- minimizing a curing cycle
- optimizing the positions of heat sources
- localizing cracking risks
- choosing the staging of concrete placement.

First example : a conventional segment

This was for a motorway structure in the Paris area, using cured precast segments (figure 4).

The temperature fields developed in them by the curing cycle originally planned were calculated ; this revealed large temperature differences between the core of the web/top slab join and the middle parts of the flanges (figure 5).

Moreover, the first laboratory tests revealed inadequate early strength. The contractor therefore had two aims : to increase the strength while at the same time diminishing the "thermal gradients".

A calculation with a more energetic cycle (figure 6) showed an aggravation of these differences : it was substantially more effective in the thin zones, already hotter than the core of the thick zones. This result, and in particular the temperature curve recorded during the Q.A.B. test, compared to the curing cycles (figure 7), demonstrated the incompatibility of these two objectives : with the cement originally planned (with a low heat of hydration), it was clear that it was impossible to "heat more" without increasing the risk of cracking, and in consequence that there was no hope of increasing the strength values with this cement.

It was accordingly decided to change cement, in this particular case, to a cement with a high heat of hydration. It was then an easy matter to choose a cycle (a moderate one not exceeding 40°C), using the results of the Q.A.B. test. Another calculation confirmed a substantial diminution, from 30 to 5°C , of the maximum temperature differences.

Optimization of the cycle is not an abstract idea. The procedure is as follows : start from the adiabatic curve obtained from the Q.A.B. test, corrected using its calibration coefficients. If we impose, in the enclosure, a temperature that exactly follows this

curve, it is clear that at any time the temperature will be uniform throughout the part.

It must be clearly understood that, except in the case of small parts (floor joists, precast slabs, pipes), heating hotter or faster than this ideal cycle can only increase the temperature differences in the part.

Since it is necessary to cool, what is imposed is in fact a growing difference between the temperature at the core (where the heat sensor is located) and the temperature of the enclosure (controlled by the boiler), a difference that must follow a curve obtained by an affinity from the evolution curve of the concrete tensile strength or, more simply, remain proportional to a concrete "maturity factor", defined analytically from the temperature alone (in passing, in the case of pretensioned elements, this constitutes a datum usable as the "pretensioning signal").

It should be noted that this is also an economic optimum. In the case of a perfectly insulated enclosure, with the ideal cycle, the heat input serves only to heat the air in the enclosure ; in practice, then, it serves only to compensate heat losses from the enclosure, and the energy outlay depends only on the quality of the insulation.

Second example : a segment with three thick webs

The next example concerns a structure on the same motorway, with segments having three webs (figure 8) heat-treated by means of electrical resistors placed on formwork.

The first segments made had visible cracks. Calculation results revealed large differences between the temperatures of the bottom slab, the web, and the top slab (figure 9). In particular, a cold core was found at the top of the web. This is explained quite simply by the placement of the resistors : there were twice as many per unit volume of concrete on the bottom slab, 22-cm thick, as on the web, 45-cm thick, and only half as many on the top slab, heated on only one side. Moving some of the resistors from the bottom slab to the top eliminated all apparent cracking.

Examination of the stresses induced by such a temperature field (figure 10) shows that, in this type of part, the greatest tensile stresses are not necessarily found at the coldest points, nor at the points of steepest temperature gradient (12).

We should add that, on this segment, as on the previous one, heat sensors placed in the concrete confirmed, to within a few degrees, the maximum temperatures (or, more precisely, the difference between the maximum temperatures and the initial temperatures, which cannot always be predicted) obtained by the calculation ; (on the other hand, the time at which the maximum is reached may be slightly off, probably because of inaccuracies concerning the

thermal properties of the concrete) (3).

Example 3 : a thick-walled cylindrical tank

This was a cylindrical concrete tank poured in horizontal lifts 2.50-m high. The concrete is 120-cm thick. In the case of this structure, we received an enquiry concerning the origin of the widely spaced and very open vertical cracks that were appearing a little above the junctions between successive placements.

Here again, the calculations were able to explain this type of cracking (figure 11).

When bank n is poured on lift $n+1$, which is 15 days old, the latter is not yet cold ; its temperature (determined by a preliminary calculation assuming a uniform initial temperature of 20°C) is between 30 and 45°C .

In a first stage, this support diffuses heat to the fresh concrete, causing setting to start earlier at the bottom than in the rest of the bank, especially in the core.

Towards $30 - 35^{\circ}\text{C}$, the fluxes are reversed, now going from the lift in progress towards the thermally inert concrete of the support. This is why, after 10 hours, well into the hardening period, the hottest point is about 50-cm above the stop end. During cooling, it is the zones that set at the highest temperatures that undergo the most thermal shrinkage. According to this, the most heavily stressed zones are in the core ; but this does not mean that the cracks observed are through cracks, because the effects of drying shrinkage (considerable with a hot concrete), which affects only a small depth of concrete, must be superimposed on the effects of thermal origin.

Having explained the cause of these cracks, it is obvious that the calculation program may be used to test and quantify the relative effectiveness of various possible preventive measures (making the wait between two lifts longer or shorter, choosing a cement that is more - or less - exothermal, insulated formwork, initial heating of the concrete, etc), while taking their respective costs into account.

Example 4 : a large caisson

The last example concerns a motorway structure (going under a river) made of prestressed caissons measuring $12 \times 25 \times 50$ metres. Given the size of the part and the thickness of the floor (130-cm) and walls (90-cm), it was possible to anticipate a high risk of cracking, especially near joints between successive placements. The extent and timing of the various stages of concrete placement could be expected to play a major role.

The calculation, by indicating in particular the average rate of cooling of each part, made it possible to quantify the mechanical

effects of restrained shrinkages and so choose the best staging.

A finite-element calculation of the floor, into which were introduced initial stresses proportional to the calculated temperatures (figure 12), made it possible to identify the zones of greatest stress with a view to working out a judicious arrangement of the strain gauges and optical fibres to be used for crack detection.

These few examples of applications illustrate the diversity of services this method can render. It should be noted that, in some cases, a mere examination of the results of the Q.A.B. test may provide enough information to guide a decision concerning :

- the specification of a curing cycle suited to a particular concrete ;
- the choice of a cement, in the two extreme cases (massive part calling for low heat of hydration ; thin parts that may benefit from a high heat of hydration to attain high strength values rapidly).

CONCLUSION

It is today possible to predict the temperatures in concrete elements during their fabrication using simple and rapid tools. The heat generated by the setting of the cement must be taken into account in this prediction. This heat can be calculated using a curve obtained from a specimen of the concrete placed in quasi-adiabatic conditions.

In many cases, observed disorders can be explained by a simple examination of the temperature distributions in the cross-sections and of their evolution during setting. In more complicated cases, various scenarios may be simulated by calculation so as to determine which technological solution is most cost-effective.

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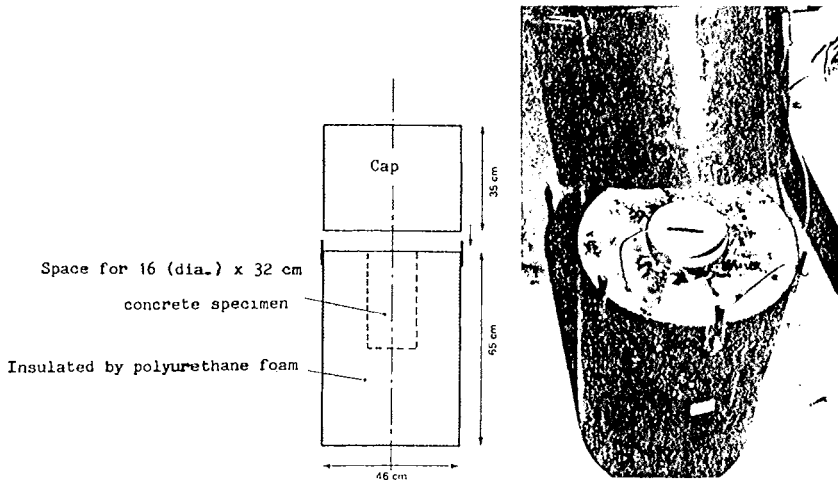


Figure 1. Diagram and photograph of the prototype designed to hold a standardized specimen (dia. 16 x 32-cm) in a cardboard mould. The temperatures are measured at the centre of the specimen by electronic sensors connected to a microcomputer. The enclosure is made from a metal pipe 50-cm in diameter. The coefficient of thermal conductivity of the polyurethane foam is $0.023 \text{ W.m}^{-1}.\text{°C}^{-1}$. A complete system including a recording device, software, and printer has since been developed and is now being sold.

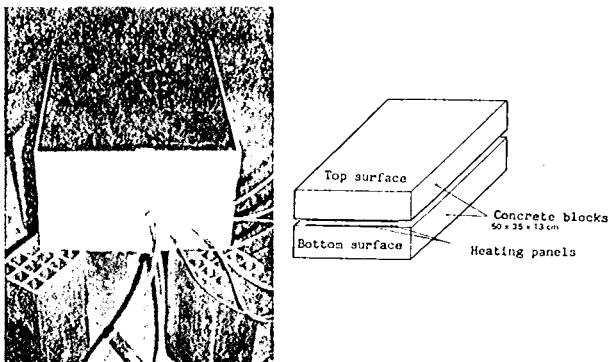


Figure 2. Diagram and photograph of the experimental device used to determine the global coefficient of surface heat transfer (Newton condition). Radiant panels having a high heat emission coefficient were chosen. The sides are insulated with 8 cm of polystyrene foam. The heat spreads well and, at thermal equilibrium, the distribution determined by temperature gages in each block is linear.

Table 3. A few transfer coefficient values determined using the device of figure 2. The second column gives these coefficients in the units specific to the BITEK program : hour, centimetre, and degree C :

Type of surface units	$W.m^{-2} \cdot ^\circ C^{-1}$	BITEK
Exposed surface (in air) :		
- ventilated	6.00	2.16
- not ventilated	4.00	1.44
Metal formwork (2-mm sheet):		
- ventilated	5.45	1.96
- not ventilated	4.00	1.44
Wooden formwork (20-mm plywood):		
- ventilated	3.10	1.12
- not ventilated	2.55	0.92
Insulated tarp	0.50	0.18
Axis of symmetry	0.	**

** in the preparation of the data set, the axis of symmetry is simply not included in the exchange elements (boundary conditions)

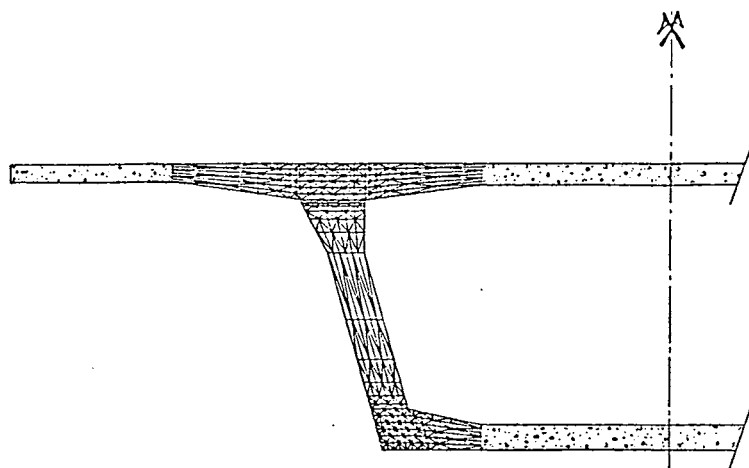


Figure 4. Cross-section of the segment of the first example (total width of segment : 16.70 m). Only the web, 45 cm thick, and the gussets (the portion represented here by its lattice, consisting of triangular elements with three nodes) were modelled. With a null exchange condition at the ends of the truncated gussets, it is perfectly possible to extrapolate the calculated temperature distributions to the slabs.

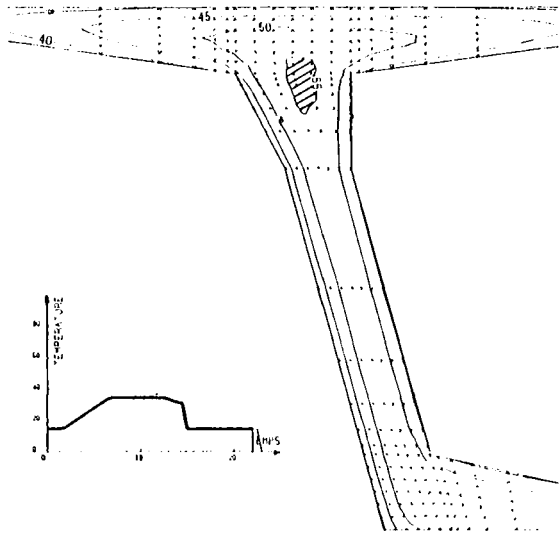


Figure 5. Temperature field calculated in the segment of example 1 (figure 4), represented by the isotherms at 22 hours (cycle at 35°C, form removal at 14 hours). The temperature of the hatch-marked zone at the top of the web is between 55 and 60°C.

PLOTS OF TEMPERATURE VS. TIME PROFILES
DARSE STRUCTURE
TOP GUSSET

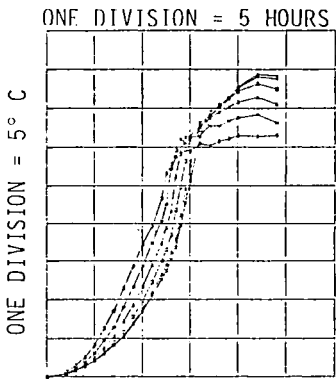
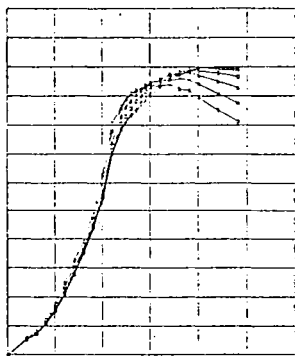


Figure 6. Evolution of the temperatures at various points of the segment of example 1, with the proposed cycle to the left (see figure 7) and a faster cycle to the right. It can be seen that the main effect is an increase in the temperature differences, from the very beginning of heating, as the price of a rather small increase in mean temperature.

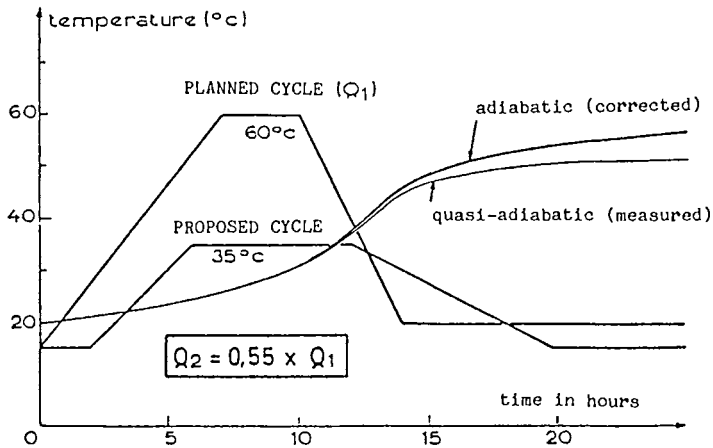


Figure 7. An examination of the planned cycle (at 60°C) and the proposed cycle (at 35°C) in example 1, compared to the result of the Q.A.B. test, makes it possible to understand why the former is ineffective, as revealed by the "profiles" of figure 6. Powerful cycles are useless with a cement having a low heat of hydration because the core of the thick zones remains unaffected by the heat treatment and follows the temperature given by the Q.A.B. test. The ratio of the areas of the two cycles is 0.55 ; this gives an idea of the energy savings achieved.

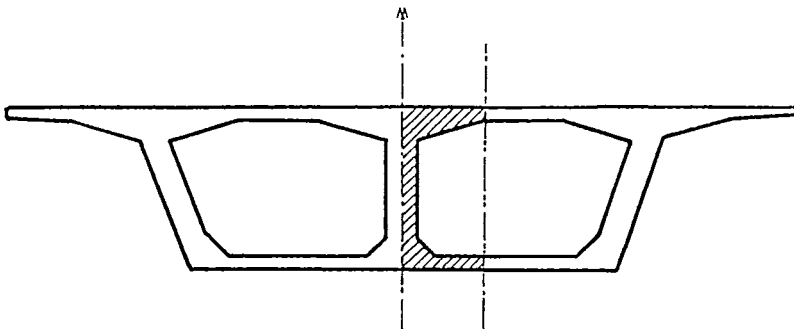


Figure 8. The three-web segment of example 2. The webs are of equal thickness (45-cm) and the gussets are identical. Taking advantage of these symmetries, we modelled only the hatch-marked portion ; with null exchange conditions on the vertical surfaces (cutaways) of this lattice, the results obtained constitute a complete image of the whole temperature field.

Figure 9. Distribution of calculated temperatures in the segment of example 2. Isotherms at 12 hours with the cement and cycle (at 60°C) initially planned. The temperatures of the three zones (bottom slab, web, and top flange) differ by from 10 to 20°C ; moreover, a cold core is clearly visible at the top of the web. These results made it possible to propose a significant improvement based on a judicious distribution of the heating resistors attached to the formwork. The zones of high temperature gradient are those where the isotherms are closest together.

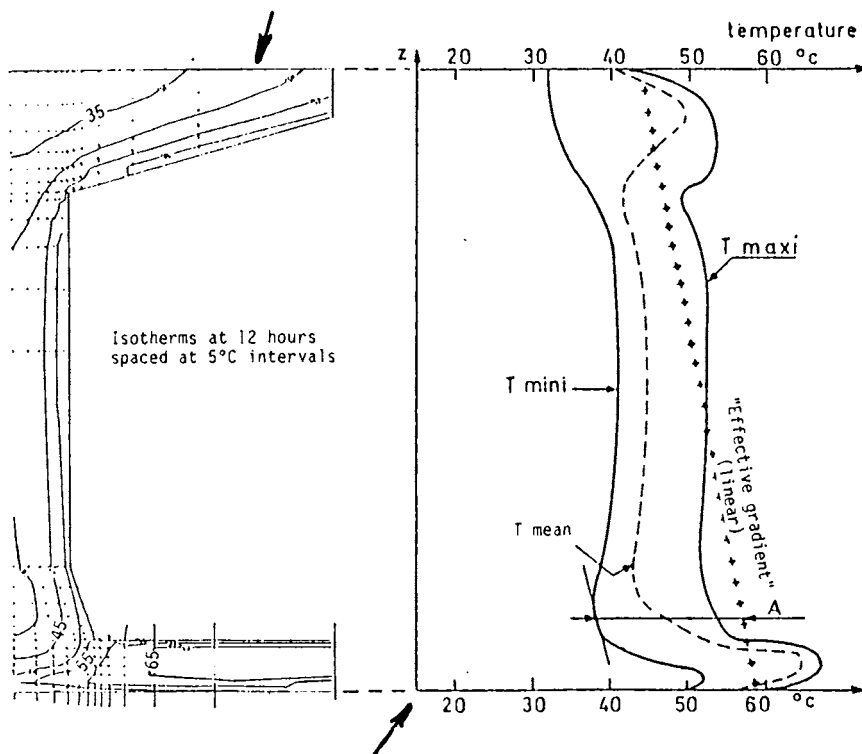


Figure 10. The distribution of temperature makes it possible to estimate the induced mechanical effects according to the conditions of support of the part. The distribution of figure 9 is represented here by the horizontal means of temperature versus height of section. It can be shown (12) that this distribution is the sum of a linear distribution (the effective gradient, shown schematically here, with allowance for the relative inertias of the slab and web) that, in the isostatic case, produces overall flexion but no stresses (unless any self weight is counted) and a non-uniform distribution (the sum and moment of which are zero) that produces only stresses. It can be seen here that the tensile stresses of thermal origin are greatest in the core of the bottom gusset of the web (where the gradient is zero), and are of the order of 4 MPa (point A).

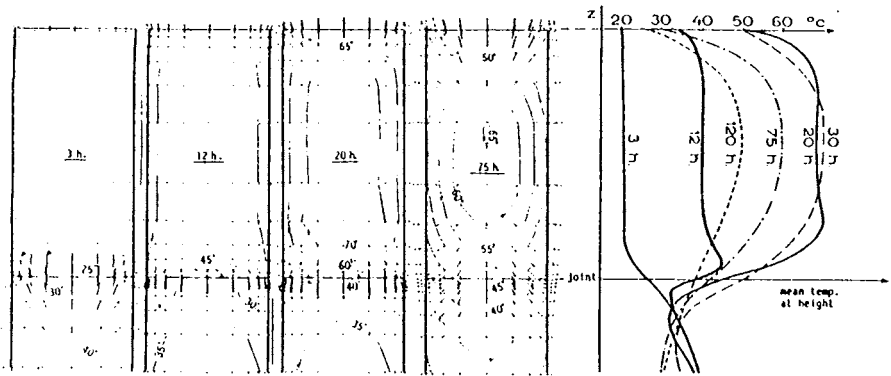


Figure 11 . Evolution of temperatures in the wall of example 3, shown to the left by the isotherms of a right cross-section, at different ages, and to the right by their vertical distributions (means across the thickness). A first calculation, assuming uniform initial temperatures, indicated a field of residual temperatures between 30 and 40°C after two weeks in the bank acting as support. When these initial temperatures are taken into account, there is found to be a hotter core located just above the joint between successive placements. This is, in general, the place where, in this type of structure, the (vertical) cracks are the widest open.

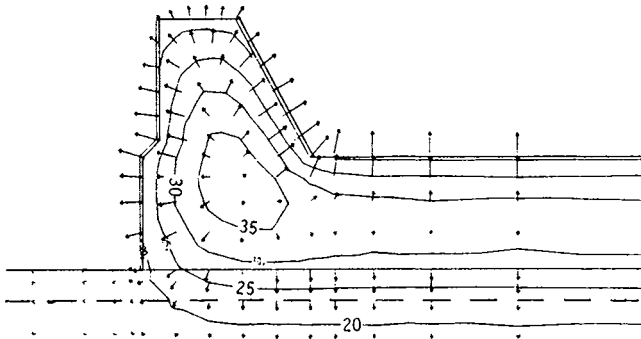


Figure 12. Distribution of calculated temperatures at 7 days in the floor (130-cm thick) of the caisson of example 4. The temperatures are moderate, because a cement with a low heat of hydration had been recommended (within the range of the rather high mechanical properties required, cements having heats of hydration differing by a factor of two can be found in the Paris area alone). Considerable diffusion into the foundation (porous concrete) and the ground, both thermally inert, can be seen. Such an asymmetrical field induces (restrained flexion) tensile stresses of the order of 3-MPa in the top fibre. A simplistic calculation with an effective gradient of 10°C over the height shows in fact that the moment resulting from the self weight would be three times as large.

Thermal and Thermomechanical Analysis of a Cylindrical Cementitious Plug Hydrating in a Borehole

by C. Gotsis, D.M. Roy, P.H. Licastro, and S. Kaushal

Synopsis: Thermal analysis was made of the effects of hydration of cylindrical configuration specimens of a slightly expansive cementitious mixture. This simulated a cylindrical borehole plug, and with modification, may be applied to other cylindrical configurations. Inputs were made to the computer program ATHENAN developed to assess the thermal history of cylindrical domains with symmetrical thermal loads. Inputs to the program were the experimental isothermal rates of heat-evolution of the cementitious mixture measured at several temperatures. Using the temperature history from ATHENAN and experimental data from the volume change and mechanical properties of the cementitious mixture in the computer program SAPIV, a stress analysis was performed, which showed that tensile stresses at the interface may arise at the early stages of hydration when the cementitious mixture tends to shrink, while small compressive stresses are present at long times in such materials when the cementitious mixture tends to expand.

Keywords: cements; compressive strength; expansion; heat of hydration; hydration; models; shear stress; shrinkage; stresses; temperature; tensile strength

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INTRODUCTION

The control of peak temperatures during concrete curing has been shown to be important in preventing thermal cracking due to excessive heating on temperature differentials between the interior and exterior of concrete bodies (1). The prediction of heat evolution in field-placed cementitious mixtures is important for assessing the possibility of undesirable stress development. Alternative means of generating the laboratory data for use in prediction of such heat development include isothermal calorimetric measurements (2) of representative small samples of the mixture at several temperatures, or adiabatic measurements of a simulated specimen (3). Recent effort has been directed to developing a computer program for integrating multiple isothermal measurements (along with thermal conductivity and diffusivity) into a predictive tool (4).

Among recent attempts to predict heat evolution of hydrating cements are those involving the consideration of factors such as particle size (5) and the effects of substitution of secondary cementitious components (2,6,7). Although the hydration process is complex, it is possible to determine the apparent activation energies for hydration (or associated property) if an Arrhenius type behavior is simulated in the material (2,5,6,7,8). These are useful for calculating expected thermal effects.

The current studies were undertaken to examine thermal



effects on cylindrical geometry cementitious bodies, which could consist of either borehole plugs in an earth or rock enclosure, or concrete enclosed by formwork.

The sealing of boreholes by cementitious mixtures for many years has been practiced in engineering applications such as oil or water exploration and mining. Increased interest has been given recently to radioactive waste storage applications, where boreholes which connect the biosphere with a geologic repository for radioactive waste storage would require sealing. These seals may restrict transport to toxic materials, isolate water bearing zones from the repository, and provide structural strength to maintain stability; they should also exhibit compatibility with the host geology and maintain effective longevity for the required period of isolation.

Research on such seals has been conducted; the results of some of these studies are presented. Salt, tuff, basalt and granite are among the repository host rocks, proposed by the U.S. Department of Energy for the disposal of radioactive wastes. An example is given in this study of an experiment in a granite host material.

Several cementitious mixtures were investigated and the mixture with the most promising combination of physical, chemical and mechanical properties was selected for study in a simulated laboratory scale plugging of a drilled granite cylinder, by experimental measurements and computer simulations. The objectives of these studies were to develop a method for deciding whether the selected cementitious mixture could be used satisfactorily for plugging boreholes without developing temperature gradients that could initiate cracking due to thermal and/or chemically generated loads (because of volume changes due to chemical interactions). The method could similarly be used for predicting thermal stresses in concrete columns. The thermal history was calculated by the program ATHENAN (4,9) and using this as input to the computer program SAPIV (10) the stress analysis was made. Because of the cylindrical geometry and the axisymmetric thermal loads only a meridional cross sectional area (obtained by slicing parallel to and through the cylindrical axis) needs to be studied, assuming material homogeneity.

The computer program ATHENAN (4,9) (Axisymmetric Thermal Nonlinear Analysis) was written in Fortran language using the form of equation (1), to calculate the temperature history and temperature contours using Fourier's law (25):

$$\frac{1}{\alpha} \partial T / \partial t = \nabla^2 T + Da/K \quad [1]$$

where the terms are (dimensionless quantities):

$$T = (\theta - \theta_R) / \theta_R$$

dimensionless temperature or unknown
dimensionless temperature at time t
+ Δt

$\alpha = \phi/\phi_R$	dimensionless thermal diffusivity
$\phi = K/\rho C_p$	thermal diffusivity, m^2/s
ϕ_R	convenient reference thermal diffusivity, m^2/s
$Da = (dQ/d\tau)R^2/(\theta_R K_R)$	dimensionless heat generation or Damkohler Group IV number
$dQ/d\tau$	heat generation, W/m^3
R	convenient reference distance, m
θ_R	convenient reference temperature
K_R	convenient reference thermal conductivity, $W/m-K$
$\nabla^2 T$	$= (\partial T/\partial r)/r + \partial^2 T/\partial r^2 + \partial^2 T/\partial z^2$
$r = n/R$	radial dimensionless distance
n	radial direction, m
$K = k/K_R$	dimensionless thermal conductivity
k	thermal conductivity, $W/m-K$
$t = \tau\phi_R/R^2$	dimensionless time or Fourier number
τ	time, s

The model assumes that the densities, heat capacities and the thermal conductivities and diffusivities of the various materials are constant. Angular symmetry ensures that $\partial T/\partial \theta = 0$.

THERMAL ANALYSIS

A cylindrical granite specimen with a height of 30 cm and a diameter 39.37 cm was used. The cylinder had a borehole of 20.32 cm diameter and 18.75 cm height, Fig. 1. The plan was to place a cylindrical plug of the selected cementitious mixture in the granite 'form' and simulate the experimental evolution of temperature versus time with the computer program ATHENAN. The origin of the granite was Cold Springs, Minnesota.

Small cylindrical holes were drilled in order to accommodate thermocouples for temperature measurement. Figure 2 shows a meridional cross-sectional area. The same figure shows the grid of the finite difference computer model ATHENAN overlayed on the cross-sectional area of the specimen. Materials 2 and 3 are the granite and material 1 is the cementitious plug. The nodes of the grid are numbered $K = 1, 2, \dots, 72$. The finite elements, numbered 1 to 52 and the boundary conditions (zero degrees of freedom for

node 12, one degree of freedom for nodes 1 to 11, 24, 40, 56 and 72 and two degrees of freedom for the other nodes) used for the SAPIV program are also shown in Fig. 2. The granite specimen was placed in a curing chamber with 90 percent humidity and left to equilibrate with the chamber at 34 ± 1.5 degrees Celsius. After thermal equilibrium was established, as indicated by temperature recordings, the cementitious mixture was prepared. The components of the cementitious mixture, designated PSU 82-022 (11), are given in Tables 1 and 2. After mixing with a planetary mixer for 30 minutes, it was poured into the granite 'borehole,' consolidated by shaking the mass, and then the top of the specimen was covered to prevent drying, although the humidity of the chamber was 90 percent. The poured cementitious mixture reached a height of 16 cm within the granite borehole.

The rate of heat evolution from hydration of the mixture was measured isothermally in SEEBECK type envelope calorimeters (12). Three samples of the mixture were taken and placed in calorimeters with water-jacket temperatures maintained at 34.5, 50, and 60.5 degrees Celsius, respectively. The heat evolution rates are plotted versus time in Fig. 3(a), while the data expressed as cumulative heat, are given in Fig. 3(b).

The heat transfer number used in the program ATHENAN to simulate the hydration of the cementitious mixture in the granite borehole was estimated for natural convection between the granite and the surrounding air at $4.0 \text{ W/m}^2\text{-K}$ (13).

The thermal properties [conductivity (12) and diffusivity] of samples of the cementitious plug and the granite were experimentally measured for use by the program ATHENAN for simulating the hydrating cementitious plug surrounded by granite; Their values were: cement, conductivity 0.8 W/m-K ; granite, conductivity 2.25 W/m-K ; cement, diffusivity $6 \times 10^{-7} \text{ m}^2/\text{s}$, and granite, diffusivity $18.3 \times 10^{-7} \text{ m}^2/\text{s}$.

The simulated temperature data (by ATHENAN code) were in good agreement with experimental measurements by thermocouples (4,9). The thermal history as calculated by ATHENAN was then used for the stress analysis.

The temperature contours (isotherms) at different times of hydration are graphically displayed in Fig. 4. The maximum temperature, reached after 6.2 hours at the center of the cementitious plug was 49.76°C [Fig. 4(b)]. After 2 days, the temperature was almost uniform at 34°C .

STRESS ANALYSIS

Having obtained the temperature distribution, stress analysis is performed using SAPIV (10), a structural analysis program for static and dynamic response of linear elastic systems. Because the problem is axisymmetric, the geometry is modeled using the Type 4 element from SAPIV element library. Type 4 element is based on an isoparametric formulation permitting specification of