RISKS OF THERMAL CRACKS IN BRIDGE COLUMNS DURING HYDRATION

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ABSTRACT

During the last decade research on thermal stresses due to hydration has been performed at Luleå University of Technology.

In laboratory tests, thermal properties, mechanical properties and risks of thermal cracking in small concrete specimens have been studied. Theoretical models have been developed and calibrated to the tests and have been implemented in two computer programs for structural analyses, HETT and TEMPSTRE-N.

Thus, it is now possible to compute thermal cracking risks due to hydration in newly cast elements considering the properties of the young concrete. Also the external restraint from adjoining structures and the restraint inside the element can be considered in the analysis.

In the paper massive columns of the Igelsta bridge in Södertälje, south of Stockholm are studied. Computations showed very high early-age tensile stresses and cracking risks at the surface. Ocularly, thermal cracks were also observed when the form was stripped from the sections first cast.

Theoretical studies with HETT and TEMPSTRE-N showed that the most effective way of avoiding cracks was in this case to cool the concrete with embedded pipes. The cooling technique was applied at the building site and no cracks has since then been observed. This has been confirmed with repeated field measurements. Stress computations have also shown that the cracking risk has been reduced considerably.

Key words: Bridge columns, thermal stresses, cracking risks, young concrete, cooling, embedded pipes.

1. BACKGROUND

The fact that concrete, especially in massive structures, tends to crack due to volume change during hydration is a problem that has been addressed by engineers for about 100 years. Measures to control cracking in the field have almost exclusively been focused on restricting temperature differences within pour and temperature rise in relation to older adjoining constructions. However, it is not until very recently that engineers with the aid of modern laboratory techniques and the prowess of modern computers have obtained more advanced understanding of the problems related to thermal stresses and thermal cracking.

If unrestrained, the concrete in a structural element would expand and contract during the early-age heating and the subsequent cooling process without any stresses being induced. In practice, however, concrete is nearly always restrained to some degree either externally by adjoining structures or internally by different temperature development in the components of the structure itself. Thus, due to these imposed restraint conditions, the temperature change will induce stresses in the concrete. A question of primary interest is of course whether the induced tensile stresses will lead to cracking or not.

When considering cracks in concrete due to hydration it is important to distinguish between

a) early age cracks in the heating stage - usually temporary. Surface cracks due to internal restraint in the concrete belong to this category.

b) permanent through cracks occurring in the cooling stage, usually due to external restraint with respect to the studied pour.

Temporary cracks usually arise within a few days from pouring and tend to close in compression at the end of the cooling period.

In order to avoid cracking caused by thermal stresses, different measures are often taken, see /1/-/6/, /16/. The temperature rise and temperature differentials due to hydration can, for example, be lowered by

a) using low-heat cements

b) reducing cement content

c) reducing placing temperature

d) using cooling techniques, e.g. with embedded pipes

e) preheating of adjoining previous casts

f) various insulation methods

The restraint of an element in a structure may be mitigated by

a) shortening the length of the section being cast by suitable arrangement of construction joints

b) favourable casting sequence

c) reducing unnecessary adherence to underlying bed rock or structure

d) allowing translation and/or deformation due to bending.

Measures and restrictions in current codes related to thermal cracking are mainly based on the ma nitude of temperature differences either within the cast section or between sections cast in differe stages. However, temperature differences generally provide only crude information about the ri of thermal cracking. Sometimes, the temperature field may, not even reveal whether a specific pa of the structure is in tension or in compression.

A characteristic feature of the thermal stress distribution in massive structures, such as the colum of Igelsta Bridge, is the occurrence of high early-age tensile stresses at the surface having a ten dency to change into compression later in the hydration temperature cycle, see Fig 1. The tensi stresses are induced by differences in the temperature between surface and interior of the sectio Restrictions for avoiding surface cracks are by tradition focused on reducing the temperature di ferences over the cross section. A maximum value of 20°C is often set as a requirement. But, a has been found in many studies /1/, /2/, /8/, /9/, /15/, the situation is much more complex. In son cases, for example when concreting under winter conditions, much higher temperature differenc can be allowed for a certain risk of cracking than at summer conditions.

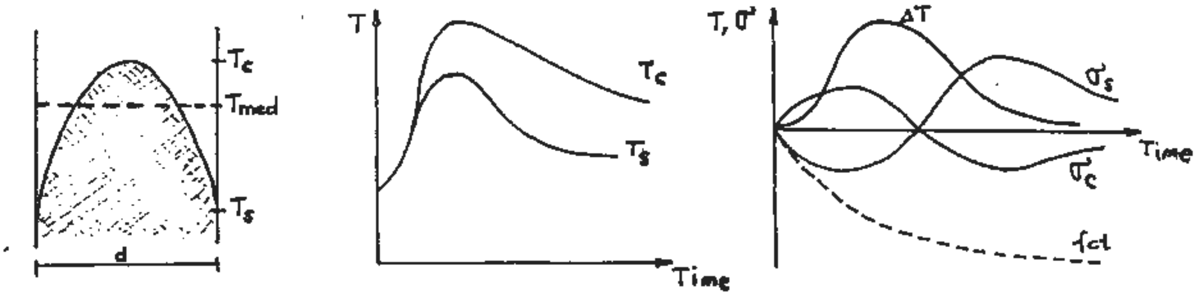

Fig 1. Differential temperature across a concrete section - T_S and T_C are temperatures at surfa and centre respectively - and thermal stresses at surface σ_S and at centre σ_C.

The studied columns of the Igelsta Bridge are particularly interesting as a case study of surface cracking risks on account of the large dimensions, rather high quality concrete (K45, se Appendi repetition of casting sequences and modern production technique /10/, see Fig 2.

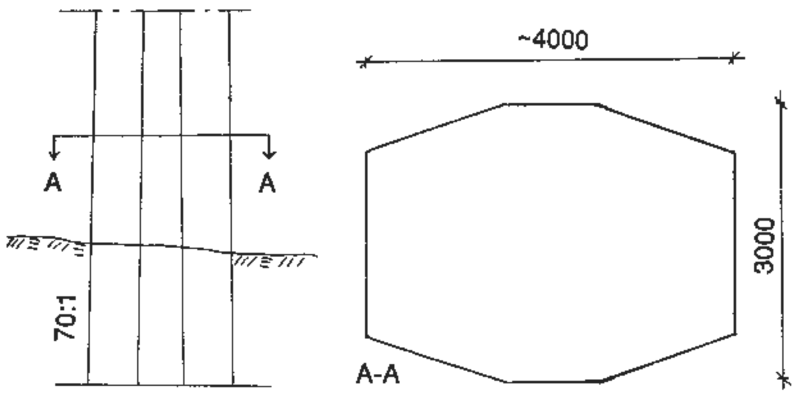

Fig 2. Column of Igelsta Bridge.

2. FACTORS INFLUENCING THERMAL STRESSES

The most important parameters when analysing early-age thermal stresses in concrete are the temperature development in the element being cast, that of the adjoining structure, the mechanical behaviour of the young concrete and the degree of restraint imposed on the element, see Fig 3.

The temperature development in the element depends on

(a) dimensions and geometry of structure

(b) thermal properties of the young concrete (heat of hydration, specific heat etc)

(c) conditions at concreting (placing temperature, form work, insulation, cooling etc)

(d) environmental conditions (ambient air temperature, temperature of neighbouring structures etc), see Fig 3.

The following properties, with regard to the mechanical behaviour of the maturing concrete, are of great importance in the thermal stress analysis:

(a) elasticity and creep (viscoelastic) behaviour at normal and high stress levels and at elevated temperatures (in some cases: plasticity)

(b) strength development

(c) thermal expansion/contraction

(d) fracture mechanics behaviour.

Generally, the restraint conditions of an element is affected by its location in the structure and the structural configuration as such. (E.g. the restraint in a wall cast on a stiff slab varies widely with the distance from the stiff edge.)

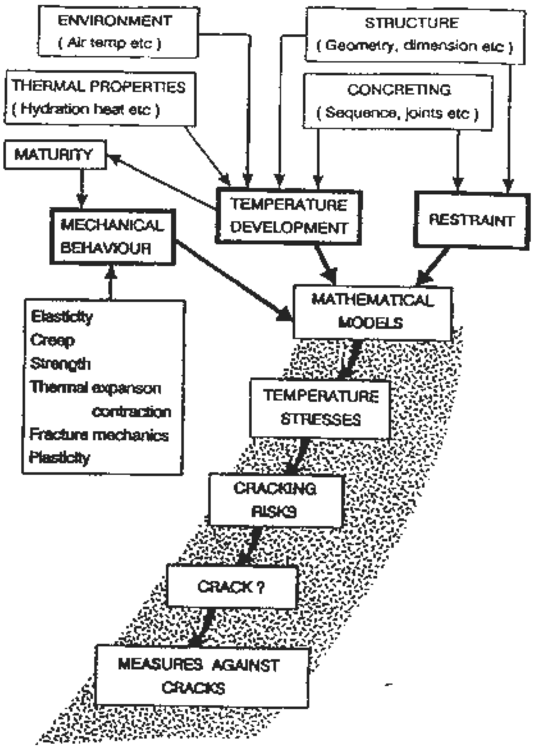

Fig 3. Analysis of early-age thermal stresses and risks of thermal cracking - influencing factors

In order to augment the knowledge regarding mechanical properties of maturing concrete, creep and relaxation tests have been performed on young concrete at the Luleå University of Technology, /7/ and /8/. Also the early-age thermal volume change has been examined, see example in Appendix.

3. STRUCTURAL ANALYSIS WITH PC-PROGRAM

A model for calculation of thermal stresses has been calibrated to the laboratory tests mentioned above (see /2/ and /8/). The model has been implemented in a computer program for structural analysis, TEMPSTRE-N. In the program, the structure is subdivided into discrete laminar layers with individual thickness. Each of the layers may be attributed different temperature developments and corresponding evolutions of maturity and material properties of its own, see Fig 4.

Calculations of thermal stresses are performed in the time domain considering elastic and creep properties of the young concrete as well as external restraint. Relaxation due to accumulated stress level is also modelled in the calculations.

The stress level i e the cracking risk is calculated as the quotient between the computed tensile stress and failure strength. Thus a stress level of 1.0 implies failure and a level higher than 0.7 is considered a substantial risk of thermal cracking because of the brittleness of the concrete and general complexity of failure mechanism.

Input temperatures required for the stress analysis of any specified case may be calculated by theoretical simulations or be obtained from field measurements. Computed temperatures in section 5 in this study have been obtained by means of the computer program HETT, /12/, /13/.

The TEMPSTRE-N analysis considers stresses in one direction and is therefore strictly applicable only when the main principal stress is very dominant. This has not proven to be a serious restriction as many applications are in fact uniaxial for all practical purposes.

However, even in situations where the temperature and stress fields are typically two-dimensional the model can be applied successfully. This is achieved by first establishing the principal stress and restraint factor in the direction of this principal stress. This may be done in e g a conventional FE-analysis. Once the restraint factor is determined the TEMPSTRE-N program may be used for calculation of the thermal stresses.

Thermal stress analysis with HETT and TEMPSTRE-N have been applied on several massive and thinner structures of different levels of complexity. Besides the Igelsta bridge, examples of applications are (see e g /2/, /8/, /15/):

- Railway box tunnels, Helsingborg and Stockholm, Sweden
- Tubular concrete tunnel in rock, hydro power plant, Carhuaquero, Peru
- Lining of rock caverns in the Middle East
- Bridge columns and caissons, The Store Baelt bridge, Denmark
- Basin walls, purifying plants, Stockholm
- Concrete cover on columns, The Öland and Smögen bridges, Sweden
- Shelter and cellar walls
- Ground slabs

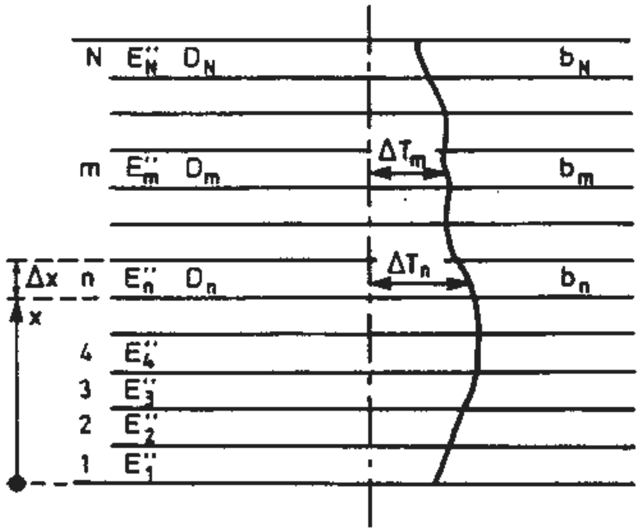

Fig 4. Analysis with TEMPSTRE-N; Subdivision of structure in laminar elements with individual thickness, temperature and early age mechanical properties.

4. DIFFERENTIAL TEMPERATURES AND THERMAL STRESSES IN BRIDGE COLUMNS

4.1 General

The massive character of the columns of the Igelsta bridge entails a temperature rise due to hydration which at the centre is close to being adiabatic. At the same time the surface of the column adjusts its temperature relatively quickly to the environment. Therefore the temperature differences between surface and centre are very important leading to high tensile stresses at the surface. The crucial issue is whether the surface stresses will result in cracking or not. When the formwork including 40 mm Styrofoam is removed the concrete surface will be exposed to thermal shock thus steepening temperature gradients and aggravating the risk of cracking.

4.2 Field measurements

Temperature measurements have been performed on one of the massive columns of the bridge for the purpose of cracking risk analysis. The temperature recordings were performed in two pouring sequences, lifts No 3 and No 4 each involving some 60 m^3 of concrete. The lifts were poured within a week from each other. Thermoelements were premounted on steel bars that were later fixed in column on form ties or reinforcement. Insulation was applied between the thermoelements and the bars thus preventing thermal conduction, see Fig 5. Fig 6 shows the location of the thermoelements.

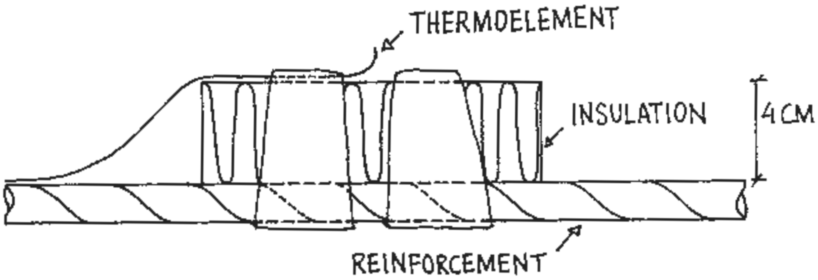

Fig 5. Thermoelement mounted on reinforcement.

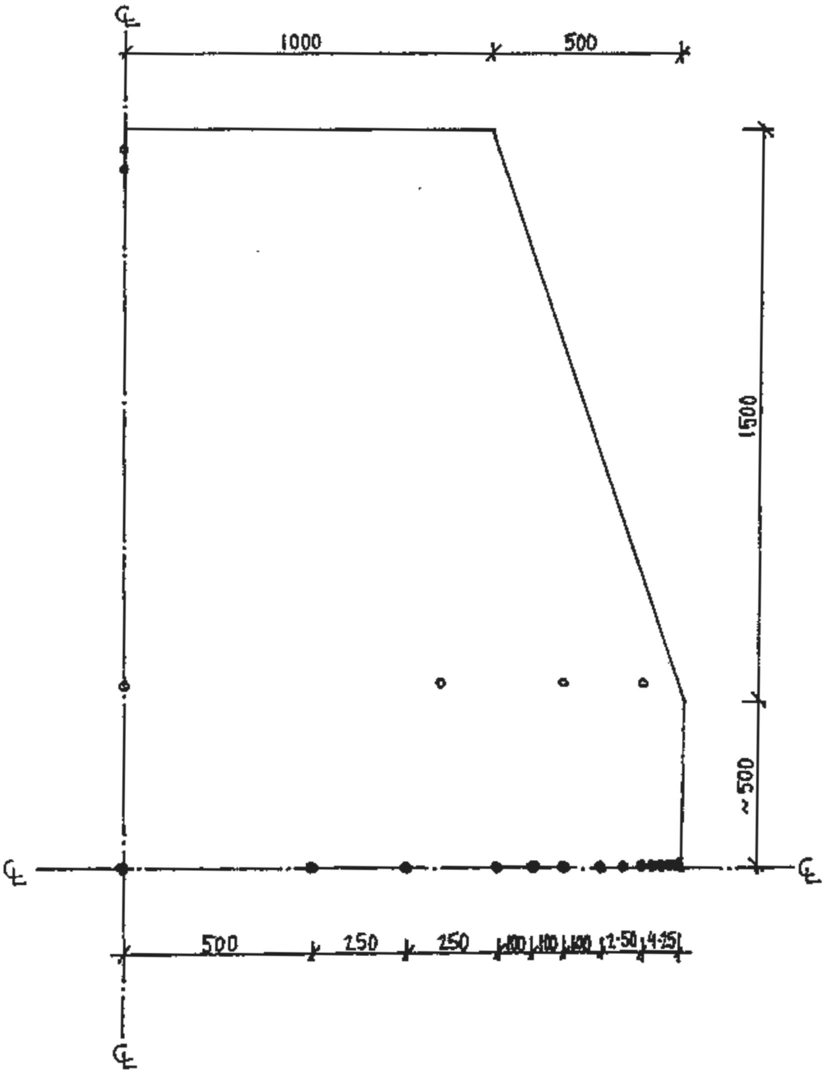

Fig 6. Locations of thermoelements in a quadrant of the column. Filled circles - thermoelements directly used in thermal stress analysis, hollow circles - thermoelement for 2D temperature field.

Main features of the temperature recordings for the two pouring sequences are listed in Table 1 showing substantial temperature differences between surface and centre before as well as after form removal. Examples of measured temperature graphs are shown in Fig 7 for three thermoelements 0 cm, 2.5 cm, and 5 cm from the surface. The large temperature drop at stripping of form for lift No 3 appears clearly in the figure. The figure also shows how air temperature fluctuations influence the surface temperature.

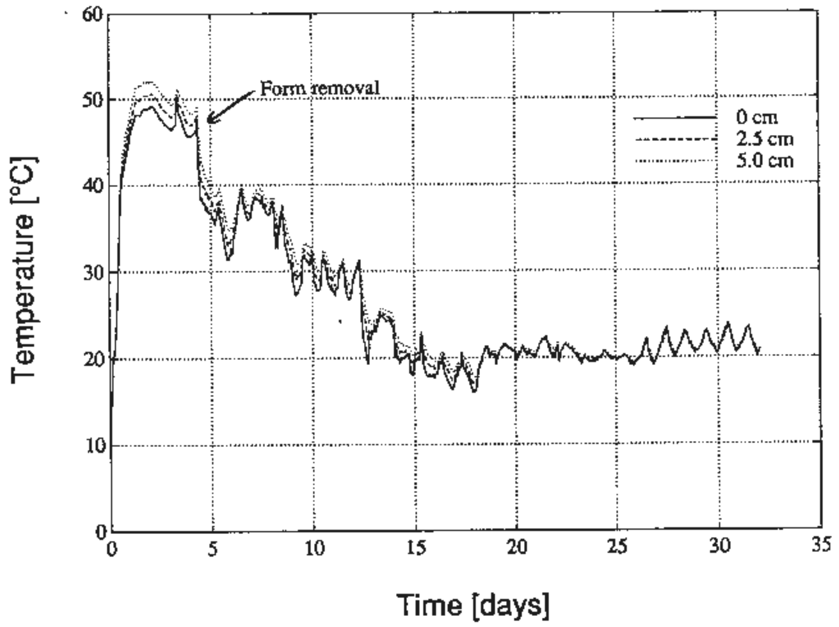

Fig 7 Examples of recorded temperatures for thermoelements located in the surface zone.

Table 1: Temperature recordings for two lifts of a massive column, Igelsta bridge - main features.

	Lift 3	Lift 4
Temperature at pouring, T_p	20.0°C	22.0°C
Maximum temperature at surface	50.4°C (48 h)	56.3°C (32 h)
Maximum temperature at centre	73.0°C (90 h)	76.4°C (90 h)
Surface temperature just prior to form removal	47.0°C (100 h)	51.2°C (100 h)
Air temperature at form removal	16.2°C	26.2°C
Temperature decrease at surface after form removal	1.75°C/h	0.5°C/h
Temperature decrease at surface after covering	0.2°C/h (10 mm blanket insulation)	0.5°C/h (no insulation)
Maximum temperature difference centre-surface	34.7°C (128 h)	50.4°C (164 h)

As can be seen in Figs 8a, 9a and from the table, temperature decrease at surface after form stripping is much faster for lift No 3 than for lift No 4. This depends on a higher air temperature at the time after form removal for lift No 4. Also, the form was first only removed some 10 mm before it one day later was stripped completely. For lift No 3 the form was removed completely at time of form stripping.

4.3 Theoretical studies

The recorded temperatures served as input data for thermal stress analysis with TEMPSTRE-N. The fluctuations of surface temperature (Fig 7) were outleveled. Discrete temperatures were given to 30 laminar elements of a symmetry cut of the column - each 5 cm thick. In Figs 8a and 9a examples of element temperatures for the two lifts are shown.

The temperature differences between the layers induce compressive stresses in the interior and tensile stresses in the surface zone, see results from TEMPSTRE-N analyses in Figs 8b and 9b. Actually, the temperature stresses are two-dimensional in the cross section of the column. However, according to earlier discussion the uniaxial TEMPSTRE-N analyses may be applied to the direction of the principal stresses. Thus, the thermal stresses in Figs 8b, 9b and 10b represent the circumferential stresses of the cross section. For both of the lifts, early tensile stresses in excess of 70% of the strength occur i e exceeding the level for cracking risk mentioned earlier.

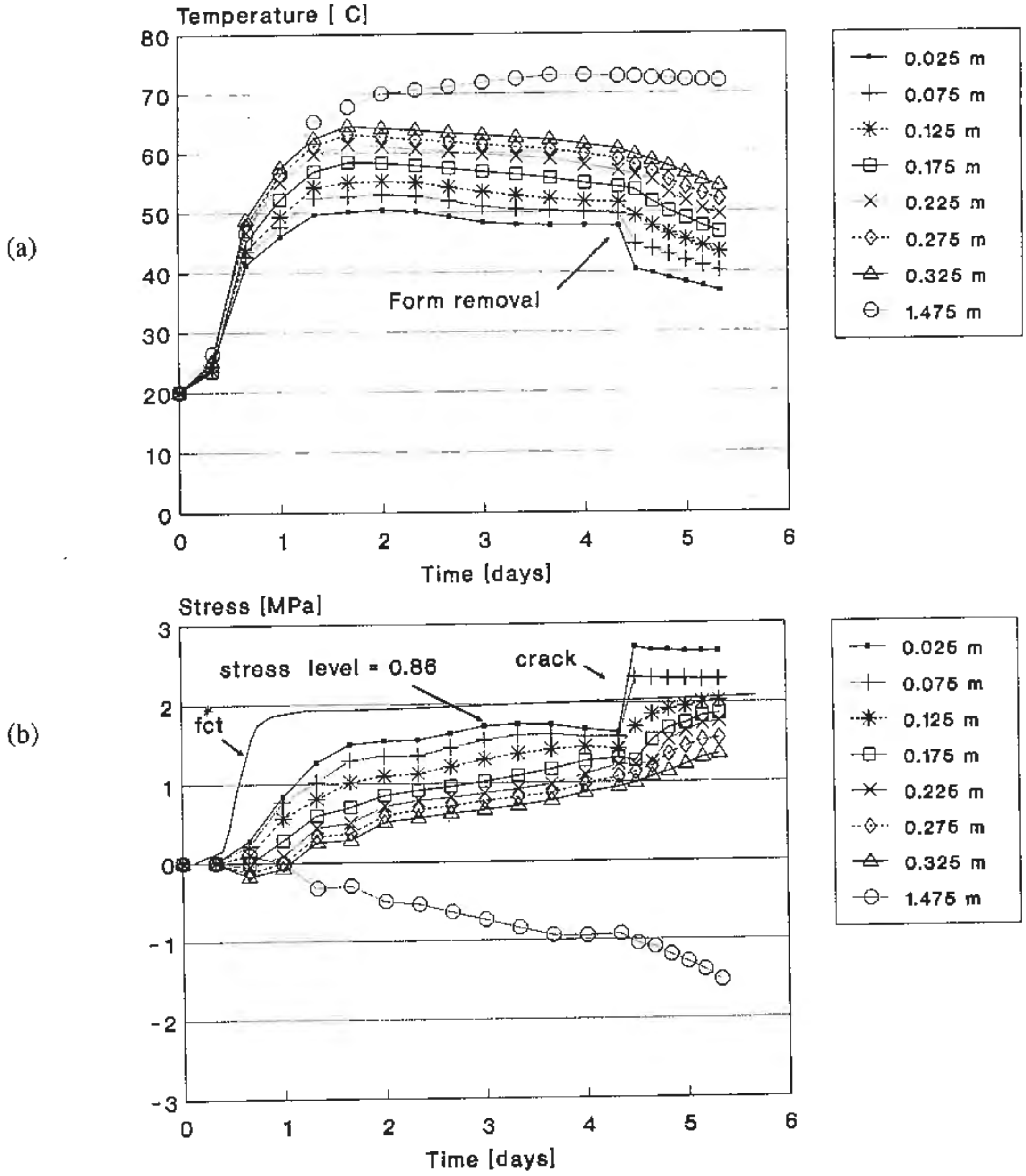

Fig 8. Input temperatures (a) and computed thermal stresses (b) for 7 surface layers and one centre layer, lift No 3, 0-5.3 days after pouring, f^*_{ct} is tensile failure strength.

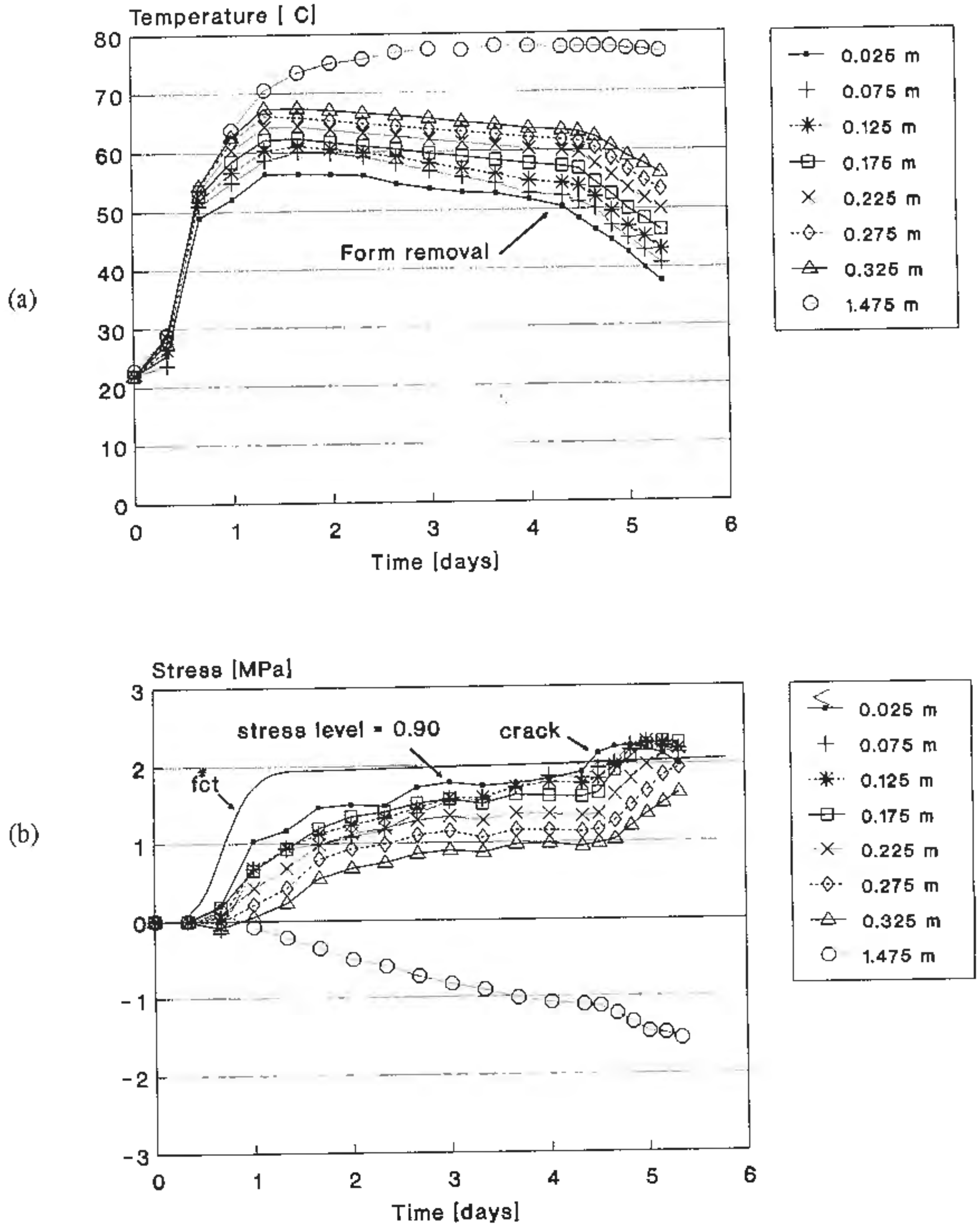

Fig 9. Input temperatures (a) and computed thermal stresses (b) for 7 surface layers and one centre layer, lift No 4, 0-5.3 days after pouring, f^*_{ct} is tensile failure strength.

For lift No 3 and No 4 almost the same high stress levels occurred in the surface layers before form removal. When the form was stripped for lift No 3 the temperature shock caused significant cracking in the surface zone, which also could be observed visually and audibly. No temperature shock occurred for lift No 4 at form removal. However, cracking was present also for this lift.

With time the tensile stresses at the surface decreased and gradually changed into compression, closing cracks - at least temporarily, see Fig 10.

In both lifts cracks went far past the reinforcement. Therefore measures are needed to prevent cracking due to risk of durability impairment.

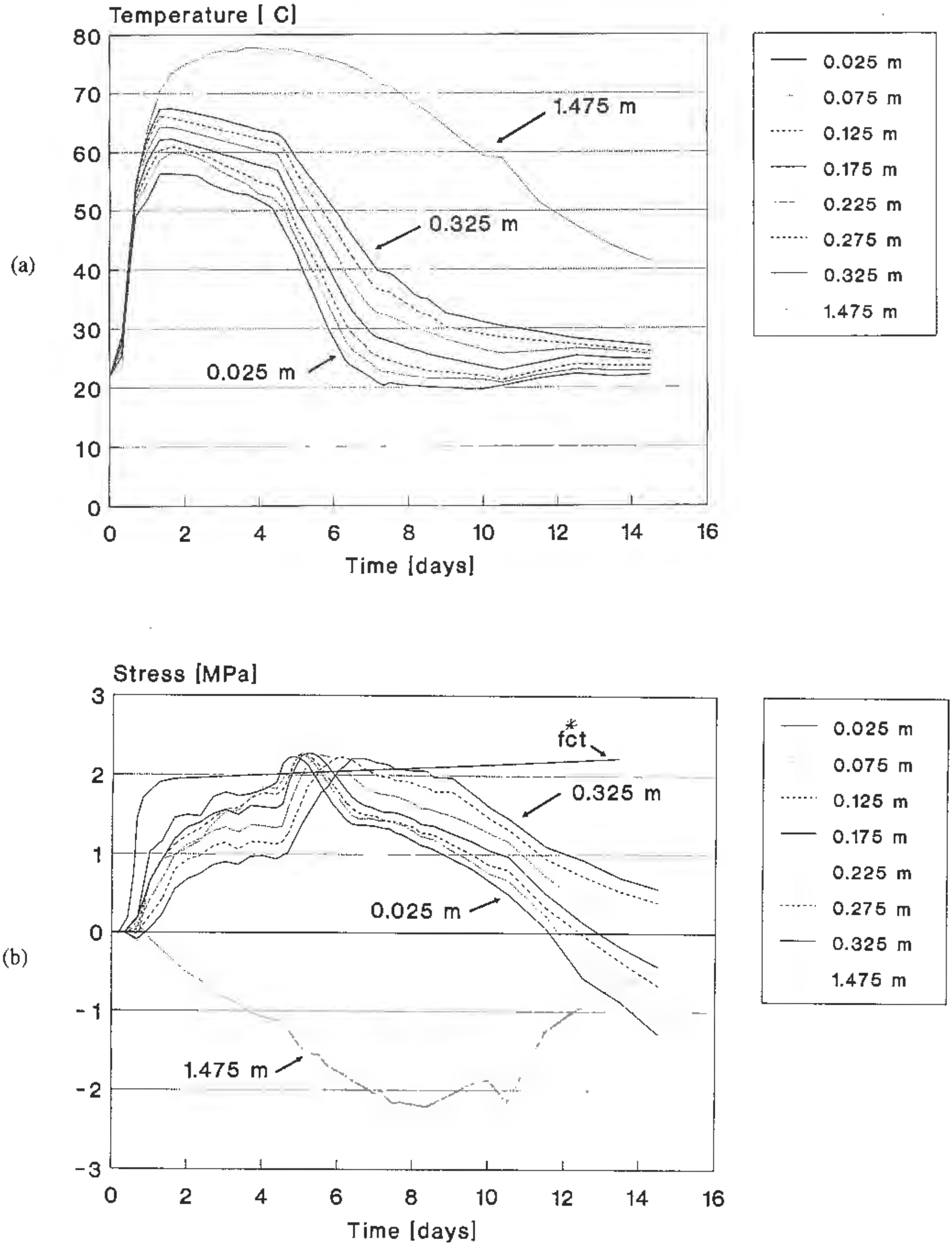

Fig 10. Temperatures (a) and computed thermal stresses (b) for lift No 4, 0-14.5 d after pouring, f_{ct}^* is tensile failure strength.

5. MEASURES AGAINST SURFACE CRACKS

At Igelsta bridge insulation of forms was used as a measure for avoiding surface cracks. This is a measure generally used for reduction of temperature differences of the cross-section of a concrete body. On the other hand insulation will lead to higher temperatures in the concrete. Higher temperatures in the surface layers lead to reduced deformability in the most critical stages which is deleterious to capacity of the concrete section to sustain differential temperatures. This implies that reduced temperature differentials by means of insulation will not effectively reduce the risk of surface cracking. Serious doubts may therefore be presented as to the beneficial effects of insulation methods of controlling cracking in early age concrete.

In the present case it is evident that the attained temperature differences are not sufficiently low to achieve "crack-free" surfaces.

To study the effect of alternative, measures against cracking (see section 2), temperature developments for different pouring conditions have been computed with the HETT-program and thermal stresses have been calculated, /10/. First adiabatic and semi-adiabatic calorimetry tests were performed (see appendix) providing input material parameters for the HETT program. Temperatures were then computed for the current pours showing acceptable agreement with the measurements.

For the various pouring conditions the thermal stresses analyses show that change of placing temperature, form insulation, time for stripping and cement content do not prevent cracking. Not even covering the stripped column with blanket insulation had any substantial effect. However, the time at which cracks occur, and crack depths are influenced by the variable conditions.

Also - as may be expected - it appeared from the theoretical studies that an effective way of avoiding cracks is to cool the concrete with embedded cooling pipes. The most efficient cooling arrangement proved to be the placing of cooling pipes according to Fig 11 (see /10/). Theoretically, it was possible to reduce the maximum temperature difference in column from more than 30°C to about 10°C and the maximum stress level (i e cracking risk) at the surface to about 0.20.

This cooling technique has been applied at the building site of the Igelsta bridge. Thus, tubes, 16 cm in diameter, were embedded in the columns and air was used as cooling medium flowing from the bottom of the column and up through the recently cast lift. For 10 lifts where the cooling method has been used so far, no surface cracks have been observed.

In order to examine the efficiency of the method, further field measurements have been carried out. Fig 12 shows examples of recorded temperatures in tubes. Although there were serious reductions of flow in two of the tubes (due to partial failure of tube walls), a considerable cooling of the column was obtained. For example, almost no flow occurred in the centre pipes, leading to high temperatures in the tube, see Fig 12. The maximum temperature in centre of section was about 10°C lower than for previous non-cooled sections and the temperatures around cooling pipe were not higher than about +45°C. The thermal stress analysis for the cooled section showed a maximum cracking risk of 0.56, which will be further reduced if no failure of cooling tubes occur.

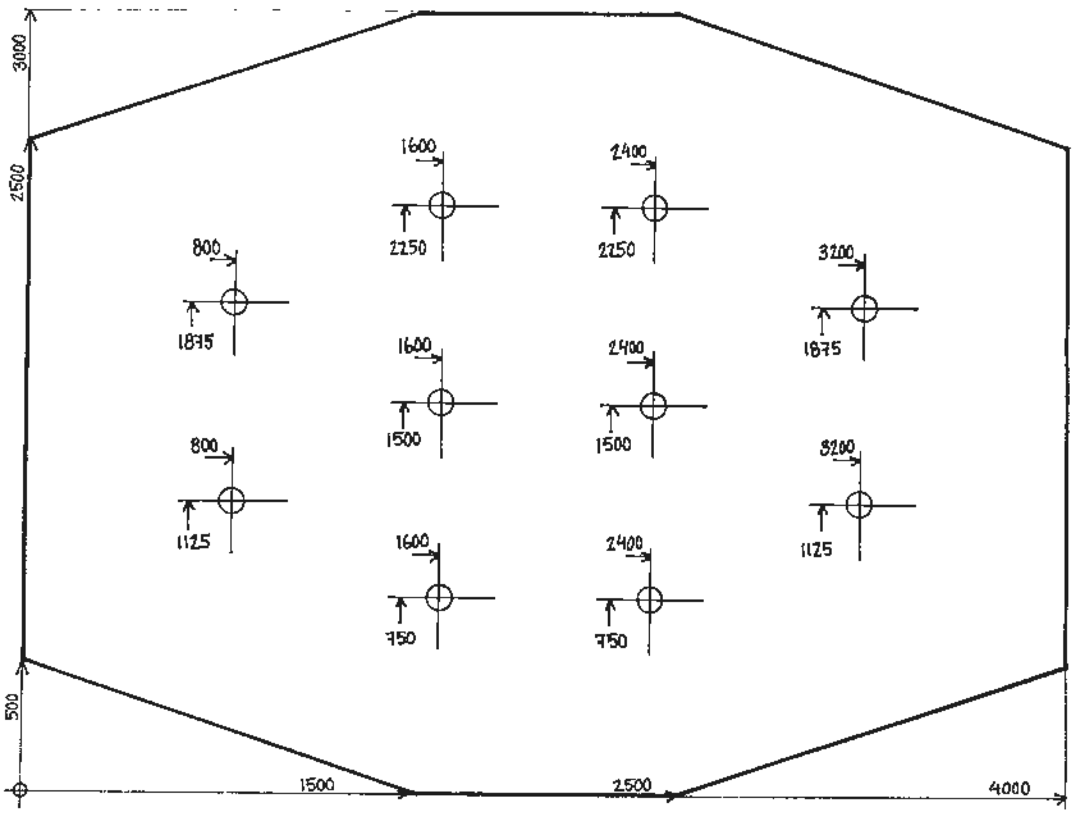

Fig 11. Cross section of column with embedded cooling pipes.

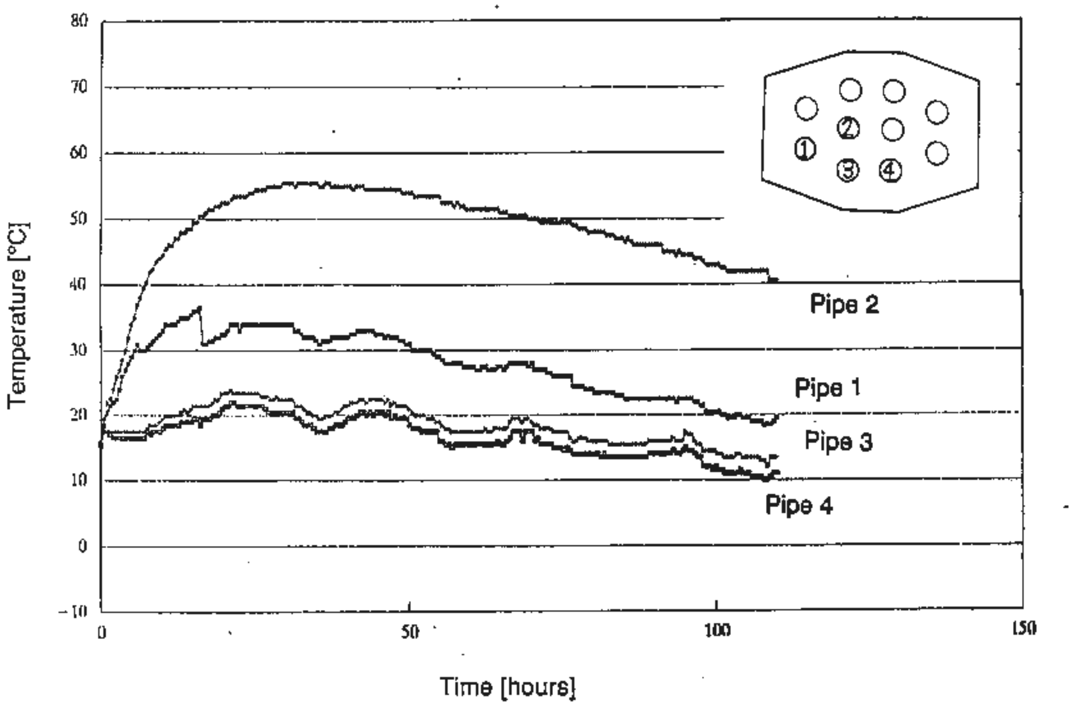

Fig 12. Recorded air temperatures in four of the tubes.

6. CONCLUSIONS

The field measurements and theoretical analysis show that

1) Reducing temperature differences with heavy insulation proved inefficient with regard to cracking risks. In particularly, removal of insulated forms presented serious difficulties.

2) Of the considered methods for control of cracking only cooling with embedded pipes was sufficiently effective.

3) HETT and TEMPESTRE-N programs could readily be used for simulating temperatures and thermal stresses in the bridge columns. Different methods for cracking control may be evaluated and compared.

ACKNOWLEDGEMENT

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APPENDIX

The thermal stress analyses of the bridge columns are based on laboratory tests on actual concrete mixtures (with actual aggregate). Cement type: Std P Degerhamn, cement content = 425 kg/m^3, $w_0/c = 0.40$, aggregate 0-8 mm: 640 kg, 8-18 mm: 510 kg, 18-32 mm: 540 kg, 28-day cube strength = 53 MPa.

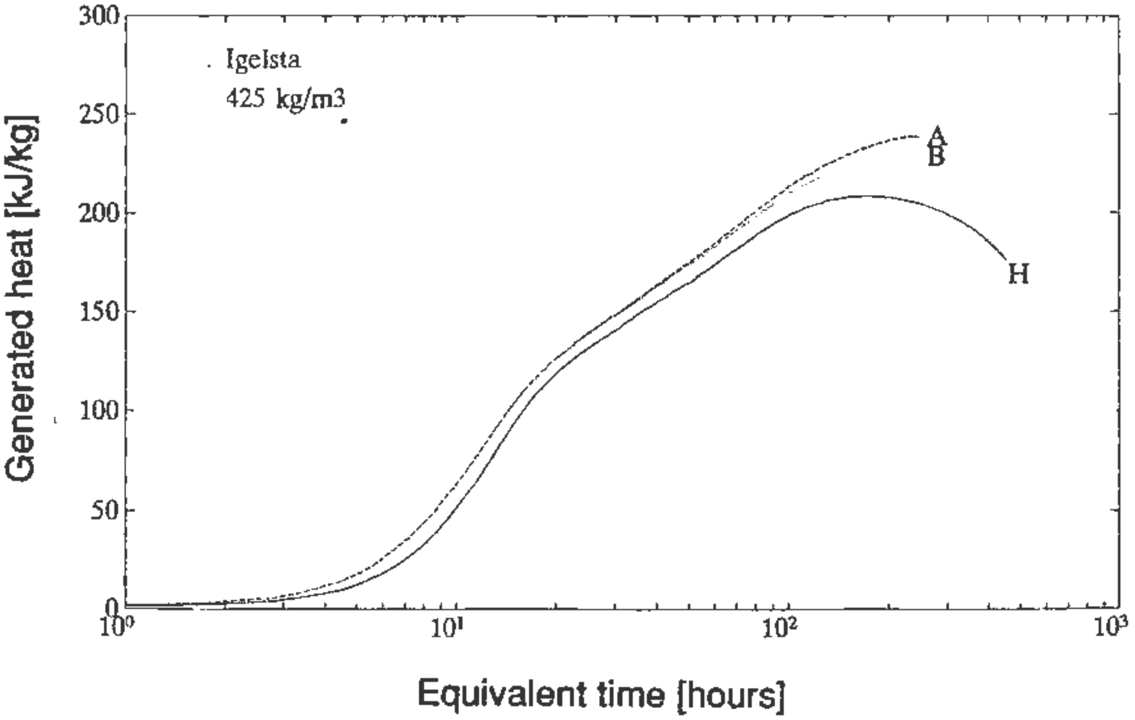

Fig A1. Adiabatic and semiadiabatic calorimetry tests.

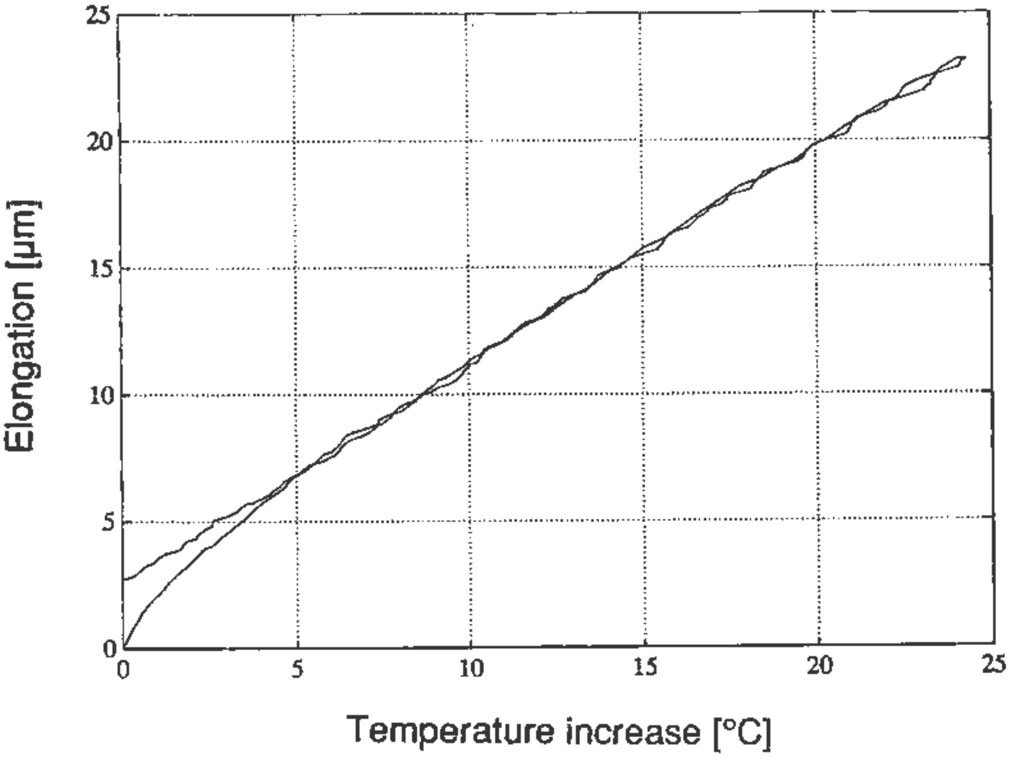

Fig A2. Thermal expansion and contraction at very early ages.

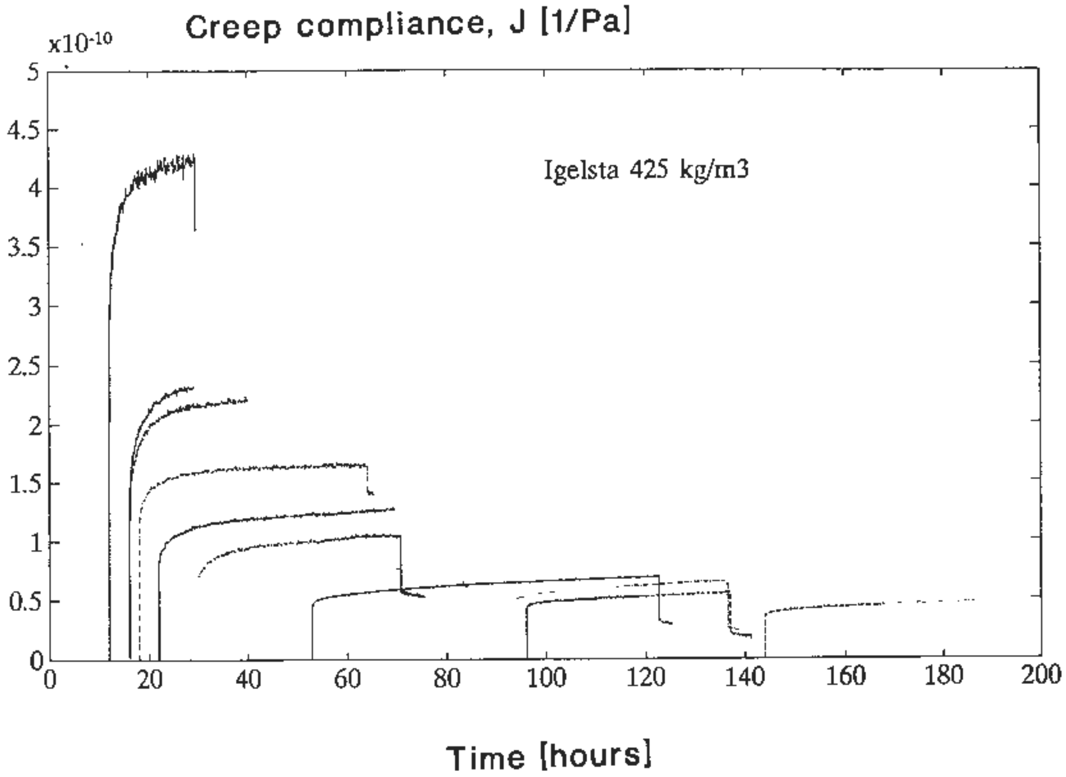

Fig A3. Creep tests at early ages.