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Thermal Stresses in Concrete Bridges

Contraintes thermiques dans les ponts en béton

Temperaturspannungen in Massivbrücken

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SUMMARY

Temperature stresses in concrete bridges result from the heat released during cement hydration and from insolation and ambient heating. In the paper thermal properties of concrete are discussed. The main results of thermal analysis and experimental research of temperature distribution on two large prestressed concrete bridges built in Prague, Czechoslovakia in recent years are presented. It is shown that in special cases temperature stresses have even greater magnitude than live load stresses.

RÉSUMÉ

Les contraintes thermiques dans les ponts en béton résultent de la chaleur se produisant au cours de l'hydratation du ciment, de l'insolation et de la chaleur ambiante. Cette étude décrit aussi les qualités thermiques du béton armé et présente les résultats principaux de l'analyse thermique et des recherches expérimentales de la distribution de température de deux ponts en béton précontraint construits à Prague, Tchécoslovaquie, dans les années récentes. L'étude montre aussi que dans des cas spéciaux, les contraintes thermiques ont une variation plus grande que les contraintes causées par les charges du trafic.

ZUSAMMENFASSUNG

Temperaturspannungen in Massivbrücken entstehen bei der Entwicklung der Hydratationswärme sowie infolge Sonnenstrahlung und Umgebungswärme. Der Beitrag umfasst eine Beschreibung der Temperaturstoffkennzahlen für Beton. Die Hauptergebnisse der Berechnung der Temperatur- und Spannungsfelder sowie Temperaturmessungen an zwei grossen Spannbetonsbrücken, die in den letzten Jahren in Prag, Tschechoslowakei, gebaut wurden, werden vorgestellt. Es ist daraus ersichtlich, dass in Spezialfällen die Temperaturspannungen sogar grösser sind als die Spannungen aus Verkehrslasten.

1. INTRODUCTION

For many years concrete bridges have been designed mainly for dead and live load stresses. In-situ measurements carried out in last decades have shown that in special cases stresses of the same or even greater magnitude can be caused by thermal loads, creep and shrinkage. While the effects of creep and shrinkage have been studied for many years and several organisations as CEB or FIP have given international recommendations for calculation of these effects, thermal response of concrete structures is a complex phenomenon nearly ignored in conventional design.

Restraints of free thermal expansions induce stresses. Localised stresses may be caused by the thermal incompability between cement mortar and aggregate which affect concrete durability. In concrete bridge design the main sources of heating are as follows:

- Insolation and ambient heating. Bridges thermally respond to solar radiation, wind exposure, and ambient temperature changes, both daily and annual cycles.
- The heat released during cement hydration. This is more important in relatively thick sections, as bridge abutments, piers and decks.

The major form of heat input is solar radiation. Although the soffit and external web surfaces may receive an amount of reflected radiation from the surroundings, this will generally be a small proportion of the direct radiation on the deck surface. Convective heat transfer occurs at the surface when a temperature difference exists between the surface and the surrounding air, and is accentuated by movement of air along the surface. Radiative losses may occur from the surface to adjecent colder surfaces or to the sky.

The reaction of cement with water is exothermic and releases a considerable quantity of heat. Modern cements are finer, and have increased outputs of heat. The temperature distribution in mass-concrete structures depend on the amount and rate of heat-of-hydration emission, and the three-dimensional heat flow. The former is a function of cement type, mix design and concrete temperature, while the latter is a function of ambient conditions, the properties of concrete, insulation and temperature distributions. The most important temperature differences are these between the hot interior and the colder surface in a few days after concreting.

Over each time increment the incremental thermal stress is the product of the incremental restrained strain and the concrete elastic modulus. The modulus is, however, a function of both time and temperature, with the early strains inducing very little thermal stresses. Thermal stresses in mass structures are significantly affected by creep. So the thermal stress distribution is a complex function of the inter-related parameters of creep, time, elastic modulus and temperature. Cracking alters the stress distribution, and thus, the change in concrete tensile strength with time and temperature should be considered.

2. THERMAL PROPERTIES OF CONCRETE

For the concrete bridge design and specially for the calculation of thermal stresses main thermal properties of concrete, as conductivity, specific heat, thermal diffusivity, thermal expansion coefficient, hydration heat of cement and surface heat transfer coefficient should be known. Many investigations have been made for large concrete dams while there thermal stresses are usually far more significant than live or dead load stresses. Unfortunately, concrete technology for these structures is quite different than that for concrete bridges. Generally, for concrete mix not thermal properties, but high ultimate strength and sufficient workability are most important in bridge technology.

Thermal conductivity measures the ability of the material to conduct heat. The conductivity of concrete depends on its degree of saturation, composition, porosity, density, temperature, and mineralogical character of the aggregate and ranges generally between about 1.3 and 2.3 ($J.s^{-1}.K^{-1}$). For concrete mix in concrete bridges with high density and low porosity the conductivity reaches the higher value.

Thermal diffusivity represents the rate at which temperature changes within a mass can take place. The range of typical values of diffusivity of concrete is between 5 . 10^{-7} and 10 . 10^{-7} ($m^2.s^{-1}$). Because of the influence of moisture in the concrete on its thermal properties, diffusivity should be measured on specimens with a moisture content which will exist in the actual structure.

Specific heat, which represents the heat capacity of concrete, is considerably increased by an increase in the moisture content of the concrete. It is little affected by the mineralogical character of the aggregate, and depends also on the actual range of temperature. The common range of values for concrete with a normal moisture content is between 0.84 and 1.20 ($J.g^{-1}.K^{-1}$).

Thermal expansion coefficient of concrete depends both on the composition of the mix and on its hygric state at the time of the temperature change. Thermal expansion coefficient varies between 7.0 . 10^{-6}(K^{-1}) for limestone - concrete and 13.0 . 10^{-6}(K^{-1}) for quartzite - concrete.

The heat of hydration is the quantity of heat, in Joules per gram of unhydrated cement, evolved upon complete hydration at a given temperature. The temperature at which hydration takes place greatly affects the rate of heat development. In fact, the heat of hydration depends on the chemical composition of the cement, and the heat of hydration of cement is very nearly a sum of the heat of hydration of the individual compounds when hydrated separately. For usual Portland cements the complete heat of hydration ranges generally between about 300 and 500 ($J.g^{-1}$). For separate compounds the heat of hydration in ($J.g^{-1}$) varies between 500 and 570 for C_3S, 180 and 270 for C_2S, 830 and 910 for C_4A, and 290 and 420 for C_4AF.

Surface heat transfer coefficient is a function of wind speed, concrete temperature, and moisture content. A relation between the top surface heat transfer coefficient h ($J.m^{-2}.s^{-1}.K^{-1}$) and wind speed w ($m.s^{-1}$) can be written as follows: $h = 11.4 + 4.66\ w$.

3. TEMPERATURE CHANGES ON THE KLEMENT GOTTWALD BRIDGE

3.1 Measurement programme

The Klement Gottwald Bridge over the Nusle valley in Prague (Fig. 1), built in 1973, is at present the largest prestressed concrete bridge in Czechoslovakia. During the construction of the bridge a large - scale research programme was under way, one item of which consisted in measuring the temperature of concrete in the superstructure. The temperatures were measured in two phases: 1^o- during the hydration process of concrete; 2^o- after the construction works were finished. In the latter phase the influence of ambient temperature changes and of insolation has been investigated with special regard to the forming of temperature gradients, and the resulting temperature stresses. In the superstructure (Fig. 2) altogether 139 thermocouples were embedded in the concrete, the elements being arranged in five cross-sections. The first phase of measurements took place in December 1969 when very low outside temperature was registered. In the second phase the concrete temperatures were measured in August 1973, when the influence of various outside atmosphere conditions was investigated /1/.

3.2 Measurements during the development of hydration heat

The temperatures of concrete, as they were ascertained in December 1969, have been influenced mainly by the following factors:
- Exceptionally low outside temperature, since during all the time interval when measurings were under way the temperature of air kept steadily below zero (^{o}C)

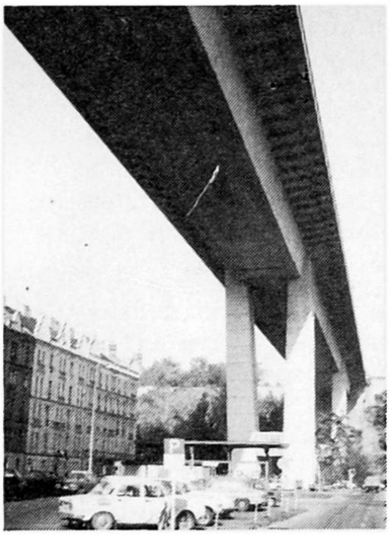

Fig.1 Klement Gottwald Bridge

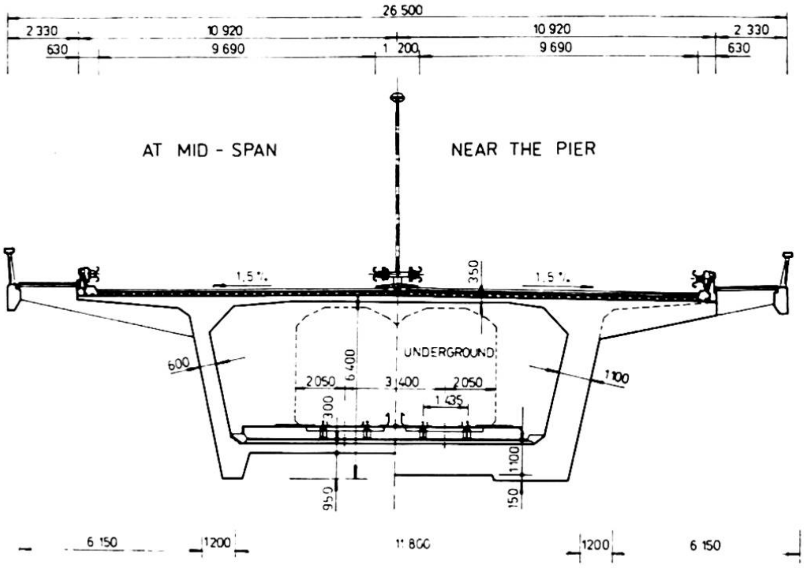

Fig.2 Cross section of the bridge

as from Fig. 4 can be seen.

- Pre-heating of the concrete mix so that the temperature at placing of the mix was some 35°C on the average. Hot air blow was used for protection of placed concrete against frost.

The concreting of each lamella 3,5 m in length was carried in 3 phases - the bottom slab, the walls and at last the top slab with the rigid frame corner and the cantilevers. In Fig. 3 the distribution of temperature in the cross section 24 hours after placing of concrete in the top slab, i.e. 7 days after the concreting of the walls is shown. At this time-point the temperature in the corner was 51.1°C, while in greater part of the walls and in the whole bottom slab the concrete temperature was deep below zero. The time-distribution of temperature in the upper corner (point a in Fig. 4 - in a thermocouple 0.80 m under the top surface of the upper slab) is shown in Fig. 4. The highest temperature-reading 56.0°C corresponds to the time point when the heating of concrete by means of hot air blow was finished.

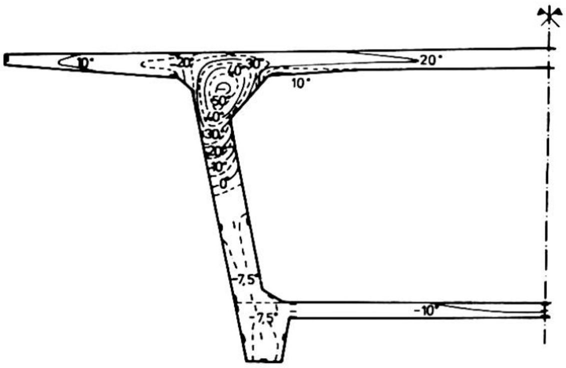

Fig.3 Temperature distribution in the cross-section

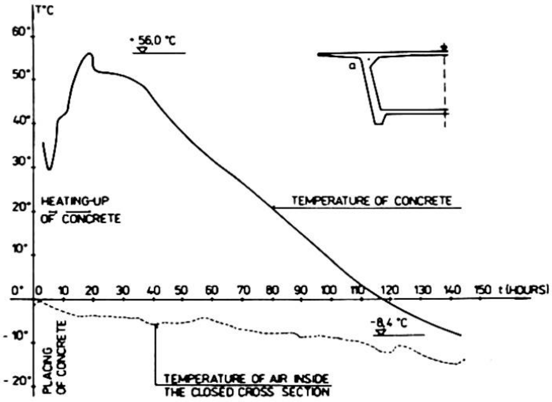

Fig.4 Time-distribution of temperature in the upper corner a

3.3 Concrete temperatures and stresses due to ambient temperature changes

Temperature of the concrete of the finished superstructure has been measured with respect to the influence of ambient temperature changes and of insolation, the measurings being performed in August 1973. The influence of various outside atmospheric conditions has been investigated, from simple temperature change without insolation to the influence of very intensive insolation. The wall exposed to insolation changes its temperature rapidly. Since the temperature variation throughout the concrete is not linear, this gives rise to secondary compressive stresses near the outer surface.

The temperature gradients in the bridge cross-section, as measured on August 9th, at 6,00 in the morning are shown in Fig. 5. The measurements show that the temperature of the bridge deck is quite stationary due to asphalt insulation and wearing surface, while the temperature gradients in the walls and the bottom slab reach comparatively high values - about $4^{o}C$ per 30 cm of depth. This part of the bridge is intensively cooled off by cold air flow in the valley. The analysis of thermal stresses using a computer for a finite element method response is based on the measured characteristics of the temperature field. For the isothermal lines shown in Fig. 5 the computed stresses can be seen from Fig. 6. Maximum tensile stresses reach 2.04 $N.mm^{-2}$ while the structural analysis gave for the whole live load maximum tensile stress 1.93 $N.mm^{-2}$.

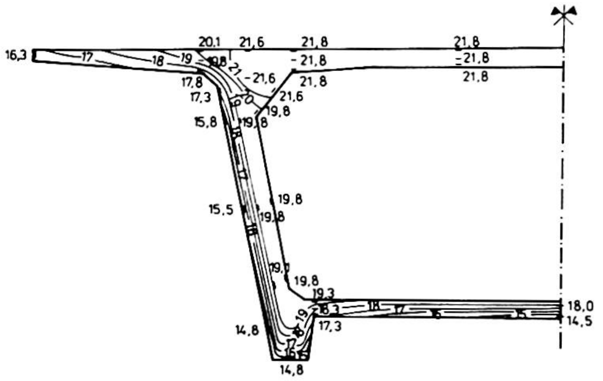

Fig. 5 Temperature field on August 9th in the morning

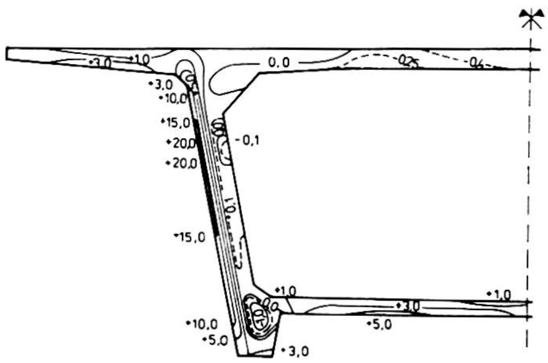

Fig.6 Tensile stresses due to temperature field on Fig. 5

4. ANALYSIS AND MEASUREMENTS OF THERMAL STRESSES ON ANTONÍN ZÁPOTOCKÝ BRIDGE

4.1 Measurements and analysis programme

The Antonín Zápotocký Bridge in Prague (Fig. 7) is the latest large and very irregular structure crossing the river Vltava built in 1988 /2/. During the construction experimental research of strains, stresses and temperature was carried out. The four main piers have prestressed concrete capping beams more than 39 m long, 5 meters high and from 2 to 4.4 meters wide where more than 600 cu.m. of concrete Class 40 was placed in one operation. Since the contractor could not ensure the cooling of the concrete mix or its components, it was necessary to analyse also the state of stress of the structure due to development of hydration heat. Fourier´s equation which describes heat movement in the capping beam was solved using triple integral of Jacobi Theta function. Basic assumption for the stress analysis was that the elastic modulus of concrete varies with time in the same manner as the heat-of-hydration emission /3/. Thermal stresses were computed taking concrete creep in consideration. During the construction of the main four piers measurements of hydration heat and strains were carried out to

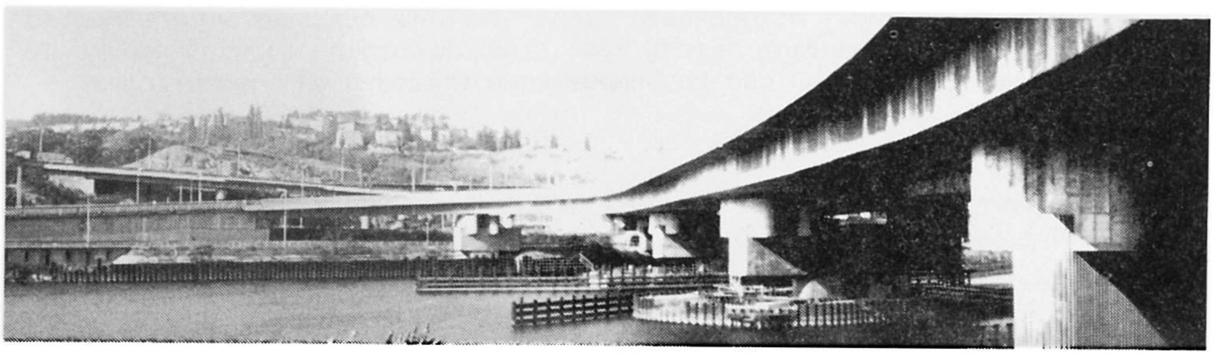

Fig.7 Antonín Zápotocký Bridge across the Vltava

prove agreement between theoretical temperature and stress distribution and experimental research.

4.2 Main results of thermal stress analysis and measurements

A general programme of thermal field and stress analysis was prepared for a 9825 T Hewlett Packard calculator according to the complexity of the problem to be solved, where thermal properties of concrete as discussed in chapter 2 were applied. From Fig. 8 the main results of thermal field analysis can be seen. Thermal stresses due to the thermal field are shown in Fig. 9. At the beginning tensile stresses develop near both surfaces of the capping beam wall, reaching maximum 3.6 $N.mm^{-2}$ 5 days after concrete placing. One year later these tensile stresses change in compressive stresses about 0.7 $N.mm^{-2}$ near the concrete surface. Fig. 10 shows the temperature distribution as measured 5 days after completing the placing of concrete in the capping beam of the pier. The measured strains showed principal agreement with the values obtained from the calculation.

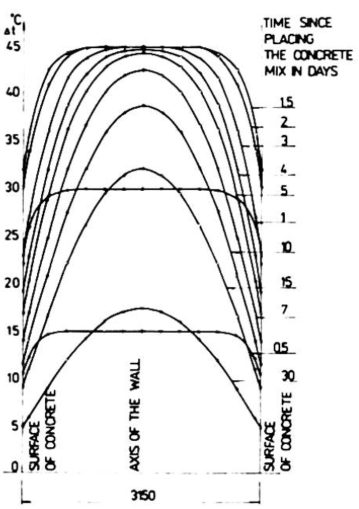

Fig.8 Temperature distribution in the capping beam wall

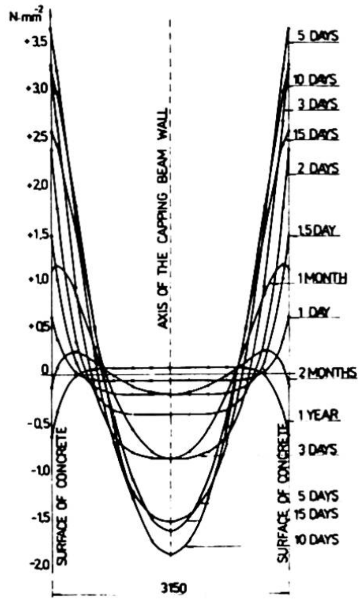

Fig.9 Thermal stresses due to temperature distribution in Fig. 8

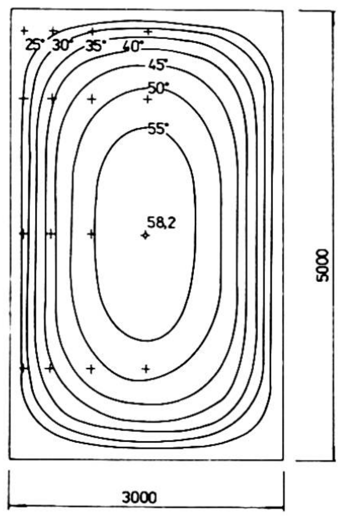

Fig.10 Measured temperature field 5 days after placing of concrete mix

5. CONCLUSION

Thermal analysis and experimental research of temperature distribution on two large bridges in Prague, Czechoslovakia showed interesting results. On the Klement Gottwald Bridge the tensile stresses due to temperature gradients influenced by the cold wind in the valley were found higher than those for the whole live load. On the Antonín Zápotocký Bridge even higher tensile stresses were computed caused by the heat released during cement hydration. According to high amount of mild steel no cracking was found due to temperature stresses on both bridges.

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