Proceedings
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Thermal Cracking in Concrete at Early Ages

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Thermal Cracking in Concrete at Early Ages

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Contents

Preface

Cracks in mass concrete structures caused by the hydration heat of the cement have been a well-known phenomenon since the beginning of this century. Methods of avoiding such cracks have been developed mainly for large concrete dams and other massive hydraulic engineering structures. In order to reduce the heat development, pozzolanas and since 1932, low-heat cements have been used. Further progress aimed at reducing the high maximum temperature caused by the hydration heat has been made by the use of very low cement contents, coarse aggregates, cooling the concrete materials, limitation of lift-heights and by pipe cooling.

Although the processes of heat generation and dissipation were familiar to engineers, the specification of the maximum permissible temperature difference between the concrete mass and its base and between the inside and the surface of a concrete block was based solely on experience. The essential properties of a specific concrete such as its tensile strength or its coefficient of thermal expansion were not considered.

In the past decades, cracks in the structural concrete of foundations, bridges, tunnel linings and other medium-sized concrete elements have become an increasing problem. It has been found that drying shrinkage is often of minor importance. The heat of hydration as well as other temperature changes were established as the main causes of restraint stresses and cracks in unreinforced as well as in reinforced concrete.

In the late sixties the first attempts were made to estimate the stresses caused by restrained thermal deformations and to compare them with the increasing tensile strength of the young concrete. Two points turned out to be extremely difficult:

- The results of thermal stress calculations depend strongly on the evaluation of the increasing stiffness of the concrete during its transition from a semi-liquid to a solid state. However, the stiffness is difficult to measure and predict.
- Restraint stresses cannot be determined with conventional methods and therefore no data were available to verify the results of stress calculations.

In 1969 the first laboratory equipment, the cracking frame, was developed which allowed model tests. By measuring the stress response of young concrete to changing temperature we gained a deeper understanding of the changes which occur when the expansion or contraction of a concrete element is prevented and consequently converted into stresses. The Munich Temperature Stress Testing Machine (1984) and similar machines in several other research institutes now allow stress measurement for any degree of restraint.

In recent years much research work has been devoted to the calculation of the early age restraint stresses and to the determination of the risk of cracking. Computer programs have been based on the properties of materials, the hydration heat development, the increase of stiffness and the decrease of relaxation capacity, the increasing tensile strength, the coefficient of thermal expansion and the influence of chemical reactions on the deformation. All these factors depend largely on age, temperature, cement type and concrete mix composition. Realistically speaking, it is only possible to assess roughly the effects of these factors. However, much progress has been made with models for the approximation of materials properties. Such models require assumptions about the restraint conditions and the expected temperature conditions on site.

Promising new methods have been developed in Japan and France to measure restraint stresses in situ. The comparison of test results both in the laboratory and in the field with the results of calculations is a source of further progress in this area.

In recent years high-strength concrete has proved to be an extremely sensitive material regarding cracking at an early age. This is not only a consequence of the hydration heat: Autogenous shrinkage due to self-desiccation and chemical reactions of the sulphate phase can also be important.

Unexpected cracking of structural or mass concrete can not always be attributed to the inexperience of the field engineer, and the limited knowledge of many problems in this area has strongly encouraged research all over the world. In 1989 RILEM, the International Union of Testing and Research Laboratories for Materials and Structures, established a Technical Committee, TC 119, on "The Avoidance of Thermal Cracking in Concrete at Early Ages". As well as the exchange of opinions and experience, the tasks of this committee are to prepare a State of the Art Report and make recommendations for the following test methods:

- Determination of the semi-adiabatic hydration heat
- Determination of the adiabatic hydration heat
- Restraint stress measurements in the laboratory with the cracking frame
- Restraint stress measurements in situ with the stress-gauge.

The International Symposium in Munich between October 10th and 12th, 1994 thus takes place in a dynamic period of development. A number of questions regarding the avoidance of early age cracking have been solved and the results are ready for practical application. To answer other questions we have many suggestions based on test results or from theoretical considerations.

It still has to be determined how these methods can be applied most effectively in practice, whether they need further development or whether they can only serve as a stimulation for further research.

In order to limit the scope of the work, drying shrinkage is not taken into consideration. Furthermore the role of steel reinforcement to avoid wide crack opening and the advantageous use of prestressing are not treated in this symposium.

The avoidance of early age cracking is a task which requires theoretical knowledge, sound engineering judgement and extensive experience. Furthermore, dedicated engineers must ensure that all the necessary considerations are carried out in practice.

I hope this symposium becomes a milestone of progress of our field.

Rupert Springenschmid
Munich, July 1994

Preface

La fissuration du beton dans les constructions massives, due a la chaleur d'hydratation du ciment est un phenomene bien connu depuis le debut du siecle. Des methodes permettant d'eviter les fissures de ce type ont ete developpees surtout pour les grands barrages en beton ou pour les structures massives du genie hydraulique. Les pouzzolanes et, depuis 1932, les ciments a faible chaleur d'hydratation sont utilises pour limiter la chaleur liberee. D'autres progres tendant a reduire le pic de temperature du a la chaleur d'hydratation ont ete realises par la mise en oeuvre de dosages en ciment fortement reduits, par I'emploi de granulats plus gros, par refrigeration des constituants du beton, par la limitation de la hauteur des levees, et par le refroidissement par circulation d'eau dans des serpentins.

Bien que les processus de production et de dissipation de la chaleur sont familiers aux ingenieurs, la specification d'une difference de temperature admissible entre la masse de beton et la fondation et entre le coeur et la surface d'un massif de beton faisait exclusivement appel a I'experience. Les proprietes fondamentales d'un beton donne, telles que sa resistance a la traction ou son coefficient de dilatation, n'etaient pas prises en compte.

Au cours des dernieres decennies, les fissures dans le beton de structure de fondations, de ponts, de revetements de tunnel, ou d'autres constructions de taille moyenne sont devenues un probleme croissant. H a ete montre que le retrait de dessiccation est souvent d'importance mineure. La chaleur d'hydratation, ainsi que les changements de temperature, sont reconnus comme etant la principale cause des contraintes dues aux deformations empechees et des fissures, tant dans le beton non arme que dans le beton arme.

A la fin des annees soixante, les premieres tentatives pour evaluer les contraintes dues aux deformations thermiques empechees et pour comparer celles-ci a la resistance a la traction du beton jeune ont ete faites. A cette occasion, deux points se sont reveles extremement difficiles.

- Les resultats des calculs de contrainte thermique dependent fortement de revaluation de la raideur du beton, raideur qui croit lors de la transition de I'etat semi-liquide a I'etat solide. Cependant, il est difficile de mesurer et de prevoir la raideur du beton.
- Les contraintes dues aux deformations empechees ne peuvent etre mesurees par des methodes conventionnelles et, de ce fait, on ne disposait d'aucune donnee pour verifier les resultats des calculs de contrainte.

En 1969 a ete developpe le banc de fissuration, premier equipement de laboratoire permettant des essais sur eprouvettes reproduisant les conditions de chantier. La mesure de la reponse en contrainte du beton jeune aux changements de temperature a rendu possible une compre-hension plus approfondie des changements qui se produisent lorsque I'allongement ou le raccourcissement d'une structure en beton est empeche et provoque I'apparition de contraintes. L'appareil d'etude des contraintes thermiques construit a Munich en 1984 ainsi que d'autres dispositifs similaires developpes dans de nombreux autres laboratoires de recherche permettent desormais une evaluation des contraintes sous n'importe quelles conditions de deformation imposee.

Un effort de recherche important a ete consacre ces dernieres annees au calcul des contraintes dues aux deformations empechees au jeune age, ainsi qu'a la determination du risque de fissuration. Les programmes de calcul doivent prendre en compte les proprietes des materiaux, le developpement de la chaleur d'hydratation, Taugmentation de la rigidite et la diminution du coefficient de relaxation, la resistance a la traction et son evolution dans le temps, le coefficient de dilatation thermique, ainsi que I'influence des reactions chimiques sur les deformations. Tons ces facteurs dependent fortement de Tage, de la temperature, du type de ciment, et de la composition du beton. De fagon realiste, il n'est que grossierement possible d'evaluer ces influences. Un progres important a ete accompli avec des modeles permettant de mieux evaluer les proprietes des materiaux. Ces modeles requierent egalement des hypotheses sur les conditions aux limites mecaniques ainsi que la prevision des temperatures in situ.

De nouvelles methodes fort prometteuses ont ete developpees en France et au Japon afin de mesurer in situ les contraintes dues a ces deformations empechees ou genees. La comparaison de resultats d'essais de laboratoire ou sur le terrain avec les resultats de calculs est la source de plus amples progres dans ce domaine.

Plus recemment, le beton a haute resistance s'est revele etre un materiau souvent plus sensible a la fissuration au jeune age. Ce phenomene n'est pas uniquement une consequence de la chaleur d'hydratation. Le retrait endogene du a I'auto-dessiccation et aux reactions chimiques de la phase sulfatique pent egalement jouer un role important.

La fissuration inattendue d'un beton de structure ou d'un beton massif ne peut pas toujours etre attribuee a I'inexperience de I'ingenieur de chantier. De ce fait, la connaissance limitee de nombreux problemes dans ce domaine a fortement suscite un effort de recherche dans le monde entier. En 1989, la Reunion Internationale des Laboratoires d'Essais et de Recherches sur les Materiaux et les Constructions a cree la Commission Technique 119 "Prevention de la fissuration d'origine thermique dans le beton au jeune age". Outre I'echange de points de vue et d'experience, le role de cette commission est de preparer un rapport sur I'etat de I'art et d'elaborer des recommandations pour les methodes d'essai suivantes:

- Determination de la chaleur d'hydratation par calorimetrie semi-adiabatique.
- Determination de la chaleur d'hydratation par calorimetrie adiabatique.
- Mesure en laboratoire des contraintes dues aux deformations empechees a I'aide du banc de fissuration.
- Mesure in situ des contraintes dues aux deformataions empechees.

Le coUoque international ayant lieu a Munich du 10 au 12 octobre 1994 s'inscrit dans une periode dynamique de developpement. Un grand nombre de questions ayant trait a la prevention de la fissuration precoce ont ete resolues et les resultats sont disponibles pour la mise en pratique. Nos suggestions de reponse a d'autres questions sont fondees sur des resultats d'essais ou derivent de considerations theoriques. H convient encore de determiner comment ces methodes peuvent etre employees le plus efficacement possible, si celles-ci necessitent un developpement

plus pousse ou si elles ne sont que destinees a stimuler des recherches plus approfondies.

Afin de limiter Tetendue du domaine, le retrait de dessiccation, le role de I'armature en acier pour limiter I'ouverture des fissures, ainsi que I'emploi avantageux de la precontrainte ne sont pas traites dans ce coUoque.

La prevention de la fissuration au jeune ^ge est un exercice qui requiert des connaissances theoriques, un sens aigu de I'ingenierie et une vaste experience. En outre, les ingenieurs consciencieux doivent s'assurer que toutes les mesures necessaires sont mises en oeuvre.

J'espere que ce coUoque marquera une etape des progres de notre domaine.

Rupert Springenschmid
Munich, Juillet 1994

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PART ONE

HEAT OF HYDRATION

(La chaleur d'hydratation)

1 NUMERICAL AND EXPERIMENTAL ADLVBATIC HYDRATION CURVE DETERMINATION

E.A.B. KOENDERS and K. van BREUGEL
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Abstract
Adiabatic hydration curves of concrete mixes are used as input for computer programs utilized for the calculation of the temperature distribution in hardening concrete. At present no standardised test equipment has been specified with which adiabatic curves should be determined. A comparison of test results obtained with different adiabatic calorimeters has revealed a substantial scatter and has evidenced the need for a thorough analysis of the cause of this scatter. Results of such an analysis are presented in this paper. Problems encountered in adiabatic testing are dealt with. A computer-based numerical model is presented, with which hydration curves can be calculated as a function of the clinker composition of the cement and the mix composition of the concrete.
Keywords: Adiabatic Calorimetry, Heat of Hydration, Modelling of Heat of Hydration, Numerical Simulation.

1 Introduction

For the evaluation of thermal problems in hardening concrete several computer programs have been developed. For most of these programs the adiabatic hydration curve of the concrete mix is an essential part of the input. Until recently the determination of these hydration curves had to be done experimentally with adiabatic calorimeters. A comparison of multi-laboratory test results has revealed a substantial scatter. This scatter originates from differences in the steering algorithms used for adjusting the temperature of the control medium, the size of the sample and the type of the mould. A practical drawback of experimental testing is that each change of the concrete mix requires an other test. This is time consuming and does often not fit in the tight time schedule of a project.

A new trend in materials science is to simulate the hardening process in cement-based materials numerically as a function of the dominant rate controlling parameters. The advantage of such numerical models would be, apart from saving time, that pure adiabatic condition can be simulated.

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In the following some of the basic features of a numerical simulation program will be presented. In the model called HYMOSTRUC, the acronym for Hydration, Morphology and STRUCtural development, hydration curves can be predicted as a function of the characteristics of the cement and the concrete mix. Due attention is given to the accuracy with which numerical predictions are possible and this as compared to the accuracy achievable with experimental tests.

2 Numerical simulation of hydration and microstructural development in cement-based materials

2.1 Basic features of the simulation model

In the computer-based simulation model HYMOSTRUC hydration curves are calculated as a function of the particle size distribution and the chemical composition of the cement, the water/cement ratio CJQ and the reaction temperature. Unlike most previously proposed models the effect of physical interactions between hydrating cement particles on the rate of hydration of individual cement particles is modelled explicitly. For numerical evaluation of the interaction between hydrating particles due attention had to be given to the stereological aspect, i.e. the spatial distribution of the cement particles in the paste.

2.2 Stereological aspects

In HYMOSTRUC cement particles are considered to be distributed homogeneously in the paste. An arbitrary particle is considered to be located in the centre of a cell "**Ix**" (Fig. 1). A cell "I^" is defined as a cubic space in which the central particle has a diameter x and further consists of 1/N^ times the original water volume and 1/N^ times the volume of all particles with a diameter smaller than x *jim.* is the number of particles with diameter x /xm in a certain paste volume.

For the assumed homogeneous spatial distribution it is relatively easy to determine the amount of cement found in a fictitious shell with thickness d surrounding a cement particle with diameter x *fim.*

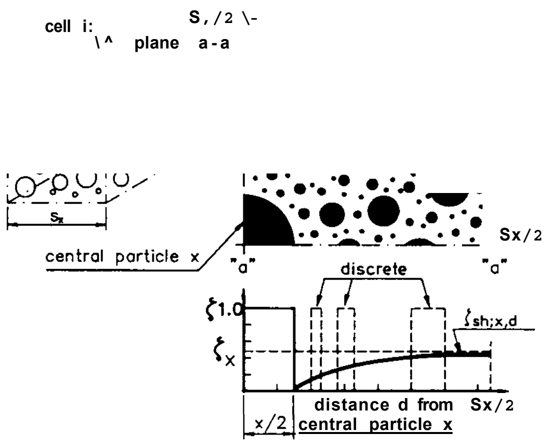

Fig. 1. Cell concept and shell density factor **fsh;x,d'**

For this purpose a shell density factor **fsh;x,d** been defined (Fig. 1, bottom part). Going from the periphery of particle x in outward direction the shell density gradually increases from zero at the outer periphery of the anhydrous particle to the cell density f^.

2.3 Particle expansion and particle interaction mechanism

In order to obtain a workable algorithm for the determination of the interaction between hydrating and expanding cement particles the following assumptions were made:

a. Particles of the same size hydrate at the same rate.
b. The ratio ***v*** between the volumes of the reaction products and the reactant decreases with increasing temperatures.
c. Reaction products precipitate in the close vicinity of the cement particles from which they are formed.
d. Dissolution and expansion of the hydrating cement particles occur concentrically.

The simplifications introduced here are, to a large extent, inherent to the statistical approach that was adopted. For a more extensive justification of these simplifications reference is made to (van Breugel).

The expansion and interaction mechanisms are schematically shown in Fig. 2. On contact with water an arbitrary cement particle x starts to dissolve under formation of reaction products. These product are formed partly inside and partly outside the original surface of the particle. In this way cement particles exhibit an outward expansion, thereby making contacts with neighbouring particles. At a certain time, say t**j**, the depth of penetration of the reaction front is 6in**.x,j** with a corresponding degree of hydration of this particle **Qtj^ j**. Further hydration of this particle goes along with further expansion and embedding of neighbouring particles. The encapsulation of other particles causes and extra

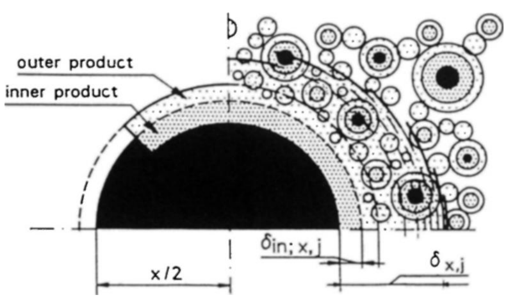

Fig. 2. Interaction mechanism for expanding particles. Left part: free expansion, formation of inner and outer product. Right part: embedding of particles, several stages.

expansion and, as a consequence of this, the encapsulation of even more particles. For this mechanism of continuous expansion and embedding of particles an algorithm has been developed with which the stereological aspect of structural development can be quantified.

2.4 Rate of penetration of the reaction front

The rate of penetration of the reaction front in an individual cement particle x at time tj is computed with the "basic rate formula" viz. (in a reduced form):

^ ™ 2 ^ = Ko(.) * [(^ ^ i l i)] - F{cUo,T(ce)} (1)

with increase of the penetration depth in time step Atj^i,' KQ (.) the basic rate factor in ptm/h, ^*tr*^-"^ transition thickness in *[im,* being the total thickness of the product layer 6^ j at which the reaction changes from a phase-boundary reaction (X = 0) into a diffusion controlled reaction (X = 1). The factor f{CL)O/T(a(t))} is a function of the w/c-ratio and the actual reaction temperature.

In an extensive evaluation program, in which a relationship between the two model parameters KQ(.) and and the clinker composition of the cement was investigated, the following expressions have been established for the default values of afore mentioned parameters:

Ko(C3S) = 0.02 + 6.6 10"^ ^ [038%]^ *[fim/h]* (2)

and:

d^^{C2S) = -0.02 * [C2S%] +4 *[fim]* (3)

For Ko(C3S) a standard deviation would hold of 0.008 /xm/h. Upper and lower bound values for the transition thickness (5t^(C2S) can be obtained by increasing or decreasing the default values according to eq. (3) by about 0.5 /xm, respectively. For ***13^*** a default value of 2 was found. For an extensive discussion about the effect of the w/c ratio and the way in which temperature effects have been allowed for in the model reference is made to (van Breugel).

By adding the amounts of hydrated cement of the individual particle fractions and dividing them by the original amount of anhydrous cement the overall degree of hydration ***01 it)*** at time t is obtained.

2.5 Prediction of adiabatic hydration curves

Assuming a linear relationship between the degree of hydration o^(t) and the liberated heat of hydration and that all the liberated heat is used for heating up of the hydrating sample, adiabatic hydration curves can be computed. In these calculations the specific heat of the concrete is considered to be a function of the degree of hydration.

3 Experimental determination of adiabatic hydration curves

3.1 Adiabatic calorimetry: Factors affecting the accuracy

Factors which are of paramount importance in view of the accuracy of adiabatic testing are:

a. Accuracy of the temperature measurements.
b. The steering algorithm (implicit or explicit)
c. The type of the moulding
d. The size of the sample
e. Heat dissipation to the environment and the way this heat loss is compensated.

The effect of these parameters can be investigated experimentally by varying them one by one or with the help of computer programs with which the adiabatic test conditions can be simulated. The latter method has the advantage of being more flexible and quicker than experimental testing. Numerical analyses of the effects of the control precision, type of steering algorithm, type of moulding and size of the sample were carried out with a 3-D computer program called ASA3D (Adiabatic Sensitivity Analysis - 3 Dimensional).

3.2 Adiabatic test device specifications

Up till now several adiabatic calorimeters have been developed. An inventory of adiabatic and semi-adiabatic calorimeters is being prepared by RILEM (Wainwright). Pending the work of RILEM and still in the absence of standardised test equipment, a new experimental set-up has been developed. A schematic view of this test set-up is shown in Fig. 3. The test specimen is a cube with a volume of 3.375 litre. The material of the moulding is 20 mm thick polyethylene (PE) with a thermal conductivity coefficient of X = 0.17 W/mK. Instead of a PE also a steel mould can be used with X = 50 W/mK. The control medium is water.

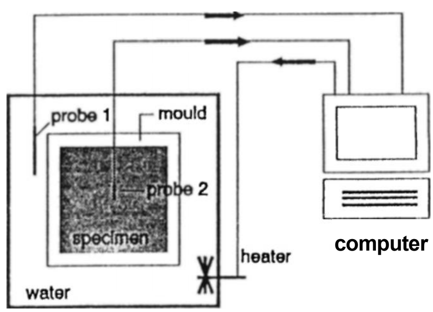

Fig. 3 Adiabatic test device Schematic.

For adjusting the temperature of the control medium a heater is used with a capacity of 3000 Watt. Maximum acceleration 60°C/h. With this heater a control precision of +0.05°C can be reached. This relatively high capacity is necessary for controlling the boundary condition if a rapid hardening material is tested, for example high strength concrete.

Temperature measurements are carried out with PTIOO elements with a precision of, for the time being, +0.05°C (probe 1) and ±0.01°C (probe 2).

3.3 Steering algorithms

Adjustment of the temperature of the control medium can be done by explicit or implicit steering.

In the explicit steering method the temperature of the control medium is adjusted step-wise to the temperature in the core of the sample at the end of each time step. During this step some heat may dissipate from the sample.

In case of implicit steering temperature adjustment is done taking into account the rate of reaction in the preceding time step by using extrapolation techniques. During each time step the temperature is adjusted gradually. Dissipation of heat to the environment is prevented almost completely in this way, which will result in a higher accuracy.

4 Results

4.1 Evaluation of variations in steering algorithm

The effects of variations in the type and accuracy of the steering algorithm on adiabatic hydration curves have been evaluated numerically and experimentally. Fig. 4 shows the results of numerical simulations. An adiabatic curve obtained with implicit steering is compared with curves obtained with explicit steering. In the latter case the control precision was adjusted at 0.05°C and 0.2°C, respectively.

4.2 Numerical evaluation of the effect of type of mould

In Fig. 5 the effect is shown of the type of mould, i.e. steel and polyethylene, on adiabatic hydration curves. The numerically obtained curves are computed for a simulated temperature control precision of ±0.2°C (explicit steering). Explicit steering is chosen here so as to enforce a noticeable effect of the type of mould. Experimental results are

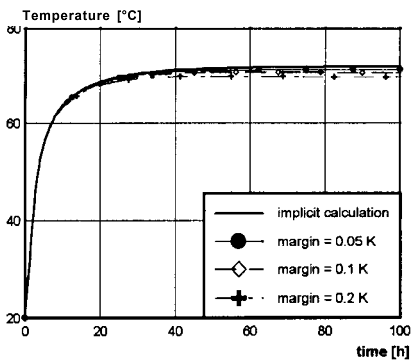

Fig. 4. Influence of steering accuracy of the adiabatic test device.

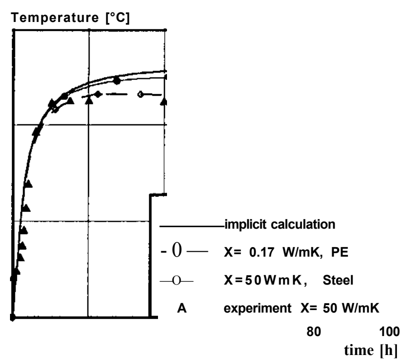

Fig. 5. Effect of type of mould on adiabatic hydration curves (explicit steering).

inserted in the figure as well. The two computed curves are compared with an adiabatic curve obtained by implicit steering. It appears that in case of explicit steering the type of mould can have a substantial influence. When an explicit steering algorithm is used reliable results can only be achieved if a mould with a high insulation capacity is used, by increasing the control precision from 0.2°C to, for example, 0.01°C, or by increasing the size of the sample.

4.3 RILEM Round Robin test

With the aim to get a better judgement of the multi-laboratory scatter of adiabatic hydration curves a Round Robin test has been carried out. Cement, sand and aggregate were provided by one of the participating laboratories. Tests were carried out with mixes with w/c ratio = 0.6. Cement, sand and aggregate were mixed in proportion 1 : 2.5 : 3.5. The calculated clinker composition of the cement was: C_3S = 53% and C_2S = 16%. Specific surface (Blaine) = 372 m^2/kg. The nominal initial temperature of the mix was T_0 = 20°C.

In Fig. 6 the temperatures reached after 200 hrs are presented. The mean value of the temperature at that time was 66.8°C. The difference between maximum and minimum temperature is 8.8°C. The multi-laboratory standard deviation was computed at 3.8°C. Assuming a normal distribution of the test results the 5% upper and lower bound temperatures at 200 hrs are 73.0 and 60.6°C, respectively.

4.4 Adiabatic hydration curves predicted with HYMOSTRUC

Theoretical adiabatic hydration curves calculated with HYMOSTRUC are inserted in Fig. 6 as well. The solid curve, which has been calculated with the default values of the model parameters **KQ(C3S),** *b^^{C2^)* ^"^^ I^I/ represents the mean value

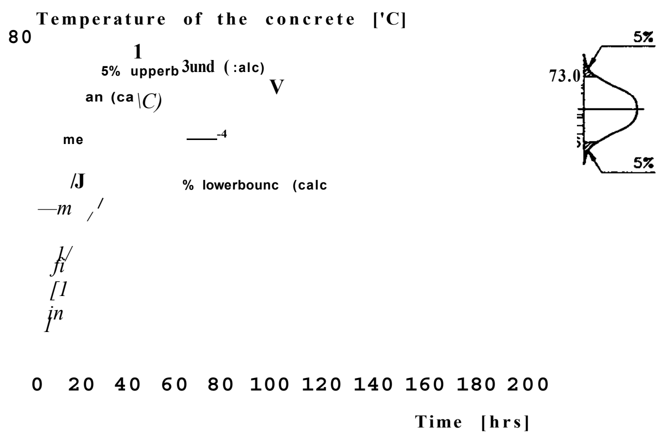

Fig. 6 Adiabatic test data obtained in round robin test compared with numerically predicted hydration curves.

of the adiabatic curve. The predicted temperature at 200 hrs is 68.5°C, which is 1.7°C higher than the mean value obtained in the Round Robin test. The predicted upper and lower bound curves are presented with dashed lines. The calculated 5% upper and lower bound temperatures at 200 hrs were 70.5 and 65.5°C. This range of 5°C, in which 90% of the adiabatic results are expected, is of the same order of magnitude as the corresponding range in the Round Robin test.

A comparison of the predicted solid curve with one of the measured adiabatic curves (dotted line) reveals that in the early stage of hydration these curves coincide quite well. After some time they start to deviate. One of the reasons for this deviation must most probably be attributed to inadequate correction of heat losses in the test device.

5 Conclusions

The results of adiabatic tests depend on, among other things, the control precision of the test device, the size of the sample and the type of the mould. Numerical simulations of adiabatic tests with the computer program ASA3D evidenced that substantial differences, up to 8°C after 100 hrs hydration, are to be expected due to differences in afore mentioned parameters. In a Round Robin test, in which seven laboratories participated, a difference between maximum and minimum temperature of 8.8°C after 200 hrs was found with a standard deviation of 3.8°C. This result was in good agreement with the results of the numerical simulations.

With the computer program HYMOSTRUC adiabatic hydration curves of the same mix as used in the Round Robin test were predicted as a function of the cement type and mix composition. The predicted mean adiabatic curve and the 5% upper and lower bound curves turned out to be in good agreement with the measured curves. For practical purposes numerically predicted adiabatic hydration curves appear to be sufficiently accurate to serve as input curves for computer programs for temperature calculations in hardening concrete.

Acknowledgment The authors wish to thank the RILEM Task Group TC119 for permission to make use to the results of the Round Robin test mentioned in this papers.

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2 THERMAL AND MECHANICAL MODELLING OF YOUNG CONCRETE BASED ON HYDRATION PROCESS OF MULTI-COMPONENT CEMENT MINERALS

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Abstract
This paper aims at developing a predictive method on both heat generation and associated evolution of strength for young concrete. Mineral compounds of cement clinker and pozzolans are focused and the hydration degree of them are computed step by step with modified Anhenius's law of chemical reaction. The specific free water and calcium hydro-oxide, that is an activator for pozzolans, are assigned as state variables representing chemical environment of pore solution. The effect of fly ash on cement and slag hydration retarded by the adsorption of calcium ion is taken into account. The strength and instantaneous stiffness of hardening concrete are related to the accumulated heat of each mineral compound and versatility of the mechanical model proposed is verified under varying temperature environments.
Kevwords: Clinker Minerals, Slag, Fly ash. Hydration, Heat Generation

1 Introduction

For making thermally induced cracks avoidable, evaluation of thermal crack risk is required at design stage. Here, the hydration heat of cement in concrete has to be modeled as a source of temperature rise. Meanwhile, the strength and stiffness evolution of concrete should be also predicted for examining thermal crack occurrence. The heat generation and varying mechanica properties of concrete at early ages are strongly related to the hydration degree of each mineral compound consisting of cement. Thus, it is desired to consistently predict them with a unified concept concerning hydration progress of cement. This approach brings an engineering advantage to enable sensitivity analysis with respect to mix proportion of concrete and sorts of cement and pozzolans with different chemical compositions and blended ratios.

To meet the engineering challenge stated above, the model has to be applicable to wider variety of clinker compositions of Portland cement and replacement by pozzolans under varying temperature. On this line, this paper proposes "multi-component model" for hydration heat anc the evolution of strength based on the Arrhenius's law of chemical reaction. Within this frame, interacting heat generation among constituent minerals is coherently treated in terms of free water and calcium hydro-oxide solution.

2 Interaction between Portland cement clinkers and pozzolans

Pozzolans react with calcium hydro-oxide as an activator which is produced by hydration of clinker minerals. Then, it is crucial to accurately appraise interaction between cement clinker

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and pozzolans for versatile modeling. For this purpose, conduction calorimetry test was conducted for taking some knowledge on effects of pozzolans replacement. The conduction calorimetry of cement paste in various mix proportions of blended materials was at 50% water cement ratio and 20°C constant curing temperature. The physical properties and chemical compositions of powder materials are given in Table 1.

Table 1. Chemical components and physical properties of cement and pozzolans used.

	specific gravity	Blaine (cmVg)	IngLos (%)	Si02	CaO	A1203	FezOs	MgO	SO3	FeO	K2O	NaaO
OPC	3.15	3290	0.6	22.1	64.1	6.2	2.6	1.4	2.1	-	0.5	0.31
slag	2.89	4000	0.0	31.3	43.3	13.2	-	6.0	2.0	0.3	-	-
fly ash	2.33	3440	0.4	48.1	8.8	27.6	5.3	-	-	-	-	-

note) chemical components : weight percentage

2.1 Reaction of slag in mixed cement

Typical heat generation rates of cement with slag are shown in Figure la. In general, two peaks of heat rate are seen in time domain. The authors attempted to extract heat released from slag existing in the mixed cement paste by subtracting heat of cement clinker (pure OPC test). Here, let us assume independency of cement reaction which is not affected by slag. With this hypothesis, heat generation rates of slag in the mixture normalized by the content of slag can be obtained as shown in Figure lb.

The heat generation rates are similar regardless of the blended contents of slag till the peak. While, there is a tendency that heat generation is quickly descending after the peak of hydration rate when higher replacement is performed. It can be simply presumed that slag can react independently under a condition where calcium hydro-oxide is sufficiently released from cement, but at the higher replacement of slag, the reaction of slag is stagnant because of shortage of calcium hydro-oxide in pore solution. In other words, the assumption that cement clinker minerals react with less influence brought by slag is acceptable to the hydration heat model. On the contrary, the influence of cement hydration on the reaction of slag is crucial for constructing multi-component modeling.

2.2 Fly ash in mixed cement

Heat generation of cement with fly ash at various mixing ratio is shown in Figure 2a. It is clear that the maximum heat generation rate is reduced with overall delay of heat in accordance with replacement of fly ash. Provided the assumption stated in slag, computed heat from fly ash was found to get negative. This indicates that mutual interaction between cement and fly ash is predominant unlike the "one way" interaction between cement and slag.

As generally known, fly ash retards the hydration of Portland cement especially at early stage of hydration and makes dormant period longer. When cement is mixed with fly ash, Ca^"" ion concentration in pore solution is depressed due to the removal by aluminum ions on the fly ash surface. This phenomena is explained such that fly ash surface acts like calcium sink. The depression of concentration retards the formation of calcium rich surface layer on clinker minerals which are precursor of reactivity. This will involve in delaying the formation of Ca(0H)2 and C-S-H gel nucleation.

Heat generation of pure slag and binary mixture of slag and fly ash were measured with addition of calcium hydro-oxide as shown in Fig.2b. This experiment aims at clarifying interaction between slag and fly ash without cement clinker since the case of triple mixture is required to be modeled from an engineering view point.

The binary pozzolans release heat as shown in Figure 2b. Pure slag with calcium hydro-oxide has the peak of heat generation around 20 hours after mixing. This is nearly the same as the case of slag-OPC composite. It is observed that fly ash delays heat generation of slag as well as cement. The retardation effect of fly ash on the slag hydration can be concluded.

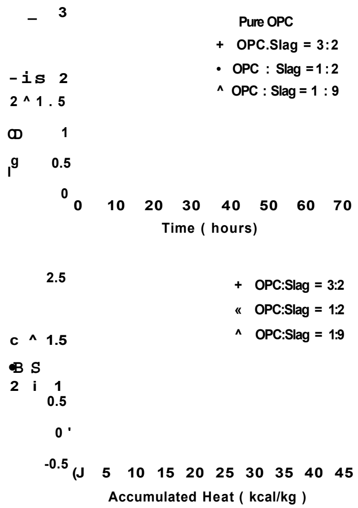


Fig.1 Conduction calorimetry of mixmre and pure slag extracted.

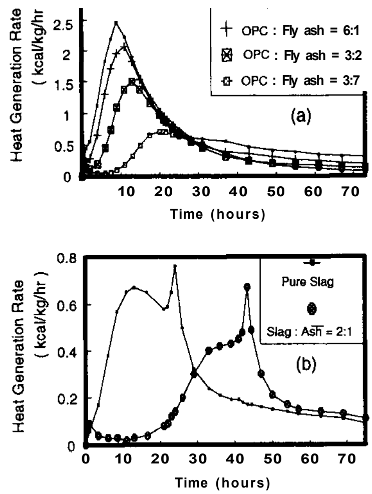


Fig.2 Effect of fly ash on conduction calorimetry.

3 Multi-component model of cement hydration heat

3.1 Modeling

The specific heat rate of cement consists of constituent mineral based heat rates. The authors take up four clinker minerals (aluminate **C3A,** ahte **C3S,** felite **C4AF,** belite C2S) and five patterns of reactions for Portland cement (Kishi et al.). In the case of mixed cement, reactions of slag and fly ash are combined with cement clinker minerals as,

$$H = P_{mono} H_{mono} + P_{C_3A} H_{C_3A} + P_{C_3S} H_{C_3S} + P_{C_4AF} H_{C_4AF} + P_{C_2S} H_{C_2S} + P_{FA} H_{FA} + P_{SG} H_{SG} \tag{1}$$

where, p_i = mass ratio of i-th component, H_i = specific heat rate of i-th component, subscript 'mono' represents the transformation of ettringite to monosulfate.

Suzuki et al. reported that Arrhenius's law can be extended to the composite with different chemical reactions of cUnker minerals by adopting variable mean activation energy uniquely specified in terms of the accumulated heat of cement. As the hydration of cement are classified into mineral compounds, the activation energy of their reactions are assumed constant one by one. According to Arrhenius's law, the temperature dependent heat rate of reaction yields.

$$\exp\left[-\frac{E_i}{R}\left(\frac{1}{T}-\frac{1}{T_0}\right)\right] \tag{2}$$

where, = activation energy of i-th component reaction, ***R*** = gas constant and ***H-*** = referential heat rate when temperature is ***To*** (=293 K).
The interaction among constitutive compounds is expressed as,

$$H_{,,,} = A\text{-}\hat{}rrF(lH.dt) \tag{3}$$

where, pi represents the reduction of probability of the contact between unhydrated compound and free water. The factor ***Qi*** represents retardation of cement and slag reaction caused by fly ash. The factor y represents reduction of hydration concerning pozzolans due to shortage of calcium hydro-oxide. The parameter (3/ is associated with increasing thickness of cluster around unhydrated compounds and the decreasing free water during hydration as.

$$/? = 1-\exp \frac{\hat{}ree}{r} \tag{4}$$

where, ***r*** and s = material constants, ωfree = free water which can be consumed by further reaction, T)/ = thickness of cluster around unhydrated compound (Kishi, et al) as.

$$\hat{} = 1 - 1 - \tag{5}$$

The retarding effect caused by fly ash on other compounds is represented as,

$$Q, = 1 + \exp(-/z \cdot \quad J - \exp(-/z \cdot p,\hat{} \cdot 4\hat{}) \tag{6}$$

where, ***h*** - material constant, ***p^A*** = mass ratio of fly ash, ()**FA** = hydration degree of fly ash. The reduction of pozzolans reaction caused by shortage of calcium hydro-oxide is formulated as.

$$1-0.06\exp i\ -5.: \quad --05 \tag{7}$$

where, ***k*** = material constant, ***Fch*** = available $Ca(OH)_2$ in the solution, ***Rch*** = $Ca(OH)_2$ necessary for reaction of pozzolans. The factor i3 expresses that consumption of $Ca(OH)_2$ by fly ash is reduced when concentration of $Ca(OH)_2$ in pore solution gets less as follows.

$$P\{PFA- \quad \hat{}FA)-\hat{} \tag{8}$$

where ***u*** and v = material constants. The multi-component model on heat generation is formulated. By solving Eq.1- Eq.8 with the thermodynamic energy conservation, temperature transition of concrete can be obtained in time and space domains (Harada, et al).

3.2 Verification of heat generation model

The material constants and referential heat rate of each compound are shown in Table 2 and Figure 3. These values were identified through adiabatic temperature rise tests and data back analysis processing. Theoretical specific heat is given as final heat generation of each clinker

Table 2 material parameters *k=5,* **M=0.35, V=0.25 for pozzolans only, E/R:** ***K,*** **:** ***cal/g***

	C3A	C3S	C2S	C4AF	ettringite	slag	fly ash
E/R	6500	6000	3000	3000	6000	5000	5000
r	2	2	2	2	2	3	2
s	2.5	2.5	2.5	2.5	2.5	-	2.5
JH	207	120	62	100	330	110	50
h	1.5	1.3	0.9	1.2	-	0.95	-

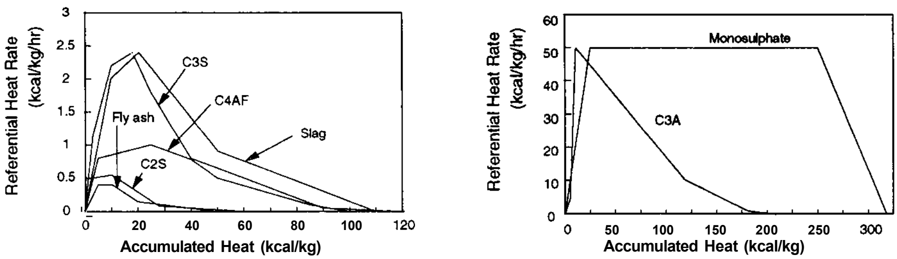


Fig.3 Assumed referential heat generation rate of cement clinkers and pozzolans.

Table 3 Concrete mix proportion, cement mineralogical analysis.

	c	W	S	G	PC	SG	FA	C3A	C3S	C2S	C4AF	SO3
mix1	400	157	658	1129	100	-	-	10.4	47.2	27.0	9.4	1.85
mix2	400	157	663	1129	100	-	-	3.7	44.4	33.7	12.5	1.85
mix3	260	140	740	1125	75	-	25	4	26	53	11	1.7
mix4	260	140	738	1120	35	45	20	2	24	53	12	1.7
mix5	260	140	740	1122	40	30	30	5	15	65	11	1.6
mix6	260	140	774	1125	25	75	-	3	44	34	12	2.5
mix7	260	140	748	1133	70	30	-	3	7	75	9	2.5
mix8	260	140	738	1117	30	50	20	4	22	57	11	2.0
mix9	260	140	738	1120	25	55	20	5	36	40	12	1.4
mix10	260	140	718	1137	32	48	20	2	24	53	12	1.3

PC : Portland cement (ordinary : OPC and moderate heat: MC), SG : slag, FA : fly ash

mineral. The adiabatic temperature rises of two Portland cement mixtures and eight mixtures with slag and fly ash as low heat type are adopted for verification as shown in Table **3.** Application of the proposed model as shown in Figure 4 are ensured for different chemical components and temperature rises of mixed cements adopted are fairly predicted.

4 Strength development - generalized mineral water ratio law

4.1 Experiment of strength development

For making strength evolution model of young concrete with temperature rise, compressive loading tests of mortars were conducted under two patterns of temperature histories. Figure 5 shows temperature history patterns under which test specimens were cured and tested. Specimen was 5cm x 10cm cyhnder and cured with sealing condition. Pattern 1 is supposed the analogy of temperature history of concrete in a massive structure at early ages. For comparison in pattern **2** the curing temperature was kept at room condition for about a week and then temperature was elevated for acceleration of hydration. The beginning of temperature rise in pattern **2** is slightly different in each mixture (MC : 7.4days, SG+MC : 6.8days, FA+MC : 9.1 days). Mix proportions are shown in Table 4. The hydration degree of each mineral constituents are computed by the multi-component model of heat generation.

The evolution of strength at pattern 1 and pattern **2** with respect to time are remarkably different though the strength at final stage are almost the same as shown in Figure 7.

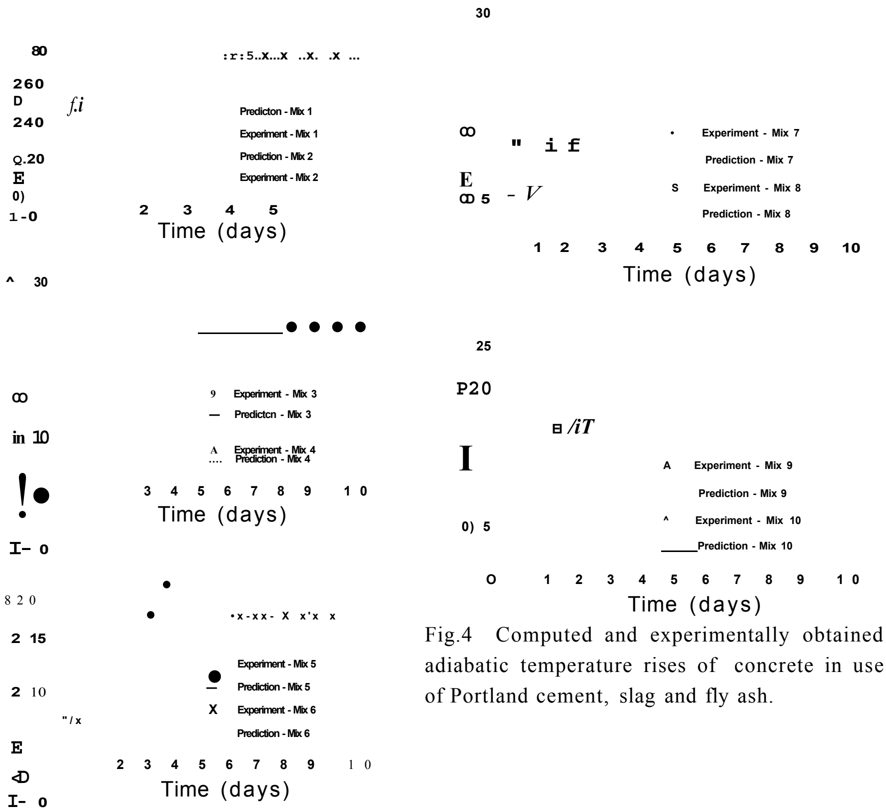


Fig.4 Computed and experimentally obtained adiabatic temperature rises of concrete in use of Portland cement, slag and fly ash.

The approach based on the hydration heat model is essential for modeling the evolution of strength. The hydration degree of each mineral constituents is not common at the nearly same strength or accumulated heat between two patterns because of the difference in thermal activity of them. This indicates that mineral compounds consisting of cement should be formulated with multi-component concept in the modeling of the evolution of strength.

4.2 Modeling

The authors (1993) reported that clear bi-linear relation was seen between the strength and the entire hydration level of cement which was computed by multi-component model and indicated the possibiUty to be able to estimate the evolution of strength with the hydration degree of cement. But no unique relation between strength and entire hydration level of whole cement is not obtained. The relation of them should be formulated according to mix proportion and used powder materials. As the model has to be applicable to any combination of materials, multi-component concept based on the mineral constituents was adopted for the strength evolution model. Four constitutive minerals (C_3S, C_2S, slag, fly ash) are taken as the effective components for the evolution of strength, and C_3A and C_4AF are assumed negligible in the proposed model. Then, the compressive strength is expressed in terms of the total differential equations as.

S>]dS:. df,^=25dQ,s +27^25^+40^G^

0= HJfl •(i H4t (9)

where, f'_c = compressive strength (MPa), ϕ_i = hydration degree indicated by the accumulated heat normalized by final heat generation. The contribution of constitutive minerals are individually formulated and the concept of cement water ratio in the compressive strength is extended to each component in the above equation.

4.3 Verification

The experimental and analytical results in terms of accumulated heat of cement (See Figure 6) fairly coincide with each other. The relations of the strength evolution and the accumulated

Table 4 Mix proportion of concrete used

mix	MC	SG	FA	W	G	S
A	761	-	-	181	808	720
B	360	360	-	181	808	720
C	478	-	192	181	808	720

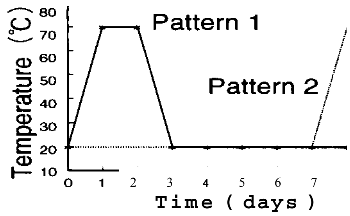
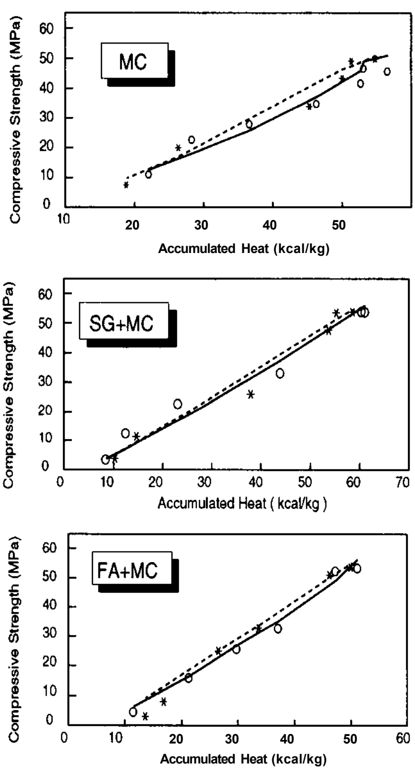
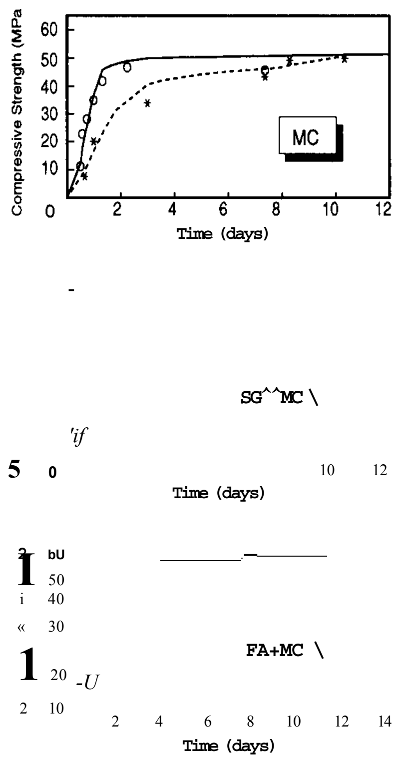


Fig.5 Induced temperature histories





Prediction Pattern 1	Experiment Pattern 1	Prediction Pattern 2	Experiment Pattern 2

Fig.6 Relation of accumulated heat and compressive strength of concrete.

Prediction Pattern 1	Experiment Pattern 1	Prediction Pattern 2	Experiment Pattern 2

Fig.7 Strength development of concrete

heat at the temperature histories of both pattern 1 and pattern 2 are similar. In mathematical view of modeling, these relations do not necessarily match. Figure 7 shows the results with respect to the curing time, in which fair agreement can be seen between experimental and analytical results as well. It is emphatic that the proposed model can rationally deal with the evolution of strength with respect to time at any temperature history.

In general, the concrete strength varies in accordance with the curing temperature and it is reduced at longer age under the elevated temperature especially experienced at the early stage. Within this study, however, the evolution of strength can be computed only by dealing with the composition of already hydrated minerals as the hydration degree though it is not clear how the structural formation of the hydrated products affect the evolution of strength. Compared with the maturity model available recently, the proposed strength model based on chemical components of cement and pozzolans has physical generality and equivalence.

5 Conclusions

The hydration heat model of cement in concrete was proposed as the multi-component one for clinker based Portland cement as well as their mixtures with slag and fly ash. The interaction among powder materials was suitably taken into account through the conduction calorimetrie study and the proposed model was experimentally verified. The strength evolution model was also proposed in terms of the accumulated heat of mineral constituents, which was computed by the hydration heat model. The models of heat generation and strength evolution were associated with each other under the unified concept concerned with hydration progress of mineral compounds. Further improvement is needed through the clarification of regulative factors having influence on the hydration of chemical compounds.

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PART TWO

PREDICTION OF TEMPERATURE DEVELOPMENT

(Calcul des champs de temperatures)

3 PREDICTION OF TEMPERATURE DISTRIBUTION IN HARDENING CONCRETE

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Abstract
This paper presents the development of a computer model to predict the temperature distribution in hardening concrete. A two-dimensional finite element thermal analysis is used to model the transient heat transfer between the concrete and the environment by taking into account the cement type and content, boundary and environmental conditions including solar radiation and artificial heating, if any. The output is the temperature distribution in the member at any moment. This is the key information for concrete maturity estimate and thermal stress assessment. The model can also be used in construction planning, such as designing the necessary curing measures to achieve the desired strength at a specific age or to control the temperature differentials in the structure to eliminate thermal cracking. Comparisons with data from various field temperature measurements confirm the analytical work.
Kevwords: Temperature, Hardening Concrete, Hydration Heat, Heat Transfer, Thermal Analysis, Temperature Prediction.

1 Introduction

The prediction of the temperature distribution in hardening concrete is of great interest to designers and contractors for several reasons: thermal cracking and deformation control or prevention, and evaluation of the concrete strength development. The importance of temperature effects in hardening concrete has inspired extensive research worldwide in recent years (Tsukayama 1974, Thurston et al 1980, Breitenbiicher 1989, Laube 1990, Wang 1994).

Temperature change is the cause of any thermal stress and thermal cracking. In concrete the hydration of portland cement is an exothermal process, releasing up to 500 Joules of heat per gram of cement (Neville 1981). The relatively low thermal conductivity of the concrete mass delays the heat dissipation into the surroundings, resulting in a substantial temperature rise in large members at the early ages. The concrete may also gain heat from the environment such as solar radiation, and from heat curing. When concrete has developed some apparent strength, any temperature change will cause stress and deformation, and often, cracking in structures. The only situation when temperature variation does not generate stress is in a statically determinate structure where the temperature varies linearly across the

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section. Therefore, the prediction of temperature distribution and history in hardening concrete is essential in order to estimate the thermal stress and strain as well as to prevent thermal cracking.

Like all chemical reactions, cement hydrates faster at higher temperature. After the concrete mix is cast, the only factor in its strength development is temperature. In hardening concrete, the temperatures are normally different at different locations of a structure at any time. For a typical thick member, the concrete strength gain at the core is much faster than near the surface during the early ages because of the temperature differences. An accurate estimate of concrete strength development, which is critical in determining the time of form stripping and/or prestressing, requires the prediction of the history of temperature distribution in the concrete. Failure to determine the concrete strength accurately in the field may either delay the construction, cause local damage or even collapse of the structure.

This paper presents a computer model to predict the temperature distribution in hardening concrete at any time. A two-dimensional finite element thermal analysis is employed to model the transient heat transfer between the concrete and the surroundings as affected by the concrete mix, thermal boundary and environmental conditions. Because of the time and temperature dependent nature of the hydration heat rate and boundary heat transfer conditions, the solution of the problem requires the step-by-step integration in the time domain.

The details of estimating concrete maturity and strength as well as thermal stress from the temperature distribution and history are presented elsewhere (Wang 1994, Wang and Dilger 1994).

2 Heat Transfer between Concrete and the Environment

At any time, the temperature distribution in a concrete cross section is the dynamic heat balance between the heat generated inside the concrete and the heat loss to, or gain from, the surroundings. In hardening concrete, the heat generated inside is hydration of the cement. For most of the actual structures whose length is much larger than the width or thickness, thermal analysis can be treated as a two-dimensional problem by assuming that the temperature distribution does not vary along the length.

The temperature distribution within a two-dimensional body or the transient heat flow within the boundaries of the body is governed by the well-known Fourier's Law (Holman 1986):

$$+ g = p\ c\ |\ | \qquad (1)$$

where, T = temperature, °C,
t = time,
k = thermal conductivity, W/(m°C)
q = rate of heat generated inside the body, W/m^
p = density of the material, kg/m^
c = specific heat of the material, kJ/(kg°C)
x,y = Cartesian coordinates in x,y directions.

There exist basically two types of boundary conditions for Eq.1. The first one is that the temperature along the boundary or a portion of the boundary is known, and the second one is that the energy transfer through the boundary is known. For ordinary engineering structures, the second type of boundary condition normally occurs. That is, the heat exchange between the concrete body and the environment in the forms of solar radiation, thermal radiation, convection, etc. needs to be evaluated in order to solve Eq.1 to obtain the temperature distribution in the cross section. The mathematical expression of the boundary condition is.

where, nx,y = direction cosines of the unit outward normal to the boundary surfaces,

qb = total boundary heat gain or loss, W/m^, including solar radiation, thermal radiation and convection, etc.

3 Boiindary Heat Transfer Conditions

It is obvious that the main difficulty in solving Eq.1 is to establish the boundary heat transfer conditions which are time and/or temperature dependent. The heat transfer through the boundaries of a concrete body takes place basically in five forms: solar radiation, convection, thermal radiation, evaporation and condensation. Of these, evaporation and condensation heat transfers are the least significant for concrete structure in normal conditions, and are neglected here in the modelling (Wang 1994).

The amount of solar radiation that reaches the concrete surface can be estimated from the structure's geographical location, orientation, altitude, atmospheric conditions, time of the day, day of the year (Duffie and Beckman 1974, Dilger and Ghali 1980). The amount of solar energy absorbed by the concrete depends on the solar radiation absorptivity of the surface which is affected by the colour and texture of the surface material.

The convection heat transfer is the heat loss to or gain from the surrounding air as a result of the air movement. It depends on the wind speed and the temperature difference between the concrete surface and the bulk air (Holman 1986).

Thermal radiation is the heat radiation emission by the concrete body. It is a function of the surface temperature and the environment temperature as well as the thermal emissivity of concrete.

When more accurate data is not available, the diurnal variation of the ambient air temperature can be assumed to follow a sinusoidal cycle between the maximum and minimum values (Hulsey 1976, Dilger and Ghali 1980).

4 Cement Hydration Heat Development

It is apparent that the total cement hydration heat of a concrete mix depends on the cement type and content. For a particular cement, the hydration rate (or the heat release rate) at any moment depends only on the total heat already released (i.e., the concrete maturity) and the current temperature (Neville 1981, Mindess and Young 1982). The temperature effects on cement hydration rate is sometimes called the temperature function. After a careful study of different functions (Wang 1994), the Arrhenius Function was chosen for this study.

When test results on adiabatic temperature rise with time for the concrete mix are available, the cement hydration heat rate at standard temperature (20°C) and any other temperature can be derived and applied to the computer model. More often, test results are, however, not available. In this case, the present study suggests, for example, the following relationships between hydration heat rate and concrete maturity at 20°C for ordinary portland cement (Wang 1994):

$$\begin{aligned} q &= 0.5 + 0.54M^{\circ}\text{-}\hat{} && \text{for } M \ \hat{}\ 10.0 \text{ hours} \\ q &= 2.2 \exp[-0.0286(M - 10.0)] && \text{for } M > 10.0 \text{ hours} \end{aligned} \qquad (3)$$

where M is the maturity of concrete in hours.

When the temperature is constantly 20°C, the concrete maturity is equal to its clock age, and when it is different from 20°C,

$$M = fH(T) \quad dt \qquad (4)$$

where H(T) is the temperature function and t is clock time.

With the introduction of the temperature function, the hydration heat rate at any temperature can be obtained for the analysis. It should be pointed out that the concrete maturity and temperature at a different location of the cross section is normally different. Therefore, the hydration heat rates at different points are normally different at a particular time as well.

5 Finite Element Transient Thermal Analysis

Following the work of Dilger and Ghali (1980) and Elbadry and Ghali (1983), three types of elements are used to model the concrete cross section including any formwork and/or insulation, i.e., 4-node bilinear rectangular elements and 3-node linear triangular elements for the body, and 2-node linear one-dimensional elements for the boundary.

The temperature field of the cross section at each time step is obtained by the variational finite element method combined with the Galerkin weighted residual method for the time domain solution. The non-linearity problem of the thermal radiation heat transfer through the boundary is bypassed by converting the non-linear radiation heat transfer into the quasi-linear "radiant convection". Since the fictitious radiant-convection-heat-transfer coefficient is only slightly temperature dependent (Maes 1980), the iteration process of finding the coefficient is avoided by the approximate extrapolation from the values of the two previous time steps.

Most of the time, concrete structures undergo changes in the early ages, such as the removal of formwork and/or insulation materials, and multi-lift casting. The thermal analysis of hardening concrete should be able to accommodate these changes. This is achieved in this study by automatically transferring the element nodal temperature and concrete maturity data from the end of the previous stage to the beginning of the current stage.

The whole analysis is carried out by computer programm FETAB coded in FORTRAN 77. It was originally developed by Dilger and Ghali (1980) to calculate the temperature distribution within the cross section of a steel-concrete composite box-girder bridge for boundary conditions of constant heat flux, convection and heat generation. Refinement and extension were done by Elbadry and Ghali (1982) to account for the time-varying boundary heat transfer conditions.

For the present study, the program is extended to include the hydration heat rate of various types of cements as functions of maturity and current temperature. It is further developed to handle the changes of boundary conditions and structure configurations such as multi-lift casting and removal of formwork. The prediction of concrete maturity and strength development in the structure is also added.

The basic input for the computer analysis includes: cement content and type, thermal properties of each conduction material and boundary, environmental conditions such as air temperatures and solar radiation intensity as a function of incidence angle, initial concrete temperature and maturity, as well as the finite element mesh of the structure. The time step length for the analysis is normally chosen as one hour for the first 2 days or so and increased for the rest of the time.

6 Results

The main output of the thermal analysis of hardening concrete is the concrete temperature at each node of the finite element mesh at each time step. The temperature contour of the cross section at a particular moment and/or the temperature history of any point, for example, are readily available through some post processing.

Fig.1 is the temperature contour at age 72 hours in a fictitious concrete column of 2mx4m cross section cast in a steel formwork without insulation in an ambient temperature varying between 0 to 10°C. The fresh concrete temperature at casting is assumed to be the average air temperature of the day, i.e., 5°C.

Fig.2 shows the comparison of computed and measured temperature development in a Imxim test column cast in an indoor environment (Cook et al 1992). The cement content was 355kg/m^, and the 28-day concrete strength was 35 MPa. The ambient air temperature during testing was about 28°C.

Fig.3 presents the comparison of computed and measured temperature development in a massive spine beam of the Gamble Bridge constructed in Vancouver, Canada in 1984 (Dilger 1985). The 1.6m thick spine beam was cast in one lift. The local weather during the first several days was mainly overcast, and the average minimum and maximum ambient air temperatures during the days were 2.5 and 9.5°C. After casting, the top concrete surface was covered with a thin thermal blanket to protect it from rain for about five days.

Fig.4 compares the predicted and measured (Tsukayama 1974) temperature distribution in a test beam cast in 3 lifts in 7-day intervals. It shows the temperature distribution along the vertical centre line of the cross section 24 hours after casting the third lift. The average ambient air temperature during the experiment was about 30°C, and $400 kg/m^3$ of ordinary cement was used in the mix.

7 Conclusions and Remarks

The present study on thermal analysis and computer modelling of temperature development in hardening concrete clearly demonstrates that it is possible to predict with reasonable accuracy the temperature development in hardening concrete in the field. Consequently, one can readily estimate the concrete maturity and development of the compressive and tensile strength at any location of the structure. Since temperature distribution and history in hardening concrete are critical data in assessing both the possibility of thermal cracking and the development of concrete strength at early ages, the computer model is a very useful tool to concrete structure designers as well as contractors. It can also be used to help in construction planning, such as designing the curing measures in order to prevent the concrete from freezing in cold weather, or to achieve the desired strength at an early age, or to maintain the specified temperature differential limits in the structure.

Actually, the model and computer program are currently used in the design and construction planning of the 13.5km long Northumberland Strait Crossing, a concrete box-girder bridge connecting the Prince Edward Island to the mainland in Eastern Canada.

It should be pointed out that thermal analysis, especially a transient one as in hardening concrete, is a very complicated process, involving many uncertainties in both material properties and environmental conditions. According to Holman (1986) and many other experts, 20-30% of error is considered excellent accuracy in practical problems. The main source of error is from the input data rather than from the modelling or computation. Many key parameters, such as the thermal conductivities of concrete and formwork/insulation materials, convection heat transfer coefficient, ambient air temperature, etc., vary in wide ranges. Nevertheless, reference values from tests and studies by various researchers in the past can be found (Wang 1994), and the selection of them for a particular case should be based on the careful examination of the field conditions. Convection heat transfer coefficients at different surfaces for different wind speeds suggested by Kehlbeck (1975) are, for example, excellent guidelines.

The hydration heat properties such as the heat libration rate and total amount of heat are very much cement dependent. Even for the same type of cements of different manufacturers (origins), the total hydration heat and its development rate at early ages differ substantially. Therefore, it is most desirable to obtain the hydration heat characteristics by testing the cement to be used when a reliable prediction of concrete temperature development is required.

The current model does not take into account the effect of

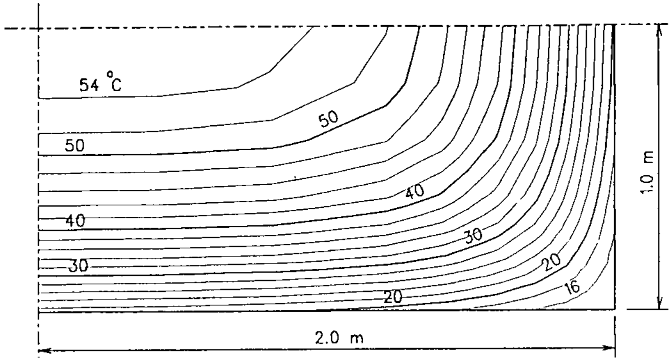

Fig.1. Temperature contour in a quadrant of a 2x4m column 72 hours after casting. (400kg/m^3 of ordinary cement, steel formwork).

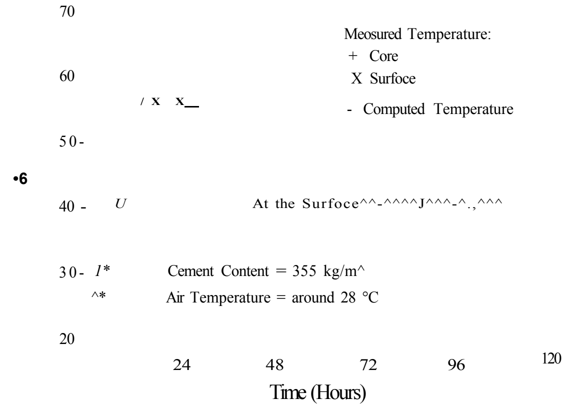

Fig.2. Comparison of computed and measured (Cook et al 1992) temperature in a column.

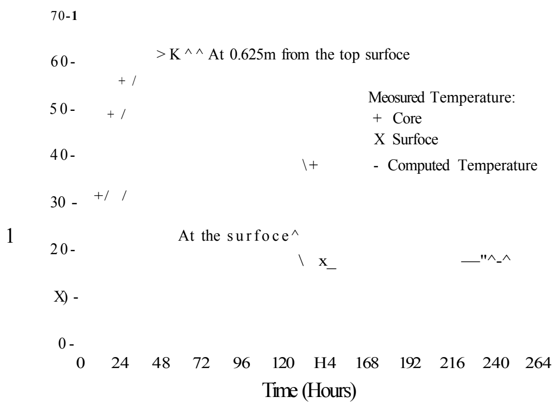

Fig.3. Comparison of computed and measured (Dilger 1985) temperature in a spine beam.

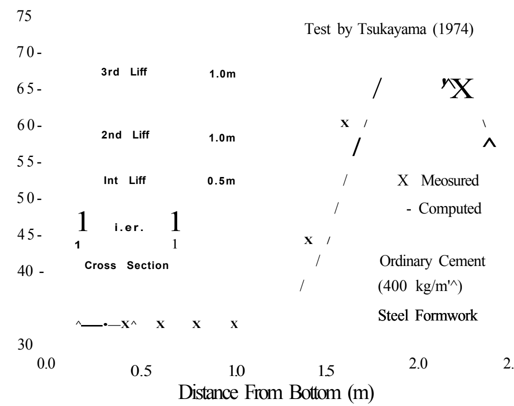

Fig.4. Temperature distribution along the vertical centre line of the cross section 24 hours after casting the third lift.

water/cement ratio on hydration heat release. The water/cement ratio is assumed to be high enough (e.g., not less than 0.4) to allow the maximum possible degree of hydration of all the cement. For high performance concrete with very low water/cement ratio, the hydration heat and strength developments are somewhat different, and further research is needed to accurately predict the thermal response.

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4 LOW-HEAT PORTLAND CEMENT USED FOR SILO FOUNDATION MAT - TEMPERATURES AND STRESSES MEASURED AND ANALYZED

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Abstract
rhc increase in the size of concrete structures in recent years has brought with it a rise in the needs of low-heat type cement. This paper introduces the properties of concrete using low heat portland cement, deals with its application to the actual mass concrete structure and reports the results of the measurement of temperature and thermal stresses of the mass concrete.
The paper also deals with thermal stress analysis on the structure and a comparison between the measured and analyzed results.
Keywords: Low Heat Portland Cement, Mass Concrete.

1 Preface

For massive concrete structures, blast-furnace slag and/or fly ash are sometimes used with portland cement for their low heat generation in Japan. In recent years, however, concrete constructions have become larger and are demanding concrete with lower heat, higher strength and durability.

With these points as background, research and development on beUte Portland cement, in which more belite is contained than in moderate heat Portland cement, is being carried out and its effectiveness on thermal and strength properties is being recognized. The basic properties of belite Portland cement (low heat portland cement) and concrete are explained here, together with its application to an actual construction.

At the construction, 1500 m^ concrete with low heat portland cement was cast in a day for 20,000 ton Cement Silo Foundation Mat.

2 Introduction of Low Heat Portland Cement

The cement is composed only from clinker and gypsum. The cement contains no other pozzolanic materials such as fly ash, blast furnace slag, silica fume or natural pozzolans. The chemical composition and the physical properties of the cement are shown in Table 1 and Table 2, compared with other Portland cements.

The chemical composition and the physical properties conform to the Moderate Heat Portland Cement of The Japanese Industrial Standard "Portland Cement" (JIS-R5210).

Concrete using low heat portland cement showed low strength at younger ages as 28 days, but high strength development in the long term.

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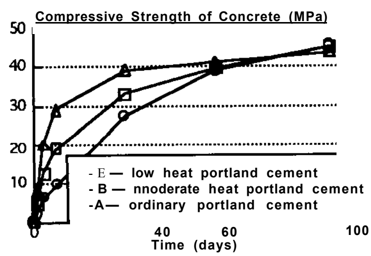
Table 1 Mineral Composition of Cements

	C3S	QS	C3A	C4AF	
low heat	27	58	2	8	(%)
moderate heat	*44*	33	4	12	(%)
ordinary	52	23	9	9	(%)

Table 2 Physical Properties of Cements

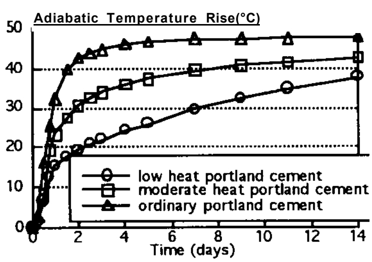
	Specf. gravity (cm2/g)	Blaine fineness	Comp. Strength (MPa)			Heat of hydration (J/g)			
			3d	7d	28d	91d	7d	28d	91d
low heat	3.22	3350	7.4	11.3	31.6	59.7	202	266	313
moderate heat	3.21	3040	11.4	16.7	35.8	51.5	270	319	352
ordinary	3.16	3250	14.8	25.2	41.5	48.1	326	373	401

Compressive strength characteristics are shown in Fig. 1.
The adiabatic temperature rise with low heat portland cement is low, particularly in the initial period compared with other concrete in Fig. 2.



Gmax 20 mm
Slump 1 2 cm
Air content 4 %
W/C 55 %
water curing
20 X

Fig 1 Strength of the Concrete using several kind of Cements



Gmax 20 mm
Slump 12 cm
Air Content 4 %
Cement 2801<g/m3
Placing
Temperature 20°C

Fig. 2 Adiabatic Temperature Rise of the Concrete

3 Application to the Structure

3.1 Silo Foundation

The concrete structure in which the cement was applied was a foundation slab of 20,000 ton Cement Silo. The shape of the foundation mat is a circular disk, and the size is 29 meter in diameter, 2.4 meter thick, which supports seven hoppers and the silo barrel. The section of the silo is shown in Fig. 3. The hoppers and the silo barrel are constructed using ordinary portland cement, but for the foundation mat that has massive section low heat Portland cement.

Table 3 Concrete Mix

Gmax	**25 mm**
Slump	**15 cm**
Air content	**4%**
W/C ratio	**57%**
s/a ratio	**48%**
W	**174 kg/m3**
C	**305 kg/m3**
S	**870 kg/m3**
G	**948 kg/m3**
Add.	**1068 ml/m3**

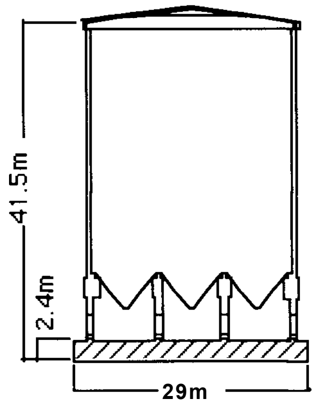


Fig. 3 Section of the Silo

3.2 Concrete

The specified strength of concrete is 24 MPa at 56 days. The concrete mix and the strength properties are shown in Table 3 and Table 4.

Table 4 Strength Properties

	Id	3d	7d	28d	56d	91d
Compressive strength (MPa)	1.95	5.15	6.78	20.6	33.3	39.5
Tensile strength (MPa)	0.26	0.50	0.69	2.12	2.97	-
Young's modulus (*100 MPa)	64	103	134	228	270	290

3.3 Placement

Concrete was placed using two concrete pumps with boom. Temperature of the concrete cast was 26 degrees in centigrade. The placing was begun at 5:30 in the morning Sept. 22, continued until 19:00 in the evening.

3.4 Temperature and Stress Measurement

Temperature, concrete stress, and re-bar stress were measured at selected points. Measuring sensors are mainly placed at the center (bottom, mid-height, and the top of the slab) , and secondary at the peripheral position of the disk. Thermo-couples are placed a little more closely than other sensors. The measurement was started from the concrete placement and continued to the age of 56 days with four hours' intervals throughout the duration.