EARLY-AGE THERMAL STRESS ANALYSIS OF CONCRETE

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EARLY-AGE THERMAL STRESS ANALYSIS OF CONCRETE

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A THESIS SUBMITTED
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Dedicated to My beloved Mother Padma and Father Perumal
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# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>ACKNOWLEDGMENTS</th>
<th>ii</th>
</tr>
</thead>
<tbody>
<tr>
<td>TABLE OF CONTENTS</td>
<td>iii</td>
</tr>
<tr>
<td>SUMMARY</td>
<td>vii</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>x</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>xii</td>
</tr>
<tr>
<td>ABBREVIATIONS</td>
<td>xvii</td>
</tr>
<tr>
<td>NOMENCLATURE</td>
<td>xviii</td>
</tr>
</tbody>
</table>

## CHAPTER 1: Introduction

1.1 General  
1.1.1 Early age thermal cracking of concrete  
1.1.2 Basic mechanism of early age thermal cracking  
1.2 Literature Review  
1.2.1 Early age material properties of concrete  
1.2.2 Thermal expansion of concrete  
1.2.3 Influence of aggregate types and other factors  
1.2.4 Thermal conductivity of concrete  
1.2.5 Thermal conductivity methods  
1.2.6 Factors affecting the thermal conductivity of concrete  
1.3 Mechanical Properties  
1.3.1 Modulus of elasticity  
1.3.2 Tensile Strength of concrete  
1.3.3 Creep behavior of young concrete  
1.4 Heat of Hydration  
1.5 Restraint condition  
1.5.1 Internal Restraint
1.5.2 External restraint
1.5.3 Restraint factor
1.6 Finite Difference Method
1.7 Finite Element Method
1.8 Prediction of early age thermal cracking
1.9 Objective and Scope

**CHAPTER 2:** Thermal properties of various concrete

2.1 Laboratory work

<table>
<thead>
<tr>
<th>2.1.1 Materials</th>
<th>35</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1.2 Mix proportions</td>
<td>35</td>
</tr>
<tr>
<td>2.1.3 Test specimens preparations</td>
<td>39</td>
</tr>
</tbody>
</table>

2.2 Thermal properties - Test methods

<table>
<thead>
<tr>
<th>2.2.1 Thermal expansion test</th>
<th>39</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.2.2 Thermal conductivity test</td>
<td>40</td>
</tr>
</tbody>
</table>

2.3 Results and discussions

<table>
<thead>
<tr>
<th>2.3.1 Thermal expansion</th>
<th>42</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.3.2 Thermal conductivity</td>
<td>46</td>
</tr>
</tbody>
</table>

**CHAPTER 3:** Development of innovative thermal conductivity System (TCS)

3.1 Shortcomings in existing methods
3.2 Basic principle of TCS
3.3 Thermal conductivity of hollow sphere shape
3.4 Optimum radius for thermal expansion test
3.5 Temperature Gradient Analysis
3.6 Prediction of mean sample temperature
3.7 Heat transfer analysis on hollow sphere

<table>
<thead>
<tr>
<th>3.7.1 Finite element analysis : ABAQUS</th>
<th>58</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.7.2 Hollow sphere with thermal contact material</td>
<td>62</td>
</tr>
</tbody>
</table>
3.8 Experimental studies on TCS and discussion on test results
  3.8.1 Verification on standard reference material (PTFE)
  3.8.2 Experimental procedure
  3.8.3 Thermal conductivity test on concrete
3.9 Advantages of invented thermal conductivity system

CHAPTER 4: Determination of early age thermal diffusivity - An analytical approach
4.1 Introduction
4.2 Importance of thermal diffusivity at early age
4.3 Basic Principles of thermal diffusivity method
4.4 An Analytical approach
4.5 Verification of the analytical solution
  4.5.1 Finite difference method
  4.5.2 Finite element method: ABAQUS
4.6 Experimental procedure to measure diffusivity at early age
4.7 Results and discussions

CHAPTER 5: Early age thermal stress analysis on massive Raft foundation
5.1 Introduction
5.2 Experimental studies on raft foundation
  5.2.1 Site monitoring
5.3 Laboratory tests
  5.3.1 Setting time
  5.3.2 Compressive strength
5.3.3 Elastic modulus 106
5.3.4 Creep test 107
5.3.5 Adiabatic temperature rise 107
5.3.6 Early age CTE – Using Kada et al Method 108
5.3.7 Autogeneous Shrinkage 110

5.4 Determination of early age thermal properties – Proposed new method 111
5.4.1 Thermal expansion 111
5.4.2 Thermal diffusivity 115

5.5 Material properties for temperature and stress analysis 115

5.6 Finite element Analysis – ABAQUS 119
5.6.1 Boundary conditions 121
5.6.2 Load cases considered 123

5.7 Results and Discussions 125
5.7.1 Temperature predictions on raft foundation 125
5.7.2 Stress predictions in raft foundation 129

CHAPTER 6: Conclusions 135

REFERENCE 139
Early-age thermal cracking is major concern in massive concrete elements, which is associated with heat of cement hydration and time dependent properties at early age. It can be predicted based on the temperature, strain and stress parameters. The key point is to predict the risk of cracking in mass concreting using reliable material models and methods for analysis. Therefore, three main factors to be considered in thermal stress analysis are temperature development in the concrete being cast, mechanical and thermal behavior of the young concrete and the degree of restraint imposed on the concrete.

The main focus of this research works is the importance of the evolving early age material properties for the thermal stress development. A new method has been devised to measure the thermal properties of concrete at early-age. This method provides for the continuous measurement of early-age thermal properties of concrete in view of the thermal properties continuously varying as concrete hardens. This method also accounts for the generation of heat of hydration at early-age which in many cases had generally added to the difficulty in measuring the early-age diffusivity.

Thermal properties of various concretes including lightweight concretes were discussed with respect to its influencing parameters such as density, age and temperature. Based on the existing guarded heat flow (GHP) method, edge heat loss was observed during the thermal conductivity measurements. This is due to the lateral heat flow from the main heater. While considering this issue, the innovative thermal
conductivity system was proposed based on radial heat flow i.e. unidirectional heat flow system to overcome the shortcoming. Double O-Ring concept was used to ensure unidirectional heat flow under perfect vacuum condition.

The accurate temperature development within the concrete at early ages requires the accurately determined heat of hydration, thermal expansion, thermal conductivity and specific heat capacity. Due to the change in state of the concrete from liquid to solid and undesirable boundary conditions at early ages, determination of those parameters at early ages is highly complicated. Under this circumstance, thermal diffusivity of concrete might be the useful parameter to determine the temperature development accurately at early ages. A new method was proposed to determine the thermal diffusivity of concrete at early age, which takes into account the heat of hydration for temperature development in the concrete. This method is also used to measure the thermal expansion of concrete at early ages.

Further, with the early age properties, a transient coupled thermal stress analysis (ABAQUS) was performed to predict the temperature and stress development for an actual raft foundation. A detailed laboratory tests was conducted on the concrete samples which was obtained from the site. In the numerical model, the visco-elastic behavior of young concrete was also simulated to predict the thermal stress accurately. Three loading combinations namely thermal properties, shrinkage and creep / relaxation of concrete were applied in the model to understand its effects in mass concrete structures. The temperature development and thermal stress predicted by finite element simulation of the raft foundation and site measured data at appropriate locations were compared. The conclusion of this study demonstrates the importance of implementing
time dependent material properties for temperature development and its significance for accurate thermal stress analysis.
LIST OF TABLES

CHAPTER 1

Table 1.1 Influences of Aggregates on CTE 9
Table 1.2 Thermal conductivity of various concretes 13

CHAPTER 2

Table 2.1 Mix proportions for Foam concrete without sand 36
Table 2.2 Mix proportions for Foam concrete with sand 36
Table 2.3 Mix proportions for high strength lightweight concrete 36
Table 2.4 Mix proportions for Pumice lightweight concrete 37
Table 2.5 Mix proportions for Normal weight concrete 37
Table 2.6 Properties of Lightweight Aggregates (LWA) 37

CHAPTER 3

Table 3.1 Case: 1 Heat flux = 37.68 watts and $k = 1.8 \, \text{W/m}^o\text{C}$ 60
Table 3.2 Case: 2 Heat flux = 56.52 watts and $k = 1.8 \, \text{W/m}^o\text{C}$ 60
Table 3.3 Case: 3 Heat flux = 75.36 watts and $k = 1.8 \, \text{W/m}^o\text{C}$ 60
Table 3.4 Case: 4 Heat flux = 53.62 watts and $k = 1.8 \, \text{W/m}^o\text{C}$ 61
Table 3.5 PTFE thermal conductivity test results summary 70
Table 3.6 LWC thermal conductivity results 76
# CHAPTER 4

Table 4.1  Calculation of thermal diffusivity from experimental data  
96

# CHAPTER 5

Table 5.1  Type of concrete materials and their mix proportions  
102
Table 5.2  Various parameters used for thermal stress analysis  
123
Table 5.3  Load cases considered for thermal stress analysis  
124
# LIST OF FIGURES

## CHAPTER 1

<table>
<thead>
<tr>
<th>Fig.</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>CTE increase with temperature for various densities of concrete</td>
<td>8</td>
</tr>
<tr>
<td>1.2</td>
<td>Thermal conductivity of concrete as function of temperature (Verlag et al., 1982)</td>
<td>19</td>
</tr>
</tbody>
</table>

## CHAPTER 2

<table>
<thead>
<tr>
<th>Fig.</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Preparation of test specimen for thermal expansion test</td>
<td>38</td>
</tr>
<tr>
<td>2.2</td>
<td>Preparation of test specimen for thermal conductivity test</td>
<td>38</td>
</tr>
<tr>
<td>2.3</td>
<td>Demec strain gauge employed for measuring the change in length</td>
<td>39</td>
</tr>
<tr>
<td>2.4</td>
<td>Guarded Hot Plate (GHP-300) thermal conductivity system</td>
<td>41</td>
</tr>
<tr>
<td>2.5</td>
<td>Relationship between CTE of concrete and density.</td>
<td>43</td>
</tr>
<tr>
<td>2.6</td>
<td>CTE of Foam concrete (with and without sand) at 40°C, 50°C and 60°C</td>
<td>43</td>
</tr>
<tr>
<td>2.7</td>
<td>CTE of Liapor concrete and Leca concrete varying with temperature</td>
<td>44</td>
</tr>
<tr>
<td>2.8</td>
<td>CTE of pumice concrete and NWC varying with temperature</td>
<td>44</td>
</tr>
<tr>
<td>2.9</td>
<td>Relationship between thermal conductivity of LWC and oven dry densities</td>
<td>45</td>
</tr>
<tr>
<td>2.10</td>
<td>Relationship between thermal conductivity of foam concrete (without sand) and temperature</td>
<td>47</td>
</tr>
<tr>
<td>2.11</td>
<td>Relationship between thermal conductivity of foam concrete (with sand) and temperature</td>
<td>47</td>
</tr>
</tbody>
</table>
Fig. 2.12 Relationship between thermal conductivity of Leca and Liapour concretes and temperature 48
Fig. 2.13 Relationship between thermal conductivity Pumice and Normal weight concrete and temperature 48

CHAPTER 3

Fig. 3.1 Density of concrete material versus weight of sphere specimen for corresponding inner and outer radius 54
Fig. 3.2 Heat flux (power) required for different temperature gradient versus conductivity of sample 55
Fig. 3.3 Temperature profile over thickness of specimen for hollow sphere 57
Fig. 3.4 Mesh generated to hollow sphere Quadratic elements (DC3D20) 59
Fig. 3.5 Contour plot of temperature distribution for semi hollow sphere (ABAQUS output) 61
Fig. 3.6 Error in hot side temperature for varying thermal contact material thickness 63
Fig. 3.7 Flow chart – Thermal conductivity system working principle 66
Fig. 3.8 Vacuum Adaptor design for thermal conductivity tests 67
Fig. 3.9 Thermal conductivity test on PTFE material 68
Fig. 3.10 Vacuum Adaptor with vacuum gauge 68
Fig. 3.11 Heater temperatures of Test Type I and Test Type II 71
Fig. 3.12 Hot side temperatures of Test Type I and Test Type II 71
Fig. 3.13 Cold side temperatures of Test Type I and Test Type II 72
Fig. 3.14 Mean temperatures of Test Type I and Test Type II 72
Fig. 3.15 Power required for Test Type I and Test Type II 73
Fig. 3.16 Special Mold design of base and cover 74
Fig. 3.17  TCS test on LWC with modified vacuum adaptor  
Fig. 3.18  Mean temperature of LWC  
Fig. 3.19  Power required versus time

CHAPTER 4

Fig. 4.1  Diffusivity as a function of reciprocal of time  
for various \((\Delta T_h/\Delta t)\)  
Fig. 4.2  Comparison of Analytical solution with Finite Difference and  
Finite Element Method for \(T_2 - T_1 = 5^\circ C, \Delta T_h/\Delta t = 1\)  
and \(\Delta T = 0.1^\circ C\)  
Fig. 4.3  Comparison of Analytical solution with Finite Difference and  
Finite Element Method for \(T_2 - T_1 = -5^\circ C, \Delta T_h/\Delta t = 1\)  
and \(\Delta T = -0.1^\circ C\)  
Fig. 4.4  Finite element mesh of solid cylinder  
Fig. 4.5  Experimental set-up for the determination of diffusivity of  
concrete at early age.  
Fig. 4.6  Variation of concrete core and oven temperature with time.  
Fig. 4.7  Adiabatic temperature rise of concrete  
Fig. 4.8  Adiabatic temperature rise of concrete at the corresponding  
equivalent age at reference curing temperature of 20°C  
Fig. 4.9  Variation of concrete thermal diffusivity with time.

CHAPTER 5

Fig. 5.1  Details of raft foundation (A, B and C are locations of  
vibrating strain gauges at midsection of raft foundation)  
Fig. 5.2  Embedded vibrating wire strain gauges.  
Fig. 5.3  Installation of embedded vibrating wire strain gauges.  
Fig. 5.4  Tested sample and penetration resistance apparatus  
Fig. 5.5  Specimens preparation for compressive, modulus of
Elasticity and creep tests.

Fig. 5.6  Installation of KM 100B strain gauges along specimen center

Fig. 5.7  Portable data logger used for thermal expansion and Autogenous shrinkage test

Fig. 5.8  Illustration of temperature cycle of specimen

Fig. 5.9  Cylindrical specimen for proposed method

Fig. 5.10  Temperature cycle obtained - proposed new method

Fig. 5.11  Corrected real strain reading from strain gauge

Fig. 5.12  Coefficient of thermal expansion of concrete on ages

Fig. 5.13  Development of Modulus of elasticity of concrete Varying with age

Fig. 5.14  Creep Compliance $J(\Delta t_{\text{load}}, t_0)$ with varying loading age $\Delta t_{\text{load}}$

Fig. 5.15  Adiabatic temperature rise curve of ATR1, ATR2, ATR3 for CS1, CS2, CS3 respectively

Fig. 5.16  Mean daily temperature (Singapore)

Fig. 5.17  Finite Element Mesh – Raft foundation

Fig. 5.18  Measured and predicted Temperature varying with time at mid section A of CS1 concreting

Fig. 5.19  Measured and predicted Temperature varying with time at mid section B of CS2 concreting

Fig. 5.20  Measured and predicted Temperature varying with time at mid section C of CS3 concreting

Fig. 5.21  Early age thermal expansion effect on the thermal strains due to ATR1
| Fig. 5.22 | Early age thermal expansion effect on the thermal strains due to ATR2 | 128 |
| Fig. 5.23 | Early age thermal expansion effect on the thermal strains due to ATR3 | 129 |
| Fig. 5.24 | Stress development at mid section C of CS1 concreting | 132 |
| Fig. 5.25 | Stress development at mid section B of CS2 concreting | 132 |
| Fig. 5.26 | Stress development at mid section C of CS3 concreting | 133 |
| Fig. 5.27 | Predicted tensile strength development (CEB- Model) | 133 |
## ABBREVIATIONS

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ATR</td>
<td>Adiabatic Temperature Rise</td>
</tr>
<tr>
<td>BFS</td>
<td>Blast Furnace Slag</td>
</tr>
<tr>
<td>CCC</td>
<td>Concrete Cracking Control</td>
</tr>
<tr>
<td>CTE</td>
<td>Coefficient of Thermal Expansion</td>
</tr>
<tr>
<td>GGBS</td>
<td>Ground Granulated Blast Furnace Slag</td>
</tr>
<tr>
<td>GHP</td>
<td>Guarded Hot Plate</td>
</tr>
<tr>
<td>LWA</td>
<td>Light Weight Aggregates</td>
</tr>
<tr>
<td>LWC</td>
<td>Light Weight Concrete</td>
</tr>
<tr>
<td>NWC</td>
<td>Normal Weight Concrete</td>
</tr>
<tr>
<td>OPC</td>
<td>Ordinary Portland Cement</td>
</tr>
<tr>
<td>PFA</td>
<td>Pulverized Fuel Ash</td>
</tr>
<tr>
<td>PTFE</td>
<td>Poly Tetra Fluoro Ethylene</td>
</tr>
<tr>
<td>RTDs</td>
<td>Resistance Temperature Detectors</td>
</tr>
<tr>
<td>SF</td>
<td>Silica Fume</td>
</tr>
<tr>
<td>TCS</td>
<td>Thermal Conductivity System</td>
</tr>
<tr>
<td>TSC</td>
<td>Tensile Strain Capacity</td>
</tr>
</tbody>
</table>
NOMENCLATURE

\( b_1, b_2 \) = Model parameters
\( c \) = Specific heat capacity
\( E \) = Activation energy
\( E_{ci} \) = Modulus of elasticity at 28 days
\( E_{ref} \) = Modulus of elasticity at 28 days age chosen as reference value
\( E_{c,i}(t) \) = Modulus of elasticity at an age \( t \) days
\( f_{ct,28} \) = Tensile strength at age of 28 days
\( f_{ct} \) = Tensile strength of concrete
\( E \) = The voltage reading in Volts,
\( I \) = Current reading in Amperes
\( J \) = Creep compliance in terms \( \Delta t_{bud} \) and \( t_0 \)
\( k_X \) = Thermal conductivities of concrete in the x coordinate
\( k_Y \) = Thermal conductivities of concrete in the y coordinate
\( k_Z \) = Thermal conductivities of concrete in the z coordinate
\( k_{dry} \) = Thermal conductivity coefficient at dry state
\( k_{moist} \) = Thermal conductivity coefficient at moist state
\( k_a \) = Thermal conductivity of aggregate
\( k \) = Thermal conductivity of concrete or mortar or aggregate
\( k_m \) = Thermal conductivity of mortar
\( l_o \) = Length at reference temperature
\( L \) = Isotropic solid cylinder of length
\( \Delta l \) = Length of change of specimen for temperature differential
\[ M \] = The equivalent age maturity function

\[ n \] = Number of iterations in the finite difference analysis

\[ \beta_{cc} \] = Coefficient describing the development of strength with time \((t)\)

\[ \beta(t) \] = Modified age coefficient with time

\[ \chi \] = Constant aging coefficient

\[ \varphi \] = Creep function

\[ \varepsilon_{cr} \] = Creep strain \( \varepsilon_{cr} \)

\[ \rho \] = Density of material

\[ \varepsilon' \] = Restrained strain

\[ \varepsilon_{el} \] = Elastic strain

\[ \sigma_{fix} \] = Fixation stress for \( \varepsilon(t) = 0 \) at time \( t \)

\[ \alpha \] = Linear coefficient of thermal expansion per degree C,

\[ \varepsilon_{th} \] = Thermal strain

\[ \varepsilon^{cr} \] = Time dependent creep deformation

\[ \varepsilon^{cr} \] = Time dependent deformation

\[ \varepsilon_{m} \] = Total actual movement

\[ \varepsilon_{R} \] = Total free strain

\[ \varepsilon_{as} \] = Autogeneous shrinkage

\[ \alpha \] = Diffusivity of concrete

\[ \sigma_{c} \] = Loading stress at \( t_0 \)

\[ \sigma(t) \] = Stress development at specific point of the structure

\[ p \] = Volume of mortar per unit volume of concrete
\( Q_h \) = Heat generated due to cement hydration and external sources

\( Q_h(t) \) = Rate of heat generation within a body, function of time and position

\( Q \) = Heat transfer rate per square area

\( Q \) = Input power of the main heater in Watts

\( r \) = Degree of reaction

\( R \) = Isotropic solid cylinder of radius

\( R \) = Universal gas constant.

\( R \) = Restraint factor for a concrete element

\( R_i \) = Inner radius of the hollow sphere

\( R_o \) = Outer radius of the hollow sphere

\( R(t, t_0) \) = Relaxation function

\( S \) = Cross sectional area of the main heater

\( t_e(T) \) = Equivalent age at the reference curing temperature

\( t_B \) = Model Parameter

\( t_0 \) = Time equivalent age in days

\( t_s \) = Apparent setting time in days

\( \Delta t \) = Chronological time interval,

\( \Delta t' \) = Time taken for temperature to rise or fall by \( \Delta T \)

\( \Delta t_{load} \) = Logarithmic of time span after loading

\( \Delta T \) = Temperature differential between initial temperature and final temperature

\( \Delta T_{ATR} \) = Change in Adiabatic temperature rise

\( T \) = Temperature profile

\( T_p \) = Peak temperature at time of striking of formwork
\( T_a \) = Ambient temperature
\( T_1 \) = Temperatures along the cylinder axis, (i.e. \( r = 0 \))
\( T_2 \) = Temperatures along the cylinder axis at the surface (i.e. \( r = R \))
\( T_{ATR} \) = Adiabatic temperature
\( T_C \) = Average concrete temperature during the time interval
\( T_f \) = Final temperature
\( T_h \) = Specimen core temperature
\( T_i \) = Inner surface temperature
\( T_{mean} \) = Mean temperature of the hollow sphere
\( T_o \) = Outer surface temperature
\( T_o \) = Initial temperature
\( T_r \) = Constant reference temperature
\( w \) = Moisture content by weight or volume
Early age thermal cracking of mass concrete is best avoided to ensure a desired service lifetime and function of a structure. Therefore, it is indispensable to perform a reliable thermal stress analysis to predict the risk of thermal cracks by considering analysis parameters that are accurate. This thesis explores the significance of using accurately obtained evolving thermal parameters of concrete as against the normally considered approximated constant values. In addition, new methods to accurately obtain the thermal conductivity and diffusivity of concrete are also discussed.

In chapter one, the motivation for this study is elaborated by discussing the various aspects of thermal and cracking parameters of concrete. Following this, thermal properties of concrete in general, including that of lightweight concrete are explored in the next chapter. Chapter three and four discuss the new methods proposed for the determination of thermal conductivity and diffusivity of concrete, respectively. Chapter five outlines a case study in which the accurately determined thermal properties of concrete are used to predict the thermal stress development in an actual mass concrete on site that had been instrumented. The conclusion of the study is provided in chapter six.
Chapter 1: Introduction

1.1 General

1.1.1 Early age thermal cracking of concrete

The goal of this chapter is to provide brief review of preceding work on early age thermal cracking of concrete and study the importance of early age material properties. In massive concrete structures, the development of high temperature differential creates severe problem which leads to early age thermal cracking of concrete (e.g. dams, nuclear reactors, raft foundations, bridge piers, pile caps, etc) and large floating offshore platforms.

An easy methodology to evaluate thermal cracking is based on tensile strain capacity i.e. thermal cracking occurs when restrained tensile strain greater than tensile strain of concrete (Bamforth, 1981). Accuracy of predicting temperature distributions and stress calculations merely depends on the appropriate effort to include the time dependent material behavior of concrete and implementing the correct boundary conditions in the analysis.

1.1.2 Basic mechanism of early age thermal cracking

Early age cracking of concrete is a well known phenomenon, which is associated with heat of cement hydration and shrinkage of concrete. As long as the cement hydration process begins, it produces considerable amount of heat. The heat evolution of hydration process increases the temperature of cement paste or of concrete. The rate of heat development in concrete depends on thermal properties of concrete mix and the rate at which heat is dissipated.

However, heat of hydration develops a substantial rise in temperature of massive concrete structures due to poor heat dissipation to surrounding
environments. Then, the rate of heat generation slows down, concrete starts to cool and contracts. There is a risk thermal gradients persuades cracks in structures. If the concrete structures are unrestrained, the expansion or contraction does not create any stresses. But in practice, partial or full restraint is unavoidable and is always present. These restraint movements induce compressive and tensile stresses in concrete, consequently causing cracking in concrete at early age. In massive concrete structure, the compressive stresses does not cause any cracking problems but tensile stresses causes cracking when tensile stress exceeds tensile strength of concrete (Harrison, 1992)

1.2 Literature Review

1.2.1 Early age material properties of concrete

The evolution of concrete properties at early age is significant. When concrete has been placed, it undergoes phase change from liquid to solid and thereafter continues to gain strength which ultimately influences other mechanical properties. These evolutions are attributable to hydration of cement which initially causes the concrete to solidify and thereafter gain strength.

On the other hand, the hydration of cement is governed by curing temperature. The rate of hydration is usually greater at early age and at higher curing temperature and gradually slows down to an insignificant level during which time the hardened concrete is relatively inert and stable. It is usually assumed that more than 90% of cement hydration would have completed within the first 28 days. Therefore, most of the concrete properties are generally reported as at 28 days as no significant changes are expected thereafter.
In the case of thermal stress analysis of mass concrete, accurate input of the rate of heat generation due to hydration of concrete is pertinent. It is also imperative that time-dependent properties of concrete at early-age are used for accuracy. In addition, the properties of concrete also depend on the curing temperature and since the temperature history within a mass concrete is varied, the properties of concrete therein can also be expected to vary with time even if the concrete has been placed at the same time. A detailed study on early age material properties would give the relative importance and its contribution to thermal cracking problem. Thereby, predicting the temperature distribution and thermal stresses would be accurate and sensible in order to control the temperature differential and limiting stresses.

### 1.2.2 Thermal expansion of concrete

Most of solids, liquids and gases change its size and or density due to effect of heat. This effect is imperative for building materials when it is used. When the building materials are subjected to change in temperature, it may expand or contract. Most of the materials expand when they are heated, and contract when they are cooled. Temperature changes may be caused by environmental conditions or by cement hydration. As the temperature drops, the concrete tends to be shortened. It is important to predict thermally induced movements in concrete which create stresses in concrete structures and leads to risk of cracking (Clarke, 1993 and ISE, 1987).

Concrete has generally positive coefficient of thermal expansion at ambient conditions but this value mainly depends on concrete mixing ingredients. Theoretically, coefficient of thermal expansion (CTE) is defined as change in unit length per degree change of temperature. It is expressed as Eq. (1.1)


\[
\alpha = \frac{1}{l_0} \frac{\Delta l}{\Delta T}
\]

(1.1)

where, \( \alpha \) is the linear coefficient of thermal expansion per degree C, \( l_0 \) the length at reference temperature and \( \Delta l \) the length of change of specimen for temperature differential \( \Delta T \). Generally, \( \alpha \) is the function of temperature i.e. \( \alpha = \alpha(T) \). It can be calculated from experiments consisting of heating-up the sample from initial temperature \( T_0 \), to the final temperature \( T_f \) and then measuring the relative elongation. Relative elongation measurement is a difficult task from the experimental point of view. This relative elongation error can be corrected by using known CTE standard bar as the reference bar during the test. There is no standard test method or practice for determining the coefficient of thermal expansion of concrete. CTE of concrete samples can be determined by determination of length change due to temperature change. Some of the available methods at present are Dilatometers (ASTM-E228-95), comparative technique, ASTM C531-00 test method, CRD-C 39-81 and TI - B Method.

Dilatometer has shown good accuracy for measuring CTE than other methods. But it is suitable for relatively small samples, typically few millimeters. Jan Toman et al., (1999) followed comparative technique for measurement of CTE of concrete. The reliability of the method was verified with standard materials which has known CTE and temperature field. An estimated value of the coefficient of thermal expansion for concrete may be computed from weighted averages of the coefficients of the aggregate and the hardened cement paste (Mehta, 1993). The amount of thermal expansion and contraction of concrete varies with factors such as type of aggregate, amount of aggregate (siliceous gravel and granite, Leca, pumice),
mix composition, water-cement ratio, temperature range, concrete age, degree of saturation of concrete and relative humidity.

1.2.3 Influence of aggregate types and other factors

Of all these factors, aggregate type and its mineralogical compositions has shown the greatest influence on the expansion coefficient of concrete because of the large differences in the thermal properties of various types of aggregates, modulus of deformation of the aggregate and also concrete contains aggregate constituting from 70 to 85% of the total solid volume of the concrete. The CTE of various aggregates is shown in Table 1.1.

In the case of high temperature changes occurring in concrete structures, Mindess et al., (2003) have described that high amount of differential thermal expansion between cement paste and aggregate creates high internal stresses. CTE of concrete is not only directly proportional to density of concrete but it also depends on concrete mix proportions (Chandra and Bertssan, 2003). CTE of concrete increases with cement content and slightly decrease with age of concrete (ACI-207.4R, 1993).

ACI committee 517 (1980) reports that early age concrete has higher thermal expansion than hardened concrete and similar conclusion was obtained experimentally by Shimasaki et al.(2002) and Kada et al.(2002). At very early age, the drastic change of CTE of concrete is mainly affected by free water presents in concrete. CTE of concretes vary directly with density and amount of natural sand used (Chandra and Bertssan, 2003).
Chapter 1: Introduction

The thermal expansion of cement paste depends on the moisture present in the paste and fineness of cement (ACI-207.4R, 1993). The moisture content presents in concretes increases CTE to some extent. It was pointed out that CTE is low at dry or saturated state and at its highest expansion value at medium moisture content approximately 5 to 10 % by volume (FIP, 1983 and ISE, 1987). Rilem (1993) has studied the relationship between the CTE of Autoclaved aerated concrete (AAC) block and influence of percentage of moisture content, porous system and water content.

Carl and Faruque (1976) have studied expansion of air dried and saturated samples for varying water cement ratio. The experimental results showed that expansion coefficient increased with decrease of water cement ratio. Chandra and Berntsson (2003) showed that under increasing temperature, CTE of LWC increases considerably. Generally, it is constant over normal operating temperature (ACI 207, 1993). Fig 1.1 shows the CTE measurement of different concrete densities tested under the room temperature to elevated temperature above 900°C.
Ribeiro et.al (2003) studied thermal expansion of epoxy and polyester polymer mortars, plain mortar and fibre reinforced mortars. They concluded that the measured thermal expansion with temperature follows a parabolic law rather than a bilinear law.

The thermal expansion of cement paste depends on moisture present in the paste and fineness of cement. It has been reported to be at the lowest expansion value when dry or saturated and at its highest expansion value at intermediate humidity range of 60 to 70% (Marshall, 1972).
Table 1.1 Influences of Aggregates on CTE (Chandra et al., 2003)

<table>
<thead>
<tr>
<th>Type of Aggregates</th>
<th>Average CTE</th>
<th>1 x 10^{-6} per K</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Aggregates</td>
<td>Concrete</td>
</tr>
<tr>
<td>Expanded shale, clay and Slate</td>
<td>-</td>
<td>6.5 – 8.1</td>
</tr>
<tr>
<td>Expanded Slag</td>
<td>-</td>
<td>7.0 – 11.2</td>
</tr>
<tr>
<td>Blast Furnace Slag</td>
<td>-</td>
<td>9.2 – 10.6</td>
</tr>
<tr>
<td>Pumice</td>
<td>-</td>
<td>9.4 – 10.8</td>
</tr>
<tr>
<td>Perlite</td>
<td>-</td>
<td>7.6 – 11.7</td>
</tr>
<tr>
<td>Vermiculite</td>
<td>-</td>
<td>8.3 – 14.2</td>
</tr>
<tr>
<td>Cellular concrete</td>
<td>-</td>
<td>9.0 – 12.6</td>
</tr>
<tr>
<td>Quartzite</td>
<td>10.3</td>
<td>12.1</td>
</tr>
<tr>
<td>Siliceous limestone</td>
<td>8.3</td>
<td>9.4-11.7</td>
</tr>
<tr>
<td>Basalt</td>
<td>6.4</td>
<td>8.3</td>
</tr>
<tr>
<td>Limestone</td>
<td>5.5</td>
<td>5.4-8.6</td>
</tr>
<tr>
<td>Sandstone</td>
<td>9.3</td>
<td>11.4</td>
</tr>
<tr>
<td>Marble</td>
<td>8.3</td>
<td>10.7</td>
</tr>
<tr>
<td>Granite</td>
<td>6.8</td>
<td>9.6</td>
</tr>
<tr>
<td>Dolerite</td>
<td>6.8</td>
<td>9.6</td>
</tr>
<tr>
<td>Gravel</td>
<td>10.3</td>
<td>12</td>
</tr>
<tr>
<td>Chert</td>
<td>11.8</td>
<td>13.2</td>
</tr>
<tr>
<td>Cement Paste-saturated</td>
<td>w/c =0.4</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>w/c =0.5</td>
<td>18-20</td>
</tr>
<tr>
<td></td>
<td>w/c =0.6</td>
<td>18-20</td>
</tr>
</tbody>
</table>

1.2.4 Thermal conductivity of concrete

Concrete is one of the most commonly used construction material and its thermal conductivity draws much importance to determine its actual thermal performance. It is a specific property of a material which is usually expressed in W/mK (Holman, 1997), Eq. (1.2)

\[ k = Q \frac{\Delta T}{\Delta x} \]

Where \( Q \) is the heat transfer rate per square area and \( \frac{\Delta T}{\Delta x} \) the temperature gradient in the direction of heat flow. It is desirable nowadays for most high rise buildings to
have good thermal insulation to utilize less energy. Thermal conductivity of both normal weight and lightweight concrete can be determined by many methods in which Guarded Hot Plate method (ASTM C177-04) has given better accuracy over testing under oven dry condition (Copier, 1979 and Salmon, 2001).

1.2.5 Thermal conductivity methods

Presently, several methods are available to measure the thermal conductivity of building materials and other materials. These are generally categorized as Steady State and Non-steady State methods. Broadly speaking, there are a number of possibilities to measure thermal conductivity of building materials, each of them suitable for a limited range of materials, depending on the thermal properties and the temperature testing range. Salmon (2001) has reviewed the accuracy of existing thermal conductivity system. It can be improved to eliminate lateral heat flow to or from main heater, improvements in data logging and advanced temperature controllers. The uncertainties in thermal conductivity measurements were discussed and evaluated based on governing variables such as thickness of sample, thermal resistance etc., in UKAS report (2001). The report stated that the thermal resistance material, lateral dimensions, heat flux required and thickness of sample should be minimum to preserve desirable accuracy.

The Steady-State technique performs a measurement when the material that is tested is completely under thermal equilibrium. The build-up process is easy i.e. it implies a stable thermal gradient during testing process and the design should ensure one dimensional heat flow (Healy, 2001). The drawback of steady state technique is
that it usually takes a long time to reach the required thermal equilibrium and requires a carefully planned laboratory experiment.

Kulkarni and Vipulanandan (1998) developed a simple steady state method which is actually a modified method of hot wire technique. Based on the linear heat source theory, Morabito (1989) proposed a new transient state thermal conductivity method which is more suitable for non-homogenous, damp and porous solids. Thermal probe can be used in-situ to measure thermal conductivity within short time compared with other methods (VanLoon et al. 1989; Elustondo et al. 2001). CRD-C44 (1965) has calculated thermal conductivity from the results of tests for thermal diffusivity and specific heat for different moisture content.

Based on steady state technique a new method has been proposed and it is discussed in next chapter. It can be used to calculate the thermal conductivity of lightweight concrete and normal weight concrete for which the new methodology is relatively cheap and good accuracy under automation technique.

1.2.6 Factors affecting the thermal conductivity of concrete

Several investigators have given various relationships for thermo-physical properties of concrete and of aggregates. These differences are mainly accounted on difference in materials, particularly on aggregate mineralogical type, macrostructures and gradation. Thermal conductivity of concrete primarily varies due to aggregate type, density, moisture content, temperature, size and distribution of pore structure (Clarke 1993; ACI 213 1999; Khan 2002). Other factors such as chemical composition of solid components, differences in the test methods, and
differences in specimen sizes have shown less effect on thermal conductivity measurement (ISE 1987).

1.2.6.1 Density

Density is a good indication of the thermal conductivity. Thermal conductivity of concrete is directly proportional to its density (Loudon 1979; Uysal et al. 2004). However, although conductivity is a function of density for a given type of concrete, it also depends on variations between concrete made from different raw materials as shown in Table 1.2. It has been observed that concretes of same density, but made with different lightweight aggregates, showed large differences in conductivity. Fundamentally, porosity and density are interrelated parameters and inversely proportionate to each other. Porosity is important for lightweight concrete (LWC) which makes considerable changes in thermal conductivity measurement (Bouguerra et al.1998). Concrete having higher porosity shows lower thermal conductivity due to its low density and higher air content.

Lightweight concretes made with cellular structure contain more air which reduces the rate of heat transfer compared with natural aggregates (Clarke, 1993). If air content is largely or partially replaced by water then the heat flow through material is quicker. It suggests that the light porous aggregates produce concrete of low thermal conductivity, whereas the heavy dense aggregates produce concrete of a higher thermal conductivity. But it is not only total air content in the porosity that governs the thermal conductivity but also other parameter such as geometry of pores and their distribution in the concrete which play a significant role in determination of thermal conductivity (Chandra and Berntsson, 2003).
Table 1.2 The thermal conductivity of various types of concrete (Loudon, 1979)

<table>
<thead>
<tr>
<th>Group No.</th>
<th>Type of LWC material</th>
<th>Dry density kg/m³</th>
<th>Thermal conductivity (k) W/mK</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>LWC with siliceous or calcareous aggregate†</td>
<td>1700-2100</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1650-1900</td>
<td>1.15</td>
</tr>
<tr>
<td>II</td>
<td>LWC with at least 50% calcareous aggregate†</td>
<td>1400-1600</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1200-1400</td>
<td>0.44</td>
</tr>
<tr>
<td>III</td>
<td>Pozzolana or foamed slag aggregate concrete†</td>
<td>1000-1200</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1000-1200</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>950-1150</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td></td>
<td>800</td>
<td>0.29</td>
</tr>
<tr>
<td>IV</td>
<td>Sintered PFA aggregate concrete†</td>
<td>1000</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>Natural Pumice aggregate concrete†</td>
<td>1200</td>
<td>0.47</td>
</tr>
<tr>
<td>V</td>
<td>Pumice concrete and foamed or BFS concrete†</td>
<td>1600-1800</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1400-1600</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1200-1400</td>
<td>0.7</td>
</tr>
<tr>
<td>VI</td>
<td>Expanded clay or Expanded shale aggregate concrete†</td>
<td>1000-1200</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td></td>
<td>800-1000</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>600-800</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&lt; 600</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>600-800</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>400-600</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td></td>
<td>400-450</td>
<td>0.19</td>
</tr>
<tr>
<td>VII</td>
<td>Perlite or vermiculite†</td>
<td>775-825</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>725-775</td>
<td>0.29</td>
</tr>
<tr>
<td></td>
<td>As above large panels Autoclave Aerated concrete†</td>
<td>675-725</td>
<td>0.27</td>
</tr>
<tr>
<td></td>
<td></td>
<td>625-675</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td></td>
<td>575-625</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>525-575</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>475-525</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td></td>
<td>425-475</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td></td>
<td>375-425</td>
<td>0.16</td>
</tr>
<tr>
<td></td>
<td>Autoclaved aerated and foamed concrete and</td>
<td>400</td>
<td>0.14</td>
</tr>
</tbody>
</table>
Chapter 1: Introduction

<table>
<thead>
<tr>
<th>Material</th>
<th>Density (kg/m$^3$)</th>
<th>Thermal Conductivity (W/mK)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light Weight Lime Concrete+</td>
<td>500</td>
<td>0.19</td>
</tr>
<tr>
<td>Autoclaved and Foam Concrete Block and Light Weight Lime Concrete Block*</td>
<td>800</td>
<td>0.41</td>
</tr>
<tr>
<td>As Above, Air Hardened Dense Concrete with Siliceous or Calcareous Aggregate+</td>
<td>1200</td>
<td>0.7</td>
</tr>
<tr>
<td>Dense Concrete with Dense Slag Aggregate+</td>
<td>2200-2400</td>
<td>1.4</td>
</tr>
</tbody>
</table>

† Standard French thermal conductivity values, ‡ Standard German thermal conductivity values, * Standard American equivalent thermal conductivity values

Thermal conductivity of concrete increases with oven dry density and represents function of given density (ISE, 1987 & Rilem, 1993 & Clarke, 1993). In certain ranges from 320 to 960 kg/m$^3$, of autoclaved cellular concretes, Rudolph and Valore (1954) showed that thermal conductivity is a close function of density, in spite of the type of specimens and testing conditions. The thermal conductivity of lightweight concrete made with cenospheres has been tested at different ages with respect to volumetric densities (Blanco et al., 2000).

Based on 400 published results, it has been suggested that calculating the oven dry and air dry state conductivity from the best fitted equations Eq. (1.3) and Eq. (1.4) in terms of density $\rho$, (Valore,1956) is most appropriate.
Chapter 1: Introduction

\[ k = 0.072e^{0.00125\rho} \text{ (Oven dry state)} \]  \hspace{1cm} (1.3)

\[ k = 0.087e^{0.00125\rho} \text{ (Air dry state)} \]  \hspace{1cm} (1.4)

The thermal performance of various concretes is related to the actual operating conditions because thermal conductivity of such materials is highly dependent on moisture content. Experimental results showed that thermal conductivity of AAC increases quite linearly with moisture content (Lippe, 1992). Santos and Cintra (1999) have simulated numerical model to understand the effect of moisture on the thermal conductivity of porous ceramic materials and results were verified with experimental study. Based on several experimental and research works, it was observed that thermal conductivity increases with percentage moisture content (Chandra & Berntsson (2003), Rilem (1993), Clarke (1993), Bonacina et al. (2003)).

The general relationship between thermal conductivity and moisture content of concretes may expressed as follows in Eq. (1.5)

\[ k_{\text{moist}} = k_{\text{dry}} + \Delta k \times w \]  \hspace{1cm} (1.5)

where \( k_{\text{moist}} \), \( k_{\text{dry}} \) and \( w \) are thermal conductivity coefficients at moist and dry state and moisture content by weight or volume, respectively. Oven dry thermal conductivity \( k_{\text{dry}} \) is more consistent and can be easily converted into air dry or any local environmental conditions wherever it is used (FIP,1983).

1.2.6.2 Aggregate

The thermal conductivity of aggregates and thus the concretes made with it, depends on the aggregates internal microstructures, its mineralogical compositions and degree of crystallization (Neville, 1995). Aggregates of higher thermal conductivity produce concrete of higher thermal conductivity. In general, the
conductivity of highly crystalline aggregates i.e. those having a well defined microstructure is high at room temperature and decreases with rise of temperature (Harmathy 1970; Chandra and Berntsson 2003).

Amorphous aggregates exhibit low thermal conductivity at room temperature and these increases slightly as the temperature rises. Lightweight aggregates, particularly manufactured ones, exhibit high chemical stability at elevated temperatures as compared with normal weight aggregates, so only the latent heat affects that must be considered are the ones associated with the dehydration of cement paste. Naturally, all crystalline materials have a higher thermal conductivity than glassy substances. Khan (2002) has reported that the concrete containing quartzite sand is found experimentally to have higher thermal conductivity than mica for varying moisture content.

Cambell and Thorne (1963) proposed a model that takes into account the influence of aggregate type on thermal conductivity and their approach is adequately accurate for aggregates having low thermal conductivity. The thermal conductivity of concrete \( (k) \) expressed in terms of volume of mortar per unit volume of concrete \( (p) \), thermal conductivity of mortar \( (k_m) \) and thermal conductivity of aggregate \( (k_a) \) is given by Eq. (1.6)

\[
k = k_m \left(2M - M^2\right) \frac{k_mk_a \left(1 - M^2\right)}{k_aM + k_m \left(1 - M\right)}
\]

where \( M = 1 - (1 - p)^{1/3} \).

The thermal conductivity of concretes depends on the porosity, volume of aggregates and types of aggregate. Since, moisture has a significant influence on thermal conductivity of concrete, material having higher porosity level yields higher
Chapter 1: Introduction

thermal conductivity. Recently, Santos (2003) reported that thermal conductivity of conventional refractory concrete varies linearly with porosity for porosity of 0 to 35%. Kim et al., (2003) studied the effects of volume fraction and justified that it is independent of moisture condition and temperature. Kim reported that thermal conductivity increases linearly with increase of aggregate volume fractions.

However, porosity of lightweight aggregates is high and the solid matrix is normally amorphous and therefore thermal conductivity of LWC might be low at room temperature but increases or remain unchanged as temperature increases whereas normal weight aggregate is crystalline and exhibits high thermal conductivity at room temperature but decreases with increase in temperature (EC4, 2002).

Conclusively, according to Jacob’s statement, the differences between thermal conductivities of different types of lightweight aggregates in a concrete mix may be related to the proportion of ‘glassy’ materials present. Because, results obtained from glassy material shows less thermal conductivity value than crystalline materials.

1.2.6.3 Mineral Admixture

The effect of mineral admixture on thermal conductivity is relatively important when it needs to be use as partial replacements in the total binder content. The use of admixture has been advanced in many ways; especially in construction industry it improves the thermal isolation and decrease the environmental contamination. Reported in research articles, compared with controlled samples increasing admixture content shows decreasing thermal conductivity. Increasing
Chapter 1: Introduction

silica fume and fly ash percentage by weight of cement content showed decreasing dry unit weight of concrete and increasing air void content (Ramazan et al., 2003). Fly ash is more effective than silica fume for decreasing the thermal conductivity.

1.2.6.4 Temperature

As discussed early, thermal performance of concrete depends on aggregate’s internal microstructures and its mineralogical compositions. Generally, thermal conductivity of concrete is independent of temperature at ambient conditions but it begins to decrease linearly at elevated temperature more than 100°C. The reason is because concrete starts to decrease its moisture content present at higher temperature (Navy, 2001). The conductivity of highly crystalline aggregates is high at room temperature and decreases with elevated temperature.

Concretes made-up of amorphous aggregate have shown low conductivity at room temperature and slightly higher conductivity as the temperature rises. Shin et al., (2002) revealed that conductivity of concrete decreases with increasing temperature. But beyond 900°C, the measured thermal conductivity is approximately equal to 50% of conductivity at ambient temperature. Thermal conductivity of concrete was reported at various densities and wide temperature ranges (Singh and Garg 1991). The correlation between thermal conductivity values and density at various mean temperatures are shown in Fig.1.2.
1.2.6.5 Curing age

Thermal conductivity of concrete does not varying significantly with curing age (Blanco et al., 2000; Kook et al., 2003). They revealed that concrete thermal conductivity is independent of curing age but considerable changes were observed due to difference in ingredients (Kim et al., 2003). Thermal conductivity of Cenosphere was tested for 5 days to 28 days of curing period. The results showed that the measured thermal conductivity remained almost the same for the tested curing age.

Gibbon and Ballin (1998) studied the thermal conductivity of concrete at early age with their specially prepared probe. The predicted thermal conductivity significantly varied due to variation in binder content, W/C ratio and aggregates but less variation was observed with age (Khan 2002). Cook and Uher (1974)
investigated the effect of adding copper and steel fibers on thermal conductivity. Their results indicated that adding both copper and steel fibre increases thermal conductivity of concrete but steel fibers had lesser effect. Sweeting and Liu (2004) measured the thermal conductivity of composite laminates. Thermal conductivity along in-plane was approximately four times greater than the through thickness conductivity for composites laminates.

1.3 Mechanical properties of concrete

Mechanical properties of concrete is essential to predict the thermal stress development in mass concrete elements. During the period in which concrete changes from almost liquid state to solid state, most of the mechanical properties rapidly vary with respect to age of concrete. Of these, modulus of elasticity, development of tensile strength and creep behavior are key parameters implemented in thermal stress analysis.

1.3.1 Modulus of Elasticity

At early age concrete starts to gain strength and stiffness, which increases with time. Concrete has more inelastic strains at 3 to 4 hours and also most of the deformations are permanent (Berggstrom et al., 1980). The well defined inelastic and elastic regions develop at age of 8 to 10 hours and in the range of 14 to 18 hours concrete shows harden concrete behavior.

At present, generalized models are available to predict the development of modulus of elasticity based on degree of hydration or maturity concepts and apparent setting time ($t_s$). It is necessary to have a model to predict actual material
behavior. Cervera et al., (1999) introduced the concept of aging degree \((k)\) which depends on hydration degree and kinematics of hydration reaction to predict the strength. CEB-FIP model code (1993) has proposed an equation to express the modulus of elasticity at an age which is not greater than 28 days with modified age coefficient \(\beta_{E}(t)\) as

\[
E_{ci}(t_0) = \beta_{E}(t_0) E_{ci}
\]

where \(\beta_{E}(t_0) = \left[ \exp \left( s \left[ 1 - 1/ \left(\frac{t_0}{28} - \frac{1}{28-1} \right)^{1/2} \right] \right) \right]^{1/2}\), \(E_{ci}(t)\) is the modulus of elasticity at an age \(t\) days, \(E_{ci}\) the modulus of elasticity at 28 days, and \(t_0\) and \(t_s\) are time equivalent age and apparent setting time in days. Another consistent model proposed by Larson and Jonasson (2003) to calculate the modulus of elasticity at time \(t_0\) by means of linear curves may be expressed as Eq. (1.8);

\[
E(t_0) = E_{ref} \times \beta_{E}(t_0)
\]

where \(E_{ref}\) is the modulus of elasticity at 28 days age chosen as reference value and \(\beta_{E}(t_0)\) is to define the material behavior by piece-wise linear curves and expressed as

\[
\beta_{E}(t_0) = \begin{cases} 
0 & \text{for } t_0 < t_s \\
 b_1 \times \log \left( \frac{t_0}{t_s} \right) & \text{for } t_s \leq t_0 < t_B \\
 b_1 \times \log \left( \frac{t_0}{t_s} \right) + b_2 \times \log \left( \frac{t_0}{t_B} \right) & \text{for } t_B \leq t_0 < 28 \text{ days} \\
1 & \text{for } t_0 \geq 28 \text{ days}
\end{cases}
\]
Chapter 1: Introduction

$t_0, b_1$ and $b_2$ are the model parameters which are to be evaluated from the laboratory tests. Shutter and Taerwe (1996) proposed a hypothesis based on degree of reaction ($r$) to evaluate the modulus of elasticity in Eq. (1.9);

$$\frac{E_{e0}(r)}{E_{e0}(r = 1)} = \left(\frac{r - r_0}{1 - r_0}\right)^b$$

(1.9)

Parameters such as, $r$, $r_0$ and $b$ depend on the concrete composition and the modulus of elasticity $E_{e0}(r)$ and $E_{e0}(r = 1)$ are at degree of reaction $r$ and $r = 1$. Degree of reaction ($r$) varies from 0 at fresh concrete state and 1 when complete hydration has taken place.

1.3.2 Tensile strength of concrete

Low development of tensile strength causes higher risk of cracking at early ages. Tensile strength of concrete $f_{ct}$ can be expressed as a function of the degree of hydration $r$ with model parameter $c$ as (Shutter and Taerwe, 1996) Eq. (1.10)

$$f_{ct}(r) = \begin{cases} f_{ct}(r = 1) \left(\frac{r - r_0}{1 - r_0}\right)^c & \text{if } r_0 \leq r < 1 \\ 0 & \text{if } 0 \leq r < r_0 \end{cases}$$

(1.10)

CEB-FIP model code (1993) also proposed tensile strength calculation with coefficient describing the development of strength with time $\beta_{ct}(t)$ and tensile strength $f_{ct,28}$ at age of 28 days as the following Eq. (1.11)

$$f_{ct}(t_0) = \beta_{ct}(t) f_{ct,28}$$

(1.11)
Chapter 1: Introduction

Further, CEB-FIP stated that the above equation overestimated the tensile strength for an age below 28 days because it depends on compressive strength by curing and member size.

1.3.3 Creep behavior of young concrete

Prediction of creep behavior at early age with acceptable accuracy is important for thermal stress calculation. It was estimated that approximately 40-50% of elastically induced stresses decreases with creep effect for fully restrained conditions. Creep in concrete at early age seems to be one of the most influencing parameter and is important to be predicted accurately for thermal stress estimation (Umehara et al., 1994; Yuan et al., 2002).

Neville et al., (1983) described several methods of creep calculation, influencing parameters and experimental procedures. Several authors proposed their creep model as a function of creep compliance with constant stress loading history. Bazant (1972) proposed the creep response based on solidification theory but which fails to represent the early age creep behavior.

Creep calculation can be done in two ways. First method is based on theory of linear visco-elasticity applied through the principle of superposition. This method considers the global response of concrete subjected to any loading history, either loading or unloading. Second method is based on an incremental formulation. This incremental creep formulation is allowed to define nonlinear effect with respect to the stress and consider the effects of time dependent physical variables.

Guenot et al., (1996) studied the creep model based on linear visco-elasticity by step-by-step numerical process and compared with existing creep models. Hattel
and Thorborg (2003) applied the creep strain increment which was calculated from creep strain rate in their numerical formulation. Creep calculation based on CEB-FIP model code (1993) gives quite good results at early ages but it needs experimental data additionally. Schutter (2002) has implemented the visco-elastic behavior of hardening concrete by means of degree of hydration based on Kelvin chain model and the results were verified with experimentally conducted creep tests. It was necessary to use the creep compliance function with varying loading history. Experimentally, creep strains can be calculated as per ASTM C512-76 method of test for creep of concrete in compression.

An alternative way is to prefer the incremental creep model in which, stress and strain increments are carried out perfectly at each time step. Apart from that, the characteristics of instantaneous elastic deformation and creep function should be considered in the constitutive law while predicting the time dependent stress behavior. Still, tensile and compressive creep behavior of concrete was considered to be equal to each other but tensile creep greatly affects the early age cracking (Mihashi et al., 2004).

Morimoto and Koyanagi (1994) reported that compressive and tensile relaxations are purely proportional to the initial stresses. Under constant load, Lennart et al., (2001) have studied the tensile basic creep response which was observed to be higher creep response at early age. Gutsch (2000) revealed the importance of maturity concept for calculating basic creep behavior at early age. For mass concreting problems, drying creep is less importance than basic creep because there is no significant amount of moisture exchange between structure and environments (ETL report, 1997).
Chapter 1: Introduction

At early age, Schutter (2003) confirmed that there is elementary coupling between creep response and heat of hydration. Linear logarithmic model (LLM) has been developed to predict the creep behavior of both young and mature concrete at all loading ages and load durations (Larsson and Jonasson, 2003). This model was developed on the basis of prescribing actual behavior of material properties with considerable accuracy. The rate of creep strain increments has been calculated (Larsson and Jonasson, 2003) from time dependent deformation $\varepsilon^c(t, t_o)$ which may be expressed in terms of creep compliance and loading stress $\sigma_c(t_o)$ as in Eq. (1.12).

$$\varepsilon^c(t, t_o) = J(\Delta t_{\text{load}}, t_o) \times \sigma_c(t_o)$$  \hspace{1cm} (1.12)

In Eq. (1.12), all the model parameters and functions are defined according to the reference mentioned. The above Eq. (1.12) can be converted into relaxation function. Neville et al., (1983) commented that after evaluating various creep methods, aging coefficient is a powerful tool to solve all common problems in creep analysis.

Trost (1967) has developed the practical method of predicting strain under varying stress or constant stress. Later, Bazant (1972) made improvement on Trost method which includes the age coefficients. Based on their proposal, relaxation function $R(t, t_o)$ can be defined with assuming constant aging coefficient $\chi(t, t_o)$.

Relaxation function $R(t, t_o)$ can be expressed as Eq. (1.13).

$$R(\Delta t_{\text{load}}, t_o) = E(t_o) \times \left(1 - \frac{\varphi(\Delta t_{\text{load}}, t_o)}{1 + \chi \times \varphi(\Delta t_{\text{load}}, t_o)}\right)$$  \hspace{1cm} (1.13)

Where $\varphi(\Delta t_{\text{load}}, t_o)$ is defined as creep function.
Chapter 1: Introduction

In some literatures, the creep function is frequently denoted by \( J(\Delta t_{load}, t_0) \) instead. Saucier et al., (1997) has indicated the benefits of relaxation of concrete in tension which may reduce the tensile stresses caused by the internal and external restraints.

1.4 Heat of hydration

As the cement hydration process begins, it produces considerable amount of heat. The heat evolution of hydration processes increases the temperature in mass concrete substantially (Springenschmid, 1995). During the concrete construction, the heat is dissipated into the soil and the air and resulting temperature changes within the structures are not significant.

However, in some situations, particularly mass concrete structures, such as dams, mat foundations, or any element more than about a meter thick, heat dissipation can’t be readily released. The mass concrete may then attain high internal temperature, especially during hot weather construction, or if high cement contents are used. The factors influencing heat development in concrete include the cement content, cement fineness, water cement ratio, placing and curing temperature, presence of mineral and chemical admixtures, and the dimensions of the structural element.

Donnell et al., (2003) developed a new methodology so called prism method to predict the total quantity of heat generated based on temperature measurements. The model was developed for blast furnace slag cements which accounts for the composed character of cement and compared with mass concrete cylinders (Schutter, 1999). Swaddiwudhipong et.al., (2002) developed the multi constituent model to
Chapter 1: Introduction

describe the rate of heat of hydration and also indicated the factor required to
account for the effect of cement on hydration. Experimentally, several adiabatic
hydration tests and isothermal tests were conducted to forecast total quantity of heat
generated.

1.5  Restraint conditions

When a concrete is prevented from moving freely due to free strains
development, stresses are created by the “restrained strain”. Since restraining
actions may lead to severe cracks, the realistic assessment of the degree of restraint
is important. Stresses develop as strain due to cooling of concrete is prevented. The
major tensile stresses calculated in this approach are likely to be overestimated. No
tensile stress would develop if the length or volume changes, associated with
decreasing temperature within a concrete mass, take place freely (ACI 207.4R,
1993). If not, restraint thus acts to limit the change in dimensions and induces
stresses in the concrete member. Such thermal stresses may eventually cause the
cracking. Harrison (1992) proposed a simplified method for predicting restrained
strains based on the assumption that concrete sets at the peak temperature and
contains no induced stresses at that time. Broadly speaking, restraint can be
classified into three types such as external restraint, internal restraint and secondary
restraint.

1.5.1  Internal restraint

It is caused by non-uniform temperature change within a concrete member
which produces Eigen stresses. Concrete portions with low temperature rise or fall
Chapter 1: Introduction

restrain parts with high temperature rise or fall, because the latter tend to expand or contract more than the former. Experience has shown that by limiting the temperature differential to 20°C, cracking can be avoided for concrete with basalt aggregate (Fitzgibbon, 1976). BS 8110 (1985), as well as Bamforth and Price (1995) used 0.36 as the internal restraint factor to calculate the limiting temperature differential.

1.5.2 External restraint

It is imposed due to the boundary conditions restraining the volume change of concrete members (BS 8110, 1985 and Bamforth, 1982). The external restraint factor in a concrete wall poured on to a rigid base is 1, however, this value is not uniform and it depends on the location. The values of restraint factor depend particularly on the difference in stiffness between the restraining body and the concrete member.

ACI 207 (1989) gives a more detailed approach for estimating restraint factors in relation to the length and height ratio. External restraint can be further sub-divided into end restraint and continuous edge restraint although in a given situation it is often a combination of the two. Usually one or the other of these forms of restraint is dominant (CIRIA, 1991). This report has attributed a theoretical value to restrained factor (R) which was based on total restraint (R=1) against an existing restraint and no restraint (R=0) on a free edge. In practice, this figure can be reduced by 50 % to take account of internal creep of the concrete. However there are conditions under which a combination of these two can occur. There are also conditions in which partial or intermittent restraint occurs for actual conditions.
1.5.3 Restraint Factor

As the concrete contracts it is restrained by adjacent structures such as foundations and older pours. The extent of this restraint is denoted by a restraint factor (PSA, 1982) as Eq. (1.14)

\[ TSC = \alpha(T_p - T_a) R \]  

(1.14)

where TSC is tensile strain capacity of concrete, \( \alpha \) the coefficient of thermal expansion of the concrete and, \( T_p \) and \( T_a \) the peak temperature at time of striking of formwork and ambient temperature, respectively. Degree of restraint factor depends primarily on the relative dimensions, strength and modulus of elasticity of the concrete and the restraining material. The restraint factor for a concrete element R may be defined as Eq. (1.15)

\[ R = \frac{\text{Free Contraction} - \text{Actual Contraction}}{\text{Free contraction}} \]  

(1.15)

Bamforth, (1982) predicted the probability of thermal cracking based on knowledge and development of free thermal strain \( \epsilon_j \) during temperature cooling down with temperature differential \( \Delta T \) and thermal expansion coefficient \( \alpha \). The restrained component of strain \( \epsilon_R \) was calculated as the difference of total free strain and actual movement (\( \epsilon_m \)) as in Eq. (1.16).

\[ \epsilon_R = \epsilon_j - \epsilon_m = \alpha \Delta T R \]  

(1.16)

Simplified elastic approaches are used to describe restraint coefficient with reasonable accuracy ACI (1973). Larson (2000) has formulated the concept of restraint coefficient based on viscoelastic approach and defined as \( \gamma_R(t) \) in Eq. (1.17), Eq. (1.18) and Eq.1.19)
Chapter 1: Introduction

\[
\gamma_R(t) = \frac{\sigma(t)}{\sigma_{fix}(t)}
\]  

(1.17)

\[
\sigma(t) = \int_{t} R(t,t_0) d\varepsilon(t) + \sigma_{fix}(t)
\]  

(1.18)

\[
\sigma_{fix}(t) = - \int_{t} R(t,t_0) d\varepsilon^0(t),
\]  

(1.19)

where \(\sigma(t)\) and \(\sigma_{fix}(t)\) are stress development at specific point of the structure and fixation stress for \(\varepsilon(t) = 0\) at time \(t\) respectively. \(R(t,t_0)\) is the relaxation function, \(\varepsilon^0(t)\) and \(\varepsilon(t)\) are the total free strain and measurable strain at time \(t\).

1.6 Finite Difference Method

The temperature development in mass concrete can be predicted from general heat transfer equation for mostly regular shaped concrete elements. The three dimensional temperature profile of concrete at early age due to cement hydration and ambient conditions can be calculated by following Fourier differential equation (J P Holman, 1992) in Eq. (1.20)

\[
k_x \frac{\partial^2 T}{\partial X^2} + k_y \frac{\partial^2 T}{\partial Y^2} + k_z \frac{\partial^2 T}{\partial Z^2} + Q_h(t,T) = c \rho \frac{\partial T}{\partial t},
\]  

(1.20)

where \(k_x, k_y\) and \(k_z\) are the thermal conductivities of concrete in the respective coordinates in W/mC, \(c\) the specific heat capacity in J/kgC, \(\rho\) the density of concrete in kg/m\(^3\), \(\alpha\) the thermal diffusivity of concrete in m\(^2\)/hour, \(Q_h(t,T)\) the heat generated due to cement hydration and external sources in W/m\(^3\), \(T\) the temperature of concrete in °C and \(t\) the elapsed time in hours.

Using central operator and equal spacing in along \(X, Y, Z\) rectangular coordinates, the finite difference method of equation (1.21) can be rearranged into
Chapter 1: Introduction

\[
T_{i,j,k+l} = \left( 1 - \frac{6\alpha \Delta t}{\Delta x^2} \right) T_{i,j,k} + \frac{x}{\Delta x^2} \left( T_{i-1,j,k} + T_{i+1,j,k} + T_{i,j-1,k} + T_{i,j+1,k} + T_{i,j,k-1} + T_{i,j,k+1} \right) + Q_{i,j,k}(t,T)
\]  

(1.21)

Thermal conductivity of concrete is assumed to be constant along all directions in the above equation. The temperature distribution can be determined on basis of adiabatic temperature rise curve and suitable boundary conditions.

1.7 Finite element simulation

In general, the process of incremental numerical simulation was split into two parts to define realistic behavior of mass raft foundation: 1) thermal problem which is related to predicting temperature field using relevant thermal characteristics, and 2) mechanical problem which is related to determine residual thermal stresses on basis of temperature field and time dependent mechanical behavior. Various finite element simulations are used worldwide to describe temperature distributions and stress analysis of mass concrete behavior such as ABAQUS, NISA, DIANA, HIPERPAV, FEMMASSE and FE etc.

The finite element solution is performed when mechanical and thermal solutions affect each other strongly and must be obtained simultaneously based on time dependent material response. Schutter (2002) developed a finite element program to analyse concrete armour units using degree of hydration based time dependent materials laws and compared with experimental tests by Cervera et al., (2002). Semi coupled, incremental thermo mechanical model proposed by Hattel and Thorborg (2003) was based on maturity concepts. Yuan and Wan (2002) used the three dimensional numerical program called Concrete Cracking Control (CCC) in which their model accounted for the effects of hydration, moisture transport and
Chapter 1: Introduction


1.8 Prediction of early age thermal cracking

There are three ways to predicting risk of thermal cracking such as temperature based approach, strain based approach and stress based approach. Temperature based criterion is the simplest way to predict the risk of thermal cracking but it was pointed out that there is no fundamental relation between temperature difference and stress level (Springenschmid, 1998). Bamforth (1981) stated limiting temperature differential causes the cracking and can be measured in terms of tensile strain capacity, restraint factor and thermal expansion.

In CIRIA report, Harrison (1991) described the limiting strain criterion to predict the occurrence of cracking if the restrained tensile strain induced by differential temperature exceeds the tensile strain capacity of concrete and the same limiting strain concepts was followed by Hunt (1971). But they have an assumption that no stresses developed during the heating phase which leads to the overestimation of tensile stresses. The total strain components in the concrete can be decomposed in to stress related and stress unrelated.

The stress related components includes creep strain $\varepsilon_{cs}$ and elastic strain $\varepsilon_{el}$ whereas, thermal strain $\varepsilon_{th}$ and autogeneous shrinkage $\varepsilon_{as}$ are stress unrelated part. Restrained strain $\varepsilon'$ can be calculated from the difference between the sum of thermal shrinkage and autogenous shrinkage i.e. total free strain, and measured strain in that instant and any drop indicates the formation of cracking (Larson et al.,
2003). The actual stress $\sigma(t)$ at time $t$ can be calculated from the applied restrain strain at time $t_0$ as in Eq. (1.22)

$$\sigma(t) = \int_0^t R(t,t_0) \, d\epsilon(t)$$

(1.22)

The abovementioned equation is employed to predict the total stress and it can be compared with actual tensile strength of concrete. However, cracking occurs when predicted tensile stress of concrete exceeds measured tensile strength.
1.9 Objective and Scope

The objective of this research is to study the early age thermal stress development in mass concrete elements as precursor to the evaluation of risk of cracking. Focus would be placed on early-age material properties and the development of finite element simulation which will be verified with field data. Also, thermal properties of various concrete would be investigated.

The scope of this study includes:

(1) An experimental investigation on the coefficient of thermal expansion and thermal conductivity of lightweight concretes (LWC) and normal weight concretes (NWC)

(2) Development of an innovative thermal conductivity system for calculating the thermal conductivity of various concrete accurately.

(3) Development of a new experimental method to measure thermal diffusivity of concrete at early age for prediction of thermal cracking. The method will be verified against numerical studies.

(4) Finite element simulation of early age thermal stress development in mass raft foundation.

CHAPTER 2

THERMAL PROPERTIES OF VARIOUS CONCRETE

Thermal properties of concrete involve the process of heat transfer in predicting the temperature and heat flow through concrete material. This chapter covers the study of coefficient of thermal expansion (CTE) and thermal conductivity (TC) of hardened concrete. A new thermal conductivity system has been developed to overcome the shortcomings in exiting methods which is discussed in detail in next chapter.

2.1 Laboratory work

2.1.1 Materials

In this study, following cementitious materials were used namely, ordinary Portland cement (OPC), ground granulated blast furnace slag (GGBS) and silica fume (SF). The river sand as fine aggregate used for concrete mixes confirmed to the M-grading of BS 882:1992. Concretes were prepared with various types of lightweight aggregate such as pumice, leca, liapour (expanded clay aggregates) and crushed granite.

2.1.2 Mix proportions

The tables (Table 2.1 to Table 2.6) show the mix proportions chosen in this study. Foam concretes were prepared with and without sand in order to study the
Chapter 2: Thermal properties of Various Concrete

effect of sand content at varying densities. Concretes were prepared using Leca 900, liapour 8 and pumice of various sizes (5mm, 10mm and 20mm) and normal aggregates.

**Table 2.1 Mix proportion for Foam concrete without sand**

<table>
<thead>
<tr>
<th>Designation/ Fresh Density</th>
<th>Dry Density kg/m³</th>
<th>Cement kg/m³</th>
<th>GGBS kg/m³</th>
<th>Fine aggregate kg/m³</th>
<th>Water kg/m³</th>
<th>w/cm</th>
<th>Volume of Foam kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>FC1-800</td>
<td>582</td>
<td>291</td>
<td>291</td>
<td>-</td>
<td>175</td>
<td>0.60</td>
<td>0.633</td>
</tr>
<tr>
<td>FC2-1000</td>
<td>790</td>
<td>370</td>
<td>370</td>
<td>-</td>
<td>222</td>
<td>0.60</td>
<td>0.532</td>
</tr>
<tr>
<td>FC3-1300</td>
<td>1103</td>
<td>490</td>
<td>490</td>
<td>-</td>
<td>294</td>
<td>0.60</td>
<td>0.382</td>
</tr>
<tr>
<td>FC4-1600</td>
<td>1415</td>
<td>609</td>
<td>609</td>
<td>-</td>
<td>366</td>
<td>0.60</td>
<td>0.231</td>
</tr>
<tr>
<td>FC5-1900</td>
<td>1727</td>
<td>729</td>
<td>729</td>
<td>-</td>
<td>437</td>
<td>0.60</td>
<td>0.0803</td>
</tr>
<tr>
<td>FC – Foam Concrete without sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 2.2 Mix proportion for Foam concrete with sand**

<table>
<thead>
<tr>
<th>Designation/ Fresh Density</th>
<th>Dry Density kg/m³</th>
<th>Cement kg/m³</th>
<th>GGBS kg/m³</th>
<th>Fine aggregate kg/m³</th>
<th>Water kg/m³</th>
<th>w/cm</th>
<th>Volume of Foam kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>FSC1-1000</td>
<td>790</td>
<td>310</td>
<td>310</td>
<td>155</td>
<td>186</td>
<td>0.60</td>
<td>0.549</td>
</tr>
<tr>
<td>FSC2-1200</td>
<td>999</td>
<td>324</td>
<td>324</td>
<td>324</td>
<td>195</td>
<td>0.60</td>
<td>0.468</td>
</tr>
<tr>
<td>FSC3-1400</td>
<td>1207</td>
<td>382</td>
<td>382</td>
<td>382</td>
<td>229</td>
<td>0.60</td>
<td>0.374</td>
</tr>
<tr>
<td>FSC4-1600</td>
<td>1415</td>
<td>343</td>
<td>343</td>
<td>686</td>
<td>206</td>
<td>0.60</td>
<td>0.308</td>
</tr>
<tr>
<td>FSC5-1800</td>
<td>1623</td>
<td>319</td>
<td>319</td>
<td>956</td>
<td>191</td>
<td>0.60</td>
<td>0.237</td>
</tr>
<tr>
<td>FSC6-2000</td>
<td>1831</td>
<td>355</td>
<td>355</td>
<td>1066</td>
<td>213</td>
<td>0.60</td>
<td>0.149</td>
</tr>
<tr>
<td>FSC – Foam Concrete with Sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 2.3 Mix proportion for high strength lightweight concrete**

<table>
<thead>
<tr>
<th>Designation/ Fresh Density</th>
<th>Dry Density kg/m³</th>
<th>Cement kg/m³</th>
<th>GGBS kg/m³</th>
<th>SF kg/m³</th>
<th>Water kg/m³</th>
<th>w/(c+SF)</th>
<th>Fine aggregate kg/m³</th>
<th>LWA kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIC1-2000</td>
<td>1836</td>
<td>500</td>
<td>0</td>
<td>50</td>
<td>165</td>
<td>0.30</td>
<td>601</td>
<td>558</td>
</tr>
<tr>
<td>LIC2-2000</td>
<td>1839</td>
<td>500</td>
<td>0</td>
<td>100</td>
<td>150</td>
<td>0.25</td>
<td>600</td>
<td>557</td>
</tr>
<tr>
<td>LC1-2000</td>
<td>1805</td>
<td>250</td>
<td>200</td>
<td>50</td>
<td>150</td>
<td>0.30</td>
<td>629</td>
<td>629</td>
</tr>
<tr>
<td>LC2-2000</td>
<td>1827</td>
<td>500</td>
<td>0</td>
<td>50</td>
<td>165</td>
<td>0.30</td>
<td>601</td>
<td>601</td>
</tr>
<tr>
<td>LC3-2000</td>
<td>1859</td>
<td>300</td>
<td>200</td>
<td>0</td>
<td>150</td>
<td>0.30</td>
<td>629</td>
<td>629</td>
</tr>
<tr>
<td>LC4-2000</td>
<td>1872</td>
<td>500</td>
<td>0</td>
<td>0</td>
<td>150</td>
<td>0.30</td>
<td>629</td>
<td>629</td>
</tr>
<tr>
<td>LIC – Liapour-8 Concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LC – Leca-900 Concrete</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**Table 2.4 Mix proportion for Pumice lightweight concrete**

<table>
<thead>
<tr>
<th>Designation/ Fresh Density</th>
<th>Dry Density</th>
<th>Cement</th>
<th>GGBS</th>
<th>SF</th>
<th>Air Content</th>
<th>Water</th>
<th>w/(c+SF)</th>
<th>Fine aggregate</th>
<th>LWA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kg/m³</td>
<td>kg/m³</td>
<td>kg/m³</td>
<td>kg/m³</td>
<td>%</td>
<td>kg/m³</td>
<td>-</td>
<td>kg/m³</td>
<td>kg/m³</td>
</tr>
<tr>
<td>PC1-1600</td>
<td>1495</td>
<td>225</td>
<td>225</td>
<td>0</td>
<td>6</td>
<td>150</td>
<td>0.67</td>
<td>656</td>
<td>454</td>
</tr>
<tr>
<td>PC2-1800</td>
<td>1681</td>
<td>450</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>150</td>
<td>0.33</td>
<td>763</td>
<td>453</td>
</tr>
</tbody>
</table>

PC – Pumice Concrete; LWA – 5mm pumice + 10mm pumice + 20mm pumice

**Table 2.5 Mix proportion for Normal weight concrete**

<table>
<thead>
<tr>
<th>Designation/ Fresh Density</th>
<th>Dry Density</th>
<th>PBFC</th>
<th>Water</th>
<th>w/cm</th>
<th>Fine aggregate</th>
<th>Coarse aggregate</th>
<th>Admixture Mira 99</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Kg/m³</td>
<td>kg/m³</td>
<td>kg/m³</td>
<td>-</td>
<td>kg/m³</td>
<td>kg/m³</td>
<td>ltrs</td>
</tr>
<tr>
<td>NWC-2420</td>
<td>2328</td>
<td>380</td>
<td>160</td>
<td>0.42</td>
<td>780</td>
<td>1000</td>
<td>3.42</td>
</tr>
</tbody>
</table>

NWC – Normal Weight Concrete; PBFC - Portland Blast Furnace Cement

**Table 2.6 Properties of Lightweight Aggregates (LWA)**

<table>
<thead>
<tr>
<th>Types of Lightweight Aggregates (LWA)</th>
<th>Aggregate size (mm)</th>
<th>Dry particle density (kg/m³)</th>
<th>Water absorption (%)</th>
<th>Porosity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>1 hour</td>
<td>24 hour</td>
</tr>
<tr>
<td>Leca 900 (L9)</td>
<td>20</td>
<td>1395</td>
<td>5.11</td>
<td>8.09</td>
</tr>
<tr>
<td>Leca 800 (L8)</td>
<td>20</td>
<td>1270</td>
<td>8</td>
<td>14.5</td>
</tr>
<tr>
<td>Leca 700 (L7)</td>
<td>20</td>
<td>1146</td>
<td>6.49</td>
<td>9.78</td>
</tr>
<tr>
<td>Leca 600 (L6)</td>
<td>20</td>
<td>968</td>
<td>7.11</td>
<td>12.1</td>
</tr>
<tr>
<td>Leca 500 (L5)</td>
<td>20</td>
<td>889</td>
<td>7.81</td>
<td>12.26</td>
</tr>
<tr>
<td>Pumice (Pum)</td>
<td>20</td>
<td>641</td>
<td>67.23</td>
<td>71.64</td>
</tr>
<tr>
<td>EPS (EPS)</td>
<td>3</td>
<td>29</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>L9 (5-10 mm)</td>
<td>10</td>
<td>1440</td>
<td>6.58</td>
<td>8.23</td>
</tr>
<tr>
<td>L9 (10-20 mm)</td>
<td>20</td>
<td>1468</td>
<td>6.2</td>
<td>7.96</td>
</tr>
</tbody>
</table>
Fig. 2.1 Preparation of test specimen for thermal expansion test

Fig. 2.2 Preparation of test specimen for thermal conductivity test
2.1.3 Test specimens preparation

The thermal expansion tests were conducted on prisms of size 100x100x400mm which were prepared for each concrete mix (Fig.2.1). Thermal conductivity tests were carried out on specimen size of 305 x 305 x 75mm 28 days after casting (Fig.2.2). After casting all specimens were properly covered with wet burlap. For each concrete mix, three specimens for thermal expansion and two specimens for thermal conductivity test were prepared. 24 hours after casting, specimens were demoulded and kept in the fog room for moist curing.

2.2 Thermal properties - Test Methods

2.2.1 Thermal Expansion Test

This method deals with determination of CTE in concrete. The coefficient of thermal expansion can be determined by dividing the change in length by change in

![Demec strain gauge](image_url)

Fig.2.3 Demec strain gauge employed for measuring the change in length.
temperature. The thermal expansion of concrete was measured by varying the temperature (40°C to 60°C) for different densities. The measured coefficient of thermal expansion is to be corrected for shrinkage in concrete. The change in length of the measured specimens, due to change in temperature was measured using Demec strain gauge (Fig. 2.3).

The test involves the measurement of change in length of prismatic specimens at each temperature interval at intervals of 24 hours. The change in length between the pivoted points i.e. demec pin points on each test specimens was measured under thermal balance.

The measurements at each temperature level were observed quickly to avoid the temperature of oven to drop below the defined temperature level. Average coefficient of thermal expansion was measured for defined temperature level. During testing, thermocouples were used for measuring the temperature of concrete. The samples were oven dried at 105°C for 3 days before testing in order to reduce shrinkage effect. After demoulding, prismatic specimens were stored in a fog room until age of testing. Then, demec pins were glued onto the surface of the specimens as shown in Fig. (2.1). The epoxy used had the ability to withstand temperature of about 120°C. Specimens were dried for an hour and thereafter measurement of initial readings was done.

### 2.2.2 Thermal Conductivity Test

Guarded hot plate (Model GHP-300) thermal conductivity system was used to measure the thermal conductivity of oven dried concrete which is relatively suitable for low thermal conductance. Using this apparatus, two identical specimens can be tested at a time. Fig.2.4 shows test set up of GHP 300-Model. During the test,
Chapter 2: Thermal properties of Various Concrete

the test stack was surrounded by sheet metal enclosure which can be removed completely to allow access to stack heaters and specimens from all sides. In order to prevent the excessive heat loss from the edge of heaters and test specimens, vermiculite was used to fill the gap between the sheet metal enclosure and test stack.

Vacuum cleaner was used particularly to remove the vermiculite at end of testing process. For normal testing to attain the state of thermal equilibrium, it takes minimum duration of about 5 to 10 hour.

Fig.2.4 Guarded Hot Plate (GHP-300) thermal conductivity system

Before starting the test, it is necessary to calculate the approximate power required for the test by presuming a thermal conductivity value of the material to be tested in order to reduce duration of test, using Eq. (2.1)

\[ Q = EI = k \times S \left[ \frac{\Delta T}{d} \right]_{\text{upper}} + \left[ \frac{\Delta T}{d} \right]_{\text{lower}} \]  

(2.1)

where \( Q \) is the input power of the main heater in Watts, \( E \) the voltage reading in Volts, \( I \) the current reading in Amperes, \( k \) the presumed thermal conductivity of the two identical test samples in W/mK, \( S \) the cross sectional area of the main
heater in \( m^2 \) \( (S=0.0232 \ m^2) \), \( \Delta T \) the temperature gradient through the sample in \( ^\circ C \) and \( d \) the sample thickness in m. Temperature of cold side was set at control console board whereas hot temperature was achieved through the main heater. The main heater voltage was then adjusted manually until reaching hot side temperature. At least four hours were required for temperature to reach thermal steady state conditions. Thereafter, at every half an hour interval, the hot side temperature was noted until the required temperature at hot side was reached. Temperature at hot side as well as cold side and the power required were noted while ensuring that temperature readings were no longer increasing or decreasing continuously. The effective thermal conductivity of concrete was calculated from Eq. (2.2),

\[
k_{eff} = \frac{EI}{S} \times \frac{1}{\left( \frac{\Delta T}{d} \right)_{upper} + \left( \frac{\Delta T}{d} \right)_{lower}},
\]

(2.2)

2.3 Results and discussions

2.3.1 Thermal expansion

As discussed in literature review, thermal expansion of concrete varies with aggregate type, cementitious binder, age, sand, density and temperature range. A series of experimental measurements were performed to measure the thermal expansion in consideration of the above mentioned influencing parameters. Results showed that the measured thermal expansion of concrete made up of foam, leca, liapour and pumice were lower than that of normal weight aggregate concrete. From Fig. 2.5, the concrete mixes showed that CTE directly varied with density as pointed out in ACI 523 (ACI 523, 1992). CTE of concretes increases with density despite the type of concretes as shown in Fig. 2.5 to Fig. 2.7. It was reported that lightweight
Chapter 2: Thermal properties of Various Concrete

![Graph 2.5](image1.png)

Fig. 2.5 Relationship between CTE of concrete and density.

![Graph 2.6](image2.png)

Fig. 2.6 CTE of Foam concrete (with and without sand) at 40°C, 50°C and 60°C
Chapter 2: Thermal properties of Various Concrete

Fig. 2.7 CTE of Liapor concrete and Leca concrete varying with temperature

Fig. 2.8 CTE of pumice concrete and NWC varying with temperature
Chapter 2: Thermal properties of Various Concrete

concretes made up of expanded shale and clay aggregate was about 50-70% lower than the gravel aggregate (FIP Manual, 1983). The measured coefficient of thermal expansion of concrete made of Leca and Liapour were approximately 10~15 % lower than normal weight concrete whereas foam concrete with and without sand varied approximately 10~50 %. This may be due to lesser amount of sand used in lightweight concrete mixes compared to normal weight concrete. Hence, it reduces the thermal expansion of lightweight concrete considerably. Thus the significant change is due to the effect of sand content in the foam concrete (Fig.2.6). Thermal expansion of foam concrete with and without sand indicates that the presence of sand slightly increases the CTE. For all type of concretes, it was observed that thermal expansion increases considerably with temperature (Fig.2.6 to Fig.2.8).

![Fig.2.9 Relationship between thermal conductivity of LWC and oven dry densities](image)

\[ k = 0.090e^{0.0012x} \]

Fig.2.9 Relationship between thermal conductivity of LWC and oven dry densities
2.3.2 Thermal conductivity

Thermal conductivity of various types of concrete was studied. From the results, it is concluded that concrete having higher density shows higher thermal conductivity and it also depends on variations between concrete mixes and different raw materials. Fig.2.9 shows the relationship between thermal conductivity of different types of concrete (Foam concrete, Pumice concrete, Leca and Liapour) and its dry density. Based on this study, the general equation obtained in terms of oven dry density $\rho$ is given in (Eq.2.3).

$$ k = 0.090e^{0.0012 \times \rho} $$

(Density is one of the primary influencing parameter and it has been concluded that light porous aggregates or lighter concrete exhibit low thermal conductivity; similarly heavy dense concrete exhibit higher thermal conductivity. Apart from that, Chandra and Berntsson (2003) have mentioned that geometry of pores and their distribution in the concrete play a significant role. Foam concretes with or without sand were studied to analyze the influence of sand content. Results indicate that sand content increases the thermal conductivity slightly. This may be due to the presence of sand increasing the heat transfer processes better than concrete made without sand.

Thermal conductivity of concrete made of Leca and Liapour was observed to decrease when admixture of ground granulated blast furnace slag (GGBS) and silica fume (SF) were added in the concrete (Ramazan et al.,2003). The reason being that increasing the percentage of admixture in concrete actually reduces the dry unit weight and air content of concrete considerably which causes the conductivity of concrete to decrease.
Chapter 2: Thermal properties of Various Concrete

Fig. 2.10 Relationship between thermal conductivity of foam concrete (without sand) and temperature

Fig. 2.11 Relationship between thermal conductivity of foam concrete (with sand) and temperature
Chapter 2: Thermal properties of Various Concrete

Fig. 2.12 Relationship between thermal conductivity of Leca and Liapour concretes and temperature

Fig. 2.13 Relationship between thermal conductivity Pumice and normal weight concrete and temperature
Chapter 2: Thermal properties of Various Concrete

The result of partial replacement of admixtures in total binder is favorable especially in construction industry as it improves thermal isolation and decreases environmental contamination.

The effects of temperature on thermal conductivity measurements of various concretes were examined. It was observed that thermal conductivity increases a small amount with temperature in pumice concrete but decreases in normal weight concrete. Concretes were tested at 30°C, 40°C, 60°C, 80°C, 100°C and 120°C and results plotted between thermal conductivity and temperatures are shown in Fig.2.10 to Fig.2.13. Thermal performance of concrete primarily depends on aggregates internal microstructures and mineralogical components.

Lightweight concretes are made up of amorphous aggregate which exhibits low thermal conductivity at room temperature and slightly increases upon temperature rise. Whereas, normal weight concrete has highly crystalline aggregates which exhibits higher thermal conductivity at room temperature and decreases when temperature rises.

Porosity of foam concretes as well as concrete made of lightweight aggregates such as Leca, Liapour and Pumice have high solid matrix which is normally on the amorphous side. At elevated temperature, industrial manufactured LWA exhibits high chemical stability in which latent heat effects on de-hydration of cement paste have to be considered. Thermal conductivity of concretes was measured only up to 120°C due to the limitation of test equipment.
This chapter describes the development of an innovative thermal conductivity measuring system which is used for determining thermal conductivity. This apparatus is suitable to measure a wide range of thermal conductivity of materials such as concretes, mortars, metals, polymer products and ceramics products etc.

3.1 Shortcomings in existing methods

At present, several thermal conductivity methods are available to measure the thermal conductivity of building materials. The primary techniques used in thermal conductivity measurements are axial flow, guarded hot plate method and hot wire method. Broadly speaking, there are a number of possibilities to measure thermal conductivity of materials each of them suitable for a limited range of materials, depending on the thermal properties and the temperature testing range.

Above mentioned techniques has shortcoming to ensure one dimensional heat flow (Healy, 2001) during conductivity measurements. The error in all existing thermal conductivity system is mainly due to lateral heat flow to or from main heater i.e. edge heat losses. Even though, there is considerable development in improving
data logging system and temperature controllers, inaccuracies due to heat losses at the edge of specimen remain unresolved.

In the radial heat flow system either of cylinder or sphere, the heat loss at edge of specimen is zero theoretically and well designed experimenting can be done to ensure unidirectional heat flow during the thermal conductivity measurements. The proposed method specially prepares radial heat flow system (Hollow Sphere specimen) which allows unidirectional radial heat flow without edge heat losses. Based on steady state technique, an innovative Thermal Conductivity System (TCS) has been developed to overcome the shortcomings in the existing methods. This system can be used to calculate the thermal conductivity of any materials and it is relatively cheap, good accuracy and completely automated.

3.2 Basic principle of TCS

The basic principle of this thermal conductivity system is to generate a radial directional heat flux through the internal surface of a hollow sphere specimen (Hot-side Temperature) while the temperature at the outer surface is kept constant (Cold-side Temperature) during the entire test. The radial heat flux is the key parameter to control the internal surface temperature. Theoretically, there are no heat losses at edge of the specimen since heat flow takes place along radial direction i.e. unidirectional heat flow. The measurement technique belongs to the steady state method which involves heater as main heating source. Internal surface of the hollow sphere is subjected to hot temperature through heaters while the temperature at the outer surface is kept stable using temperature controlled chamber with an effective
Chapter 3: Development of Innovative Thermal Conductivity System

cooling system. The required radial heat flux is generated through a solid-state heater which is mounted inside the hollow sphere specimen.

In practical aspects, two semi hollow sphere specimens were cast separately and joined together to form the test specimen. O-ring concept was used in order to make good contact between the two semi-spheres and also to ensure that there is a perfect vacuum condition inside the specimen.

Specially designed vacuum adaptor was used to attain the vacuum inside the specimen. Before joining the two semi hollow specimens together, heater and Resistance Temperature Detectors (RTDs) were kept inside the specimen. Special software program was used to control the power required to heat up the hot side temperature automatically. Detailed experiment studies were done for standard reference material (Teflon) and concrete and will be discussed at the end of this chapter.

3.3 Thermal conductivity of hollow sphere shape

Heat transfer through insulation systems like building materials may involve several modes of heat transfer such as conduction through the solid materials, conduction or convection through the air in the void spaces and if the temperature is adequately high, radiation exchange between the solid matrix surfaces may also take place. In general, the thermal conductivity of the measuring system must include all modes of heat transfer process. That is the reason why thermal conductivity of insulation material is called to be effective thermal conductivity (Salmon, 2001).

For sphere shaped specimen, it often experiences temperature gradient in the radial direction and hence may be treated as one dimensional problem. Theoretically,
there is no heat loss in radial heat flow. Under the steady state conditions, thermal conductivity (k) of hollow sphere is equal to

\[
k = \frac{q}{4\pi} \left[ \frac{R_o - R_i}{R_i R_o \Delta T} \right]
\]

(3.1)

and the thermal resistance is

\[
R = \frac{1}{4\pi k} \left( \frac{1}{R_i} - \frac{1}{R_o} \right)
\]

(3.2)

Where \(T_i\) and \(T_o\) are the inner and outer surface temperature, and \(R_i\) and \(R_o\) the inner and outer radius of the hollow sphere, respectively.

### 3.4 Optimum radius for thermal conductivity test

It is necessary to have an optimum specimen size for the thermal conductivity tests. This can be selected on basis of weight of specimen, heat transfer rate and material to be tested. Because of uncertainties, the thermal conductivity is estimated as a function of thickness of specimen so that the optimum specimen thickness for thermal conductivity tests can be selected. For this study, four group of concrete specimen size were chosen to examine the optimum inner and outer radius considering the weight as factor such as, G1\((R_o=100\text{mm} & R_i=25\text{mm})\), G2\((R_o=125\text{mm} & R_i=50\text{mm})\), G3\((R_o=135\text{mm} & R_i=60\text{mm})\), G4\((R_o=175\text{mm} & R_i=60\text{mm})\). For practical reasons, heavy samples should be avoided as it can cause difficulty in experimental arrangements and handling.

The choice of sample thickness depends on the material to be tested and its thermal resistance, since the thermal resistance of material depends on thickness of material and thermal conductivity. In general, homogeneous materials may be tested for any suitable thickness ranging between 25 to 75 mm. There are other important
considerations for optimum sample thickness which are mentioned by ASTM standards and ISO 8302 specifications.

**Fig.3.1** Density of concrete material versus weight of sphere specimen for corresponding inner and outer radius

The graph has been generated theoretically between density of concrete and its corresponding weight of sphere specimen for various inner and outer radius of sphere. **Fig.3.1** gives an idea to choose the inner and outer sphere radius and desired thickness between 25 - 75 mm. For concrete materials, the spacing between aggregate and aggregate size have to be taken into account while determining the thickness of specimen. Concrete sample of 75mm thickness has been chosen for testing in order to ensure concreting homogeneity.
Chapter 3: Development of Innovative Thermal Conductivity System

3.5 Temperature Gradient Analysis

In practice, experience has shown that temperature gradient of 10°C to 30°C is most convenient for fine thermal conductivity tests. Higher temperature gradient requires more heat flux to achieve the required hot side temperature. On the other hand, low temperature gradient makes achieving and maintaining steady state condition more difficult. Usually the set temperature fluctuates in measurement by plus or minus few degree Celsius during the testing. So an optimum temperature gradient is necessary for thermal conductivity tests.

![Graph showing heat flux (power) required for different temperature gradient versus conductivity of sample](image)

**Fig. 3.2** Heat flux (power) required for different temperature gradient versus conductivity of sample

The graphs were generated for thermal gradient of 10°C, 14.23°C, 20°C, 28.46°C and 30°C with respect to thickness of hollow sphere (**Fig. 3.2**). Amongst them, any temperature gradients can be chosen for sample testing. It is good to choose the thermal gradient based on temperature profile to avoid the mean
temperature value in fraction. The graph was plotted between the heat fluxes required for testing versus corresponding temperature gradient (Fig.3.2). It has been concluded from figure that increasing temperature gradient needs higher heat flux for testing with regards to varying the thermal conductivity.

3.6 Prediction of mean sample temperature

The purpose of predicting the mean sample temperature is to represent the testing temperature of the specimen in rounded figure and to avoid mean sample temperature in fraction. The hollow sphere temperature profile can be calculated theoretically from the assumed temperature profile \( T \) as shown (Fig.3.3) with adequate boundary conditions.

\[
T = \frac{A}{r} + B \quad \text{(3.3)}
\]

The following two boundary conditions are sufficient to solve for the constants \( A \) and \( B \) in Eq. (3.3). \( T = T_i \) at \( r = R_i \) and \( T = T_o \) at \( r = R_o \).

Then, \( T \) becomes

\[
T = T_i - (T_i - T_o) \left( \frac{r}{R_i} \frac{1 - \frac{R_i}{R_o}}{1 - \frac{r}{R_i}} \right) \quad \text{(3.4)}
\]
Chapter 3: Development of Innovative Thermal Conductivity System

![Temperature profile diagram]

**Fig.3.3 Temperature profile over thickness of specimen for hollow sphere**

The thermal conductivity of sample is calculated for mean sample temperature so as to represent proper identity. $T_{\text{mean}}$ of the hollow sphere can be calculated from temperature profile in Eq. (3.4) over the specified boundaries

$$T_{\text{mean}} = \frac{\int_{R_i}^{R_o} T \, dr}{\int_{R_i}^{R_o} dr} = \frac{\int_{R_i}^{R_o} (T_i - \frac{R_o (T_i - T_o)}{R_o - R_i} \left(\frac{R_i}{r} - 1\right)) \, dr}{\int_{R_i}^{R_o} dr}$$  \hspace{1cm} (3.5)

Substituting inner and outer radius of the sphere $R_i = 50\text{mm}$ & $R_o = 125\text{mm}$ into Eq. (3.6) gives the relation between the $T_{\text{mean}}, T_i$, and $T_o$ as

$$T_{\text{mean}} = \frac{(R_o - R_i)T_i + \frac{R_o (T_i - T_o)}{R_o - R_i} \left(R_i \times \ln \frac{R_o}{R_i} - (R_o - R_i)\right)}{R_o - R_i}$$ \hspace{1cm} (3.6)

$$T_{\text{mean}} = \frac{26.35T_i + 48.65T_o}{75}$$  \hspace{1cm} (3.7)
Chapter 3: Development of Innovative Thermal Conductivity System

For example, to test concrete material of 75 mm thickness at $T_{mean}$ of 60°C, the inner surface temperature $T_i$ can be calculated to be equal to 78.46°C for cold side temperature of $T_o = 50°C$. In the experiment the cold side temperature, $T_o$ should be fixed and the hot side temperature can be achieved using the heater.

### 3.7 Heat Transfer Analysis on Hollow sphere

The temperature distribution and heat conduction of hollow sphere can be verified with finite element analysis. For heat conduction analysis, thermal conductivity of the material is an important parameter which controls the rate of heat flow in the medium. ABAQUS software programme was used to verify the theoretical temperature profile.

#### 3.7.1 Finite Element Analysis: ABAQUS

A pure heat transfer analysis was performed to determine the temperature distributions in hollow sphere and to study the effect of thermal contact materials if it is used to build close contact between the two semi-hollow spheres. Heat transfer analysis was performed using heat transfer elements and heat transfer procedure. Within a step, heat flux and boundary conditions were specified using the steady state conditions. Surface heat flux and boundary conditions were defined at heat transfer step. There is no fundamental physical meaning in choosing time scale in steady state heat transfer analysis; the time scale was assigned conveniently for output identification only.

Two kind of heat transfer analysis were done with consideration to with and without thermal contact materials. Practically, thermal contact materials (Example:
silicon rubber material, silicon solid paste etc) can be used to make close contact between the specimen. It ensures perfect heat transfer process between the two hollow semi-spheres and no heat leakages around the specimens joint. Theoretically calculated heat flux and corresponding thermal conductivity were used as input to the model and the results were compared with theoretically predicted temperature.

Fig.3.4 Mesh generated to hollow sphere Quadratic elements (DC3D20)

Free meshing technique was applied to hollow sphere without paste using quad-dominated element shape options (Fig.3.4). A DC3D20-20 node quadratic heat transfer brick element type was chosen for analysis. Total number of elements and nodes were 4968 and 21846, respectively. Analysis was carried out for varying thermal gradient of 20°C, 28.46°C and 30°C. The following tables (Table 3.1 to Table 3.4) show the temperature distribution based on theoretically derived results and ABAQUS analysis outputs. Results showed good agreement between theoretically derived results and output from ABAQUS. There was a minor variation observed for effect of thermal contact material.
### Table 3.1 Case: 1 Heat flux = 37.68 watts and \( k = 1.8 \, \text{W/m}^\circ\text{C} \)

<table>
<thead>
<tr>
<th>Temperature gradient</th>
<th>Thickness</th>
<th>Temperature distribution results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Theoretical</td>
</tr>
<tr>
<td>( ^\circ\text{C} )</td>
<td>( \text{mm} )</td>
<td>( ^\circ\text{C} )</td>
</tr>
<tr>
<td>20.00</td>
<td>75.00</td>
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<td>60.00</td>
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<td>70.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57.03</td>
</tr>
</tbody>
</table>

### Table 3.2 Case: 2 Heat flux = 56.52 watts and \( k = 1.8 \, \text{W/m}^\circ\text{C} \)

<table>
<thead>
<tr>
<th>Temperature gradient</th>
<th>Thickness</th>
<th>Temperature distribution results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Theoretical</td>
</tr>
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<td>( ^\circ\text{C} )</td>
<td>( \text{mm} )</td>
<td>( ^\circ\text{C} )</td>
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</tr>
<tr>
<td></td>
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<td>60.54</td>
</tr>
</tbody>
</table>

### Table 3.3 Case: 3 Heat flux = 75.36 watts and \( k = 1.8 \, \text{W/m}^\circ\text{C} \)

<table>
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<th>Temperature gradient</th>
<th>Thickness</th>
<th>Temperature distribution results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Theoretical</td>
</tr>
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<td>( ^\circ\text{C} )</td>
<td>( \text{mm} )</td>
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<td>90.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>64.05</td>
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</table>
Chapter 3: Development of Innovative Thermal Conductivity System

Table 3.4 Case: 4 Heat flux = 53.62 watts and k = 1.8 W/m°C

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<th>Thickness</th>
<th>Temperature distribution results</th>
</tr>
</thead>
<tbody>
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<td></td>
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<td>mm</td>
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<td>0.00</td>
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</tr>
<tr>
<td>$T_{mean}$</td>
<td>60.00</td>
<td>60.02</td>
</tr>
</tbody>
</table>

Fig.3.5 Contour plot of temperature distribution for semi hollow sphere (ABAQUS output)

Temperature distribution of hollow sphere with and without thermal contact material was studied. For this study, cold side temperature ($T_o$) was taken as 50°C and thermal conductivity was kept constant at 1.8 W/m°C. Four different cases were considered for heat transfer analysis on the basis of temperature gradient. Fig.3.5
Chapter 3: Development of Innovative Thermal Conductivity System

represents the contour plot of temperature distribution of sphere over thickness. Theoretically calculated temperature agreed well with the numerically predicted temperature.

3.7.2 Hollow sphere with thermal contact material

Analysis was similar to previous one except that thermal contact material was present between the two semi hollow spheres. Heat transfer analysis was also carried out same as for previous analysis. The previous analysis is the ideal situation without joints but practically two semi hollow spheres is used jointed together. In order to avoid the heat transfer through the gap between the two semi hollow spheres, thermal contact material was used. Although practically, measurements would not be carried out near the specimen joint but it is necessary to control heat flow. The purpose of this analysis is to know the effects of various thicknesses of thermal contact material being used and how it affects the accuracy of measured conductivity. The thermal contact material of 1.0 mm, 2.0 mm and 3.0 mm were considered in the heat transfer analysis.

Results showed that thermal contact materials affected heat transfer process considerably (Fig.3.6). Due to that it may require additional heat flux to heat to the required temperature which will maximize or minimize the conductivity coefficient marginally if thickness of joint is less than 3mm. The effect of thermal contact material of 1.0 mm, 2.0 mm and 3.0 mm thickness was verified for varying thermal conductivity of thermal contact material. The percentage error increases with increasing thickness of thermal contact material. From Fig 3.6, it is shown that the percentage error also depends on thermal conductivity of testing sample.
Fig. 3.6 Error in hot side temperature for varying thermal contact material thickness

Fig. 3.6 shows the source of potential error due to thermal contact material which certainly alters the temperature profile, and thus produces the error in conductivity measurements. This analysis gives an idea to select suitable thermal contact material for thermal conductivity measurement. But it is important to ensure that these thermal contact materials should not affect the heat transfer processes significantly. Thermal contact material of 2mm size would be suitable for testing and the measured error would be within the testing limit. From this analysis, it is recommended to use thermal contact material with thermal conductivity equal to or higher than that of the testing materials.

UKAS report indicated that the uncertainties were mainly associated with the imperfect surfaces in thermal conductivity measurements. Salmon (2001) has provided the estimated overall uncertainty in thermal conductivity measurements as
Chapter 3: Development of Innovative Thermal Conductivity System

a function of specimen thickness and proved that highest accuracy of ± 1.3% could be obtained for specimen thickness of 50 mm to 75 mm. Sample thicknesses less than 50mm and greater than 75 mm increase the overall uncertainties due to error in thickness measurement and heat losses respectively. The uncertainty of about ±5% has been pointed out as a design criterion for whole range of temperature and thermal resistance measurements (Salmon, 2001). The proposed thermal conductivity measuring system has overall uncertainty of about ±3.907%.

3.8 Experimental Studies on TCS and discussion on test results

The principle behind the measurement may be simple, but construction of the apparatus requires careful attention to ensure that one dimensional heat flow is achieved in the specimen. Temperature measurements closely approximate the true $\Delta T$ across the specimen section. The instrument consists of temperature controller to achieve hot side temperature and cold side temperature. The apparatus can test one sample at a time. A temperature controlled chamber is used to control the cold side temperature whereas hot side temperature is controlled through heater. The fixed power input to the heater is provided by regulated DC supply. Heater is allowed to raise the hot side temperature until thermal equilibrium is reached. The required testing time is dependent on the mass of the specimens and operating temperatures.

Fig.3.7 shows the flowchart of the working principle of developed thermal conductivity measuring system. This test can be performed on any samples provided that samples can be molded in semi hollow sphere shape.
Chapter 3: Development of Innovative Thermal Conductivity System

START

Switch on System and Set the Value of STi, STo, SD, T

IF Check Inner Heater to reach STi

Yes

IF Current Temp (CTi) < Set Temp (STi)

No

Inside Heater On Mode

IF Current Temp (CTo) < Set Temp (STo)

No

Outside Heater On Mode

IF CTi is between STi-0.25 and STi+0.25

Yes

Measure Power(q) = Voltage x Current (E x I)

IF CTo is between STo-0.25 and STo+0.25

Yes

Timer Starts for Checking Steady State (SS) Conditions

Check SS if (STi-0.25<CTi<STi+0.25) and (STo-0.25<CTo<STo+0.25)

Yes

A

B
3.8.1 Verification on Standard Reference material (PTFE)

Laboratory tests were performed on standard reference material TEFLO (PTFE). Thermal conductivity tests were conducted at mean sample temperature at 40°C. During the trial tests, it was observed that there was an uneven thermal gradient between top and bottom of inside sphere specimen. According to theory, if there is no air inside the specimen heat transfer is through radiation process alone and the measured mean temperature would not be different. Instead the measured temperatures were different values within the specimen because there is air inside in which some of the energy from the heater is transferred by radiation and some by convection.
Chapter 3: Development of Innovative Thermal Conductivity System

In order to ensure heat flow only through radiation process, there was a need to create vacuum inside the specimen. Fig.3.8 shows the design of vacuum adaptor to rectify the above mentioned problem. Fig.3.9 and 3.10 show the PTFE testing specimen and vacuum adaptor with vacuum gauge respectively.

![Vacuum Adaptor design for thermal conductivity tests](image)

**Vacuum Adaptor**

**Fig.3.8 Vacuum Adaptor design for thermal conductivity tests**
Chapter 3: Development of Innovative Thermal Conductivity System

Fig. 3.9 Thermal conductivity test on PTFE material

Fig. 3.10 Vacuum Adaptor with vacuum gauge
Chapter 3: Development of Innovative Thermal Conductivity System

The vacuum adaptor maintained vacuum inside the specimen and thus only radiation took place in the vacuum body during the heat flow and there was no conduction or convection process in the vacuum.

In between the specimen joints, silicon based glue was used to create air tightness. In order to ensure vacuum tightness, O-ring was used between the specimens. Thermal conductivity test was conducted at mean sample temperature of 40\(^{\circ}\)C. Two conditions of testing were performed at mean sample temperature of 40\(^{\circ}\)C.

**Type I:** Temperature sensor RTD was mounted on only top of Teflon specimen

**Type II:** Temperature sensor RTDs were mounted on four different locations in the Teflon specimen.

### 3.8.2 Experimental Procedure

The developed thermal conductivity system is user-friendly to perform the thermal conductivity tests. Once the system is switched on, the hot and cold temperatures values are then set in the display screen. The system also allows user to set steady state period so that once the system has attained the desired set temperatures, the system will be at steady state for that period. The working principle of TCS is according to the flow chart shown.

Specialized software was used to automate the adjustment of the heater energy so that convergence takes place automatically. The computer coding was written to control the temperature and power required. This was achieved by a optimal converging procedure. Due to the automation procedure, this system does not require approximate power input before starting of test and the testing time depends on the material to be used. Normally it takes about 3-4 hours to reach
Chapter 3: Development of Innovative Thermal Conductivity System

steady state condition. Once the system attains the desired steady state time, then mean temperature and effective thermal conductivity of material can be calculated from Eq. (3.6) and Eq. (3.1). The observed test results of two cases were plotted (Fig. 3.11 Fig.3.12 Fig.3.13, Fig. 3.14 and Fig.3.15).

Table 3.5 shows the measured thermal conductivity and the relative error. From the results the measured k-values had error of less than 6.3 % and the measured conductivity values were greater than actual value. Between Type I and Type II tests, Type II test was marginally affected by convection heat processes as discussed early. That was the reason the error obtained from Type II was greater than Type I. The predicted values were also compared with system allowable uncertainty as shown in Table 3.5.

Table 3.5 PTFE Thermal conductivity test result summary

<table>
<thead>
<tr>
<th>Type</th>
<th>k- Value (measured)</th>
<th>k - value(Allowable 3.907 % Uncertainty)</th>
<th>Error based on actual value</th>
</tr>
</thead>
<tbody>
<tr>
<td>W/mK</td>
<td></td>
<td>(k measured - 3.907%)</td>
<td>(k measured +3.907%)</td>
</tr>
<tr>
<td>Test Type I</td>
<td>0.25961</td>
<td>0.2495</td>
<td>0.2698</td>
</tr>
<tr>
<td>Test Type II</td>
<td>0.26696</td>
<td>0.2565</td>
<td>0.2774</td>
</tr>
<tr>
<td>Actual k-Value</td>
<td>0.25104</td>
<td>Nil</td>
<td>Nil</td>
</tr>
</tbody>
</table>

Note : Overall uncertainty associated with thermal conductivity measurement k is 3.907% (Predicted by method error propagation) 
Error = (Average k- value - Actual value)x100/Actual Value
Fig. 3.11 Heater temperatures of Test Type I and Test Type II

Fig. 3.12 Hot side temperatures of Test Type I and Test Type II
Chapter 3: Development of Innovative Thermal Conductivity System

Fig. 3.13 Cold side temperatures of Test Type I and Test Type II

Fig. 3.14 Mean temperatures of Test Type I and Test Type II
Fig. 3.15 Power required for Test Type I and Test Type II

3.8.3 Thermal conductivity test on concrete

The developed TCS has also been calibrated against Lightweight Concrete (LWC) and was found to be working satisfactorily. The hollow sphere concrete specimen of 250 mm outer diameter and 100 mm inner diameter (thickness 75mm) was used for testing. Special mold was required to cast the concrete specimen to required shape. Fig.3.16 shows the special mold base and cover design. Before testing the specimen, the specimen surfaces were smoothened to achieve even surfaces in order to maintain the good contact.
Chapter 3: Development of Innovative Thermal Conductivity System

Fig. 3.16 Special Mold design of base and cover
Chapter 3: Development of Innovative Thermal Conductivity System

The vacuum adaptor was modified for concrete testing as shown (Fig. 3.17). Double O-Ring concept was used in between the sample to ensure high vacuum level. Compared to vacuum level obtained for previous test using SRM-Teflon (Stand reference material), LWC was lower. This is due to uneven LWC surfaces. The measured values have been checked against exiting results and detailed test results are tabulated in Table 3.6. Fig 3.18 and Fig. 19 show the mean temperature and power required for LWC material.

Fig. 3.17 TCS test on LWC with modified vacuum adaptor
Table 3.6 LWC thermal conductivity results

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Average k-Value (measured)</th>
<th>k - Value (Allowable 3.907% Uncertainty)</th>
<th>Error based on actual value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Type I</td>
<td>1.059</td>
<td>1.0176, 1.1004</td>
<td>%</td>
</tr>
<tr>
<td>Actual k-Value (From Literature)</td>
<td>0.80 - 1.20 (Range for LWC)</td>
<td>Nil, Nil, Nil</td>
<td>Nil</td>
</tr>
</tbody>
</table>

Note: Overall uncertainty associated with thermal conductivity measurement k is 3.907% (Predicted by method Error propagation)
Error = (Average k-value - Actual value)x100/Actual Value
3.9 Advantages of invented thermal conductivity system

The developed thermal conductivity system has the following advantages over other existing systems

- Improved temperature control systems (Resistance Temperature detectors, RTDs) are used as temperature measuring sensors.
- More accurate instrumentation in which whole thermal conductivity measuring process has been controlled automatically.
- Importantly, unidirectional heat flux through the specimen is ensured in this thermal conductivity system by adopting hollow sphere specimen. Also, theoretically there is no lateral heat flow and thereby there are no edge heat losses in this system.
- Complete automation and relatively cheap with good accuracy.
CHAPTER 4

DETERMINATION OF EARLY AGE THERMAL DIFFUSIVITY
AN ANALYTICAL APPROACH

4.1 Introduction

The evolution of concrete properties at early age is very significant. When fresh concrete has been placed, it undergoes phase change from liquid to solid, and the hydration reaction of the cement and water generates heat, sometimes referred to as heat of hydration. Under these circumstances, the determination of thermal conductivity and specific heat capacity for concrete at early age may be complicated. On the other hand, diffusivity of concrete, evolving with time, can be determined notwithstanding the generation of heat due to hydration reaction and can be used in place of thermal conductivity and specific heat capacity, for simulating the temperature development in concrete.

A new and simple method for determining diffusivity of concrete at early age that takes into account the heat of hydration is proposed. The experimental method is devised based on an analytical solution obtained with heat flow equation as the starting point. The analytical solution has been verified by finite difference and finite element analyses. Accordingly, a laboratory experiment was carried out and the diffusivity of the concrete at early age was determined.
4.2 Importance of Thermal diffusivity at early age

Three main factors to be considered in thermal stress analysis of concrete at early age are temperature development in the concrete being cast, mechanical properties of the concrete at early age and degree of restraint imposed on the concrete. To compute the temperature development within a concrete at early age accurately, it is prudent that the accurately determined heat of hydration, thermal conductivity and specific heat capacity of the concrete are used. However, thermal conductivity and specific heat capacity of concrete evolve with time at early age. Furthermore, concrete at early age changes state from liquid to solid and generates heat of hydration making the accurate determination of thermal conductivity and specific heat capacity very intricate.

Many researchers have proposed methods to determine the thermal conductivity and specific heat capacity of concrete at early age as discussed in literature review, most of which involve inserting a probe into concrete and measuring the heat transfer, but more often do not take into consideration the generation of heat due to hydration reactions. In some of these methods, the thermal diffusivity is determined by evaluating several unknown quantities by curve fitting or iteration process, giving rise to uncertainties. Nevertheless, not taking into consideration the heat generation due to hydration reactions may still be considered as the major setback in all these methods.

The perennial difficulty to accurately determine the thermal conductivity and specific heat capacity of concrete at early age arises due to the change in state of the concrete from liquid to solid and generation of heat during hydration. The change in state of concrete from liquid to solid causes variability in moisture content in
addition to the difficulty in sample preparation, both of which are undesirable test boundary conditions. In addition, the generation of heat in the concrete during hydration complicates the accurate measurement of heat flow which is an essential parameter to determine the concrete thermal properties.

To circumvent this problem, the diffusivity of concrete may be used instead, to determine the temperature development in the concrete. It is defined as ratio of thermal conductivity and volumetric heat capacity. For this, the heat flow equation may be rewritten as Eq. (4.1)

\[
\frac{\partial T}{\partial t} = \alpha \left( \frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} + \frac{\partial^2 T}{\partial z^2} \right) + \frac{\partial T_h}{\partial t} \tag{4.1}
\]

Where diffusivity of concrete, \( \alpha \) incorporates the thermal conductivity \( k \), the specific heat capacity \( c \), and the density \( \rho \) of the concrete.

4.3 Basic principle of thermal diffusivity method

A new method is proposed in this chapter to determine the diffusivity of concrete at early age directly. The diffusivity can be determined accurately with reasonable ease notwithstanding the change in state of the concrete from liquid to solid and the generation of heat during hydration. On the hindsight, the temperature distribution within a concrete is computed using diffusivity of concrete and hence appears to be the more appropriate parameter to determine.

In the new test method, fresh concrete sample in a cylindrical container of diameter 150 mm and height 300 mm is subjected to a cyclic temperature while the temperature distribution profile within the specimen is maintained the same. Under this boundary condition, the diffusivity of the concrete can be determined from the
change in temperature at the core of the concrete specimen over a small time interval. This then allows the diffusivity of the concrete to be computed at small time intervals and hence establish the variation of diffusivity of the concrete with time.

To compute the diffusivity, an analytical solution for the temperature distribution within a solid cylinder of finite length and prescribed boundary condition was first derived. The analytical solution was verified against two numerical models, one using finite element method and the other using finite difference method. The input for the analytical solution includes the rate of temperature rise within the concrete due to heat of hydration, in addition to the boundary conditions, that is, the core and surface temperatures, and the diameter of solid cylinder. The rate of hydration is usually greater at early age and at higher curing temperature and gradually slows down to an insignificant level during which time the hardened concrete is relatively inert and stable. To compute the heat of hydration at the appropriate concrete age and curing temperature, the maturity concept developed by Freiesleben Hansen (1977) and Pedersen (2003) was used.

The experimental procedures and the result of a laboratory test to obtain the diffusivity of concrete varying with time are presented in this chapter.

### 4.4 An Analytical approach

The differential heat conduction equation for a homogeneous, isotropic solid cylinder of radius $R$, length $L$ and where the axis of the cylinder coincides with the coordinate $z$, may be expressed as (Carslaw et al., 1973) and (Luikov, 1968)
where \( r, z \) and \( \phi \) are coordinates of a cylindrical coordinate system admissible within the limits \( 0 \leq r \leq R, 0 \leq z \leq L \) and \( 0 \leq \phi \leq 2\pi \) and \( Q_h(t) \) the rate of heat generation within a body that may be a function of time and position. For a cylinder whose length is considerably greater than its diameter, the heat flow along \( z \) axis may be assumed negligible. In addition, if the heat transfer between the cylinder surface and the surroundings occurs uniformly over the whole surface, heat flow along \( \phi \) axis may also be assumed to be negligible. Hence, substituting

\[
\frac{\partial^2 T}{\partial \phi^2} = 0
\]

(4.3)

and

\[
\frac{\partial^2 T}{\partial z^2} = 0
\]

(4.4)

into Eq. (4.2) and rearranging some terms result in

\[
\frac{\partial T}{\partial t} = \alpha \left( \frac{1}{r} \frac{\partial T}{\partial r} + \frac{\partial^2 T}{\partial r^2} \right) + \frac{Q_h}{c\rho}.
\]

(4.5)

Now applying Laplace transformation to the differential equation, the ordinary differential Bessel equation for the transform \( T(r, s) \) is obtained as

\[
rT''(r, s) + T'(r, s) - r \left( \frac{s}{\alpha} \right) \left[ T(r, s) - \left( \frac{T_i}{s} + \frac{Q_h}{s^2 c\rho} \right) \right] = 0
\]

(4.6)

where \( T_i \) is the initial temperature in the concrete. The general solution of Eq. (4.6) may be written as

\[
T(r, s) - \left( \frac{T_i}{s} + \frac{Q_h}{s^2 c\rho} \right) = AI_0(z) + BK_0(z)
\]

(4.7)
where,

\[
I_0(z) = J_0(iz) = 1 + \frac{z^2}{2^2} + \frac{z^4}{2^2 \cdot 4^2} + \frac{z^6}{2^2 \cdot 4^2 \cdot 6^2} + \ldots ,
\]

(4.8)

\[
K_0(z) = -\left[\log\left(\frac{1}{2}z\right) + C\right]I_0(z) + \left(1 + \frac{1}{2}\right)\left(\frac{1}{2}z\right)^1 + \left(1 + \frac{1}{2} + \frac{1}{3}\right)\left(\frac{1}{2}z\right)^2 + \ldots ,
\]

(4.9)

\[
z = \left(\frac{s}{\alpha}\right)^{1/2} r ,
\]

(4.10)

\[A \quad \text{and} \quad B \quad \text{the constants independent of} \quad r \quad \text{which can be determined from the boundary conditions and} \quad C = 0.5772 \quad \text{the Euler constant. Now, considering only the cases wherein the temperature along} \quad z \quad \text{axis cannot be infinity, constant} \quad B \quad \text{would have to be equal to zero since when} \quad r \rightarrow 0 , \quad K_0 \left\{ \left(\frac{s}{\alpha}\right)^{1/2} r \right\} \rightarrow -\infty . \quad \text{Hence, Eq. (4.7) reduces to},
\]

\[
T(r, s) - \left(\frac{T_1 + \frac{Q_0}{s^2 cp}}{s}\right) = A I_0 \{ z \} .
\]

(4.11)

For the case of the initial boundary condition being at steady state, the temperature distribution in the mid section of the cylinder may be given by [9]

\[
T_i = \left(1 - \frac{r^2}{R^2}\right)(T_1 - T_2) + T_2
\]

(4.12)

where \(T_1\) and \(T_2\) are the temperatures along the cylinder axis, (i.e. \(r = 0\)) and at the surface (i.e. \(r = R\)), respectively. Now, if the surrounding temperature is increased by \(\Delta T\) and kept constant, then

\[
T(R, s) = \frac{T_2 + \Delta T}{s},
\]

(4.13)

and applying this boundary condition in Eq. (4.11), the constant \(A\) can be found as
Chapter 4: Determination of Early age thermal diffusivity - An Analytical Approach

\[
A = \frac{\left( \frac{T_2 + \Delta T}{s} - \frac{T_1}{s} \right) - \frac{Q_b}{s^2 c \rho}}{I_0 \left( \frac{s}{\alpha} \right)^{1/2} R} \tag{4.14}
\]

Substituting \( A \) into Eq. (4.11) and rearranging, the solution for the transform will be

\[
T(r,s) = \left( \frac{T_1}{s} + \frac{Q_b}{s^2 c \rho} \right) = \left( T_2 + \Delta T - T_1 \right) \frac{\varphi(s)}{s \psi(s)} - \frac{Q_b}{c \rho} \frac{\varphi(s)}{s^2 \psi(s)} \tag{4.15}
\]

where

\[
\varphi(s) = I_0 \left( \frac{s}{\alpha} \right)^{1/2} r \tag{4.16}
\]

and

\[
\psi(s) = I_0 \left( \frac{s}{\alpha} \right)^{1/2} R \tag{4.17}
\]

The solution of Eq. (4.15) obtained by inverse Laplace transformation,

\[
T(r,t) = \left( T_1 + \frac{Q_b}{c \rho} \right) + L^{-1} \left[ \left( T_2 + \Delta T - T_1 \right) \frac{\varphi(s)}{s \psi(s)} \right] - L^{-1} \left[ \frac{Q_b}{c \rho} \frac{\varphi(s)}{s^2 \psi(s)} \right] \tag{4.18}
\]

would result as

\[
T(r,t) - (T_2 + \Delta T) - \frac{Q_b R^2}{4 \alpha c \rho} \left( 1 - \frac{r^2}{R^2} \right) = \sum_{n=1}^{\infty} \left( T_i - (T_2 + \Delta T) \right) A_n J_0 \left( \mu_n \frac{r}{R} \right) \exp \left[ -\mu_n^2 \left( \frac{\alpha t}{R^2} \right) \right] \tag{4.19}
\]

where \( I_0(z) = J_0(iz) \) and \( \mu_n \) is the \( n \)th root of \( J_0(iz) = 0 \). Choosing constants \( A_n \) and \( B_n \) such that the initial conditions are satisfied and limiting to first term after substituting Eq. (4.12) for initial conditions, Eq. (4.19) may be expressed as
Chapter 4: Determination of Early age thermal diffusivity - An Analytical Approach

\[ T(r, t) = T_2 + \Delta T + \frac{Q_h R^2}{4 \alpha c \rho} \left(1 - \frac{r^2}{R^2}\right) + \left[1 - \frac{r^2}{R^2}(T_1 - T_2) - \Delta T - \frac{Q_h R^2}{4 \alpha c \rho} \right] \exp\left[-\frac{5.783 \alpha t}{R^2}\right]. \]  

(4.20)

Now for temperature at the core of the solid cylinder, that is at \( r = 0 \),

\[ T(0, t) = T_1 + \left( T_2 - T_1 \right) + \Delta T + \frac{Q_h R^2}{4 \alpha c \rho} \left[1 - \exp\left(-\frac{5.783 \alpha t}{R^2}\right)\right]. \]

(4.21)

Next, the temperature rise of the concrete due to hydration reaction can be obtained from

\[ dT_h = \int \frac{Q_h}{c \rho} \, dt \]  

(4.22)

and in the case of piecewise linear wherein \( Q_h, c \) and \( \rho \) remains constant for a small time interval \( \Delta t \), the temperature rise would then be

\[ \Delta T_h = \frac{Q_h}{c \rho} \Delta t. \]  

(4.23)

Substituting Eq. (4.23) into (4.21), the temperature at the core is now given by

\[ T(0, t) = T_1 + \left( T_2 - T_1 \right) + \Delta T + \left(\frac{\Delta T_h}{\Delta t}\right) \left(\frac{R^2}{4 \alpha}\right) \left[1 - \exp\left(-\frac{5.783 \alpha t}{R^2}\right)\right]. \]  

(4.24)

The plot of diffusivity against time, using Eq. (4.24), for various \( \Delta T_h/\Delta t \) gives a series of straight lines as shown in Fig.4.1.
Chapter 4: Determination of Early age thermal diffusivity - An Analytical Approach

Fig. 4.1 Diffusivity as a function of reciprocal of time for various \((\Delta T_h/\Delta t)\)

A test specimen of diameter 150mm, temperature differential \((T_2 - T_1)\) of 5°C and surrounding temperature increment of \(\Delta T = 0.1^\circ C\) had been assumed for the plots of Fig. 4.1.

Inspecting Eq. (4.24) in the light of Fig. 4.1, it can be deduced that the straight lines may be represented by the equation

\[
\alpha = A \left(\frac{1}{t}\right) + B \tag{4.25}
\]

where, when \(1/t = 0\)

\[
\alpha = B = \frac{R^2}{-4((T_2 - T(0,t)) + \Delta T)} \left(\frac{\Delta T_h}{\Delta t}\right) \tag{4.26}
\]

and when \((\Delta T_h/\Delta t) = 0\), \(B = 0\) and
Chapter 4: Determination of Early age thermal diffusivity - An Analytical Approach

\[ \alpha t = A = \frac{\ln \left( \frac{(T_2 - T_1) + \Delta T}{(T_2 - T(0,t)) + \Delta T} \right)}{5.783} R^2. \]  

(4.27)

Substituting Eqs (4.26) and (4.27) into (4.25), diffusivity can be obtained as

\[ \alpha = R^2 \left[ \frac{\ln \left( \frac{(T_2 - T_1) + \Delta T}{(T_2 - T(0,t)) + \Delta T} \right)}{5.783t} - \frac{\left( \frac{\Delta T_h}{\Delta t} \right)}{4((T_2 - T(0,t)) + \Delta T)} \right]. \]  

(4.28)

4.5 Verification of the analytical solution

Eq. (4.28) which forms the basis for measuring diffusivity of concrete at early age was verified against the solutions from finite difference and finite element analyses.

4.5.1 Finite difference method

The finite difference approximation of Eq. (4.5) can be obtained by substituting the respective differentials,

\[ \frac{\partial T}{\partial t} = \frac{T_{i(i+1)} - T_i}{\Delta t} \]  

(4.29)

\[ \frac{\partial T}{\partial r} = \frac{T_{(i+1)} - T_{(i-1)}}{2\Delta r} \]  

(4.30)

and

\[ \frac{\partial^2 T}{\partial t^2} = \frac{T_{(i-1)} - 2T_i + T_{(i+1)}}{\Delta r^2} \]  

(4.31)

into it as

\[ \frac{T_{i(i+1)} - T_i}{\Delta t} = \alpha \left[ \left( \frac{T_{(i-1)} - 2T_i + T_{(i+1)}}{\Delta r^2} \right) + \frac{1}{r} \left( \frac{T_{(i+1)} - T_{(i-1)}}{2\Delta r} \right) \right] + \frac{Q_h}{cp} \]  

(4.32)
Chapter 4: Determination of Early age thermal diffusivity - An Analytical Approach

which can be further simplified to

\[ T_{i(r+1)} = \left( 1 - \frac{2\alpha \Delta t}{\Delta r^2} \right) T_i + \alpha \Delta t \left( \frac{1}{\Delta r^2} - \frac{1}{2r \Delta r} \right) T_{i(\rightarrow)} + \alpha \Delta t \left( \frac{1}{\Delta r^2} + \frac{1}{2r \Delta r} \right) T_{i(\leftarrow)} + \frac{Q_i \Delta t}{c \rho} \]  

(4.33)

Now setting

\[ \frac{2\alpha \Delta t}{\Delta r^2} = 1, \]  

(4.34)

diffusivity can be obtained as

\[ \alpha = \frac{\Delta r^2}{2 \Delta t}. \]  

(4.35)

Finally, substituting Eq.s (4.23) and (4.35) into (4.33),

\[ T_{i(r+1)} = \left( \frac{T_{i(\rightarrow)} + T_{i(\leftarrow)}}{2} \right) + \left[ (T_{i(\rightarrow)} - T_{i(\leftarrow)}) \times \left( \frac{\Delta r}{4r} \right) \right] + \left( \frac{\Delta T_i}{\Delta t} \right) \Delta t'. \]  

(4.36)

Using 100 nodes which gives \( \Delta r = 1.5 \text{mm} \) for \( R = 75 \text{mm} \), and for \( \Delta T/h/\Delta t = 1 \), \( \Delta t \) was obtained from

\[ \Delta t = \frac{\Delta t'}{n}. \]  

(4.37)

where, \( \Delta t' \) is the time taken for temperature to rise or fall by \( \Delta T \) and \( n \) the number of iterations in the finite difference analysis. The solution from the finite difference method was compared against Eq. (28) for \( (T_2 - T_1 = 5^\circ C & \Delta T = 0.1^\circ C \) and \( (T_2 - T_1 = -5^\circ C & \Delta T = -0.1^\circ C \) as shown in Fig.4.2 and Fig. 4.3, respectively, for \( \Delta T/h/\Delta t = 1 \).
Chapter 4: Determination of Early age thermal diffusivity - An Analytical Approach

Fig. 4.2 Comparison of Analytical solution with Finite Difference and Finite Element Method for $T_2 - T_1 = 5^\circ C$, $\Delta T_h/\Delta t = 1$ and $\Delta T = 0.1^\circ C$

Fig. 4.3 Comparison of Analytical solution with Finite Difference and Finite Element Method for $T_2 - T_1 = -5^\circ C$, $\Delta T_h/\Delta t = 1$ and $\Delta T = -0.1^\circ C$
4.5.2 Finite element method - ABAQUS

In the finite element analysis using ABAQUS, the cylinder was modeled with 900 numbers of 20-noded quadratic heat transfer brick elements containing a total of 4375 nodes as shown in Fig.4.4. A boundary condition of $T_2 - T_1 = \pm 5^\circ C$ [$\pm 9^\circ F$] and a body heat flux corresponding to $\Delta T/h/\Delta t = 1$ was applied using user-defined subroutines in the finite element analysis and the time taken for the core temperature to rise/fall by 0.1°C was determined for various diffusivity and plotted as shown in Fig.4.2 and Fig.4.3, respectively.

Fig.4.4 Finite element mesh of solid cylinder

Fig.4.2 and Fig.4.3 verify the validity of the analytical formulation of Eq. (4.28) which can now be used to determine the diffusivity of concrete at early age, experimentally.
4.6 Experimental procedures to measure diffusivity at early age

A new and simple method has been devised to measure the diffusivity of concrete at early age. This method allows the diffusivity of concrete at early age to be measured continuously as it evolves with time. Besides catering to the changes in the state of the concrete from liquid to solid at early age, the new method also takes into account the heat generated due to hydration reaction. The change in state of the concrete and the generation of heat due to hydration reaction had been an impediment to measuring the thermal properties of concrete at early age.

The principle of the proposed method is that diffusivity of concrete can be computed from the rate of temperature change at the core of a test specimen when the surrounding temperature is increased or decreased. Eq. (4.28) is used for the computation wherein the temperature profile within the concrete would have to be at steady-state before each increment of $\Delta T$ of the surrounding temperature so that the initial condition assumed in deriving the equation is satisfied.

In the laboratory procedure to measure diffusivity, fresh concrete is placed in a steel container of diameter 150mm and height, at least 300mm. A type-T thermocouple is embedded in the concrete specimen to measure the temperature at the core during the test. A strain gage can also be embedded in the concrete specimen to concurrently determine the coefficient of thermal expansion from the same experiment (Tamilselvan et al., 2005). It is necessary that the height of the container is at least twice its diameter to ensure that the assumption of 2-D radial heat flow at the mid-height of the concrete specimen is valid.

The steel container, together with the fresh concrete, is then placed in an oven. The oven is equipped with a Shimaden SR25 controller which can be
programmed to set the oven temperature based on the temperature feedback from the thermocouple placed in the concrete, as illustrated in Fig.4.5. The controller is programmed such that it initially set the temperature of the oven to 10°C higher than the concrete temperature. When the concrete temperature has increased to 5°C lower than the oven temperature, the controller then maintains this temperature differential by accordingly increasing the oven temperature at an increment of $\Delta t = 0.1^\circ C$ as the concrete’s temperature increases.

![Flowchart](image)

**Fig.4.5 Experimental set-up for the determination of diffusivity of concrete at early age.**

When the concrete temperature reaches 70°C, the controller then sets the temperature of the oven to 5°C lower than the concrete temperature and maintains this temperature differential by accordingly decreasing the oven temperature at $\Delta T = 0.1^\circ C$, as the concrete temperature decreases. When the concrete temperature is
lowered to 30°C, the controller then again sets the temperature of the oven to 5°C higher than the concrete temperature and maintains this temperature differential by accordingly increasing the oven temperature at an increment of $\Delta t = 0.1^\circ C$ as the concrete’s temperature increases.

Due to the heat of hydration, the time for each cycle whereby concrete temperature increases from 30°C to 70°C and decreases back to 30°C, would vary. This cycle is repeated until the effect of heat of hydration of the concrete on the cycle time is negligible, which in this case was after 60 hours as illustrated in Fig.4.6. The cyclic period of 60 hours has been chosen based on the adiabatic temperature rise of particular concrete mix used. Normally, the time period would vary depending on type of concrete use.

![Fig.4.6 Variation of concrete core and oven temperature with time.](image)

The change in diffusivity of concrete, thereafter, would be negligible. In this
test, the concrete test specimen was inadvertently kept at room temperature of 30°C for 10 hours before placing in the oven and starting the test. This has caused the inability to compute the diffusivity for the first 10 hours but, on the other hand, demonstrates that diffusivity can still be computed nevertheless. For a proper test, the concrete test specimen should have been placed in the oven immediately after batching.

Concurrent to this test, the adiabatic temperature rise of concrete as shown in Fig. 4.7 was also determined for the same concrete using an adiabatic temperature rise chamber developed in National University of Singapore (Ng et.al. 2005). The chamber consists of a double-wall hot-guard system where an auxiliary heater essentially prevents heat loss at the boundary of the concrete specimen being tested.

![Fig.4.7 Adiabatic temperature rise of concrete](image_url)

To determine diffusivity using Eq. (4.28), the change in temperature of
Determination of Early age thermal diffusivity - An Analytical Approach

Concrete with time due to hydration reaction, \( \frac{\Delta T_h}{\Delta t} \) is required. Although \( \frac{\Delta T_h}{\Delta t} \) depends on the curing temperature, it can be obtained from the adiabatic temperature rise of the concrete by applying the maturity concept, developed by Freiesleben Hansen and Pedersen (1977).

In the maturity concept, the equivalent age maturity function is given by

\[
M = t_e(T_r) = \sum_{0}^{\infty} \exp \left[ \frac{E}{R} \left( \frac{1}{273 + T_r} - \frac{1}{273 + T_C} \right) \right] \Delta t.
\]  (4.38)

where \( t_e(T_r) \) is the equivalent age at the reference curing temperature, \( \Delta t \) the chronological time interval, \( T_C \) the average concrete temperature during the time interval, \( T_r \) the constant reference temperature, \( E \) the activation energy and \( R \) the universal gas constant. With the application of the maturity concept, the \( \frac{\Delta T_h}{\Delta t} \) at any arbitrary curing temperature can be approximated from the change in adiabatic temperature with time, \( \frac{\Delta T_{ATR}}{\Delta t} \) at the same maturity, as

\[
\frac{\Delta T_h}{\Delta t} = \left( \frac{\Delta M_h}{\Delta t} \right) \left( \frac{\Delta t}{\Delta M_{ATR}} \right) \frac{\Delta T_{ATR}}{\Delta t}.
\]  (4.39)

Substituting Eq. (4.38) into (4.39), \( \frac{\Delta T_h}{\Delta t} \) can be obtained from

\[
\frac{\Delta T_h}{\Delta t} = \exp \left[ \frac{E}{R} \left( \frac{1}{273 + T_{ATR}} - \frac{1}{273 + T_h} \right) \right] \frac{\Delta T_{ATR}}{\Delta t}.
\]  (4.40)

The accuracy of the approximation will increase when \( \Delta t \) is small.

Assuming piecewise linear at an appropriate time interval \( \Delta t \), which in this case \( \Delta t = 0.5 \) hours was adopted, the equivalent age maturity at reference temperature \( T_r = 20^\circ C \) was determined. Next, the adiabatic temperature rise with time \( \frac{\Delta T_{ATR}}{\Delta t} \) at the same equivalent age maturity at the same reference temperature was obtained from adiabatic temperature rise test result in Fig. 4.8 and
Chapter 4: Determination of Early age thermal diffusivity - An Analytical Approach

Table 4.1. Hence, the change in temperature rises of the specimen with time due to hydration, \( \Delta T_p/\Delta t \) with respect to testing time can be computed from Eq. (4.40) knowing the corresponding adiabatic and specimen core temperature, \( T_{ATR} \) and \( T_h \), respectively, at the same equivalent age maturity. Knowing \( \Delta T_h/\Delta t \), the diffusivity of the concrete evolving with time can then be determined from Eq. (4.28).

Table 4.1 Calculation of thermal diffusivity from experimental data.

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### Chapter 4: Determination of Early age thermal diffusivity - An Analytical Approach

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<td>4.0711</td>
<td>412.0157</td>
<td>80.38</td>
<td>0.1384</td>
</tr>
</tbody>
</table>
The diffusivity of the concrete determined henceforth is shown in Fig. 4.9.

Values at the beginning of the test and immediately after each peak and trough of the test specimen’s core temperature plot shown in Fig. 4.6 has been discarded as temperature within the test specimen at these instants are not at steady state yet.

Fig. 4.8 Adiabatic temperature rise of concrete at the corresponding equivalent age at reference curing temperature of 20°C
4.7 Results and discussions

The result shows that thermal diffusivity decreases as the concrete hydrates and then attain a constant value of 0.0007m$^2$/hr thereafter. The diffusivity of concrete was found to be significantly at early ages approximately before 30 hours tending to a constant value thereafter.

Higher moisture content of concrete at early age would induce higher thermal conductivity and specific heat capacity. However, the influence of moisture content on thermal conductivity of the concrete seems to be more significant contributing to the higher thermal diffusivity at early age.

![Graph showing variation of concrete thermal diffusivity with time.](image)
CHAPTER 5

EARLY AGE THERMAL STRESS ANALYSIS ON MASSIVE RAFT FOUNDATION

5.1 Introduction

An accurate thermal stress analysis needs accurate inputs to determine the temperature development and stress history. Three main factors to be considered in thermal stress analysis are temperature development in the concrete being cast, mechanical behavior of the young concrete and the degree of restraint imposed on the concrete (Springenschmid, 1998). The accurate representation of heat of hydration, thermal conductivity and specific heat capacity is critical for temperature development prediction. Similarly, thermal expansion coefficient, Young’s modulus and Poisson’s ratio are equally important to determine the strain development and thereby the prediction of stress. Due to the difficulties in determining early age thermal properties accurately, constant values have been used for most of the thermal stress analysis. A new method which has been developed to determine the thermal diffusivity evolving with time was covered in the previous chapter and the measurement of the time-dependent coefficient of thermal expansion will be covered in this chapter.

This chapter will discuss on the numerical analysis carried out using the finite element software, ABAQUS, to model and accurately predict the temperature
and stress development in a raft foundation. The input material parameters are obtained from conducting elaborate laboratory test on concrete samples obtained from the site. Appropriate subroutines were also written to supplement the finite element software to handle time-dependent material properties. Results obtained from the numerical analysis were compared with the temperature and strain monitored on site using appropriate instrumentations. The result clearly demonstrates the consequences of using time-independent constant material parameters as against the use of materials parameters evolving with the hydration of cement.

5.2 Experimental studies on raft foundation

The mass concrete structure to be analyzed is a section of the raft foundation constructed at Fusionpolis and One-north MRT station project in Singapore and is considered to be one of the deepest basement construction projects. The casting of raft foundation was divided into many segments and only three segments namely CS1, CS2 and CS3 were considered in this study. The construction of each segment was done by simultaneously pumping concrete from various points. Each segment was cast consecutively one after the other after a prescribed time interval. The time lags in casting CS2 and CS3 with that of CS1 were 192 hours and 310 hours, respectively. The depth of the raft foundation varied between 3m and 5m and the boundary of the two thicknesses are demarcated by dotted lines in Fig. 5.1. Various laboratory tests were performed on the samples of the concrete mix (Table 5.1) as will be illustrated later.
5.2.1 Site Monitoring

Temperature and strain monitoring were carried out in the raft foundations at locations shown in Fig.5.1.

---

**Fig.5.1 Details of raft foundation (A, B and C are locations of vibrating strain gauges at midsection of raft foundation)**

**Table 5.1 - Type of concrete materials and their mix proportions**

<table>
<thead>
<tr>
<th>Materials</th>
<th>Cement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Portland blast furnace cement</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>Natural sand/manufactured sand</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>Crushed granite</td>
</tr>
<tr>
<td>Admixture</td>
<td>Mira 99, Mid range water reducing.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mix proportion (SSD per m³ of concrete)</th>
<th>Cement (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Fine aggregate (kg/m³)</th>
<th>Coarse aggregate (kg/m³)</th>
<th>Admixture (ltrs)</th>
<th>A/C</th>
<th>W/C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>380</td>
<td>160</td>
<td>780</td>
<td>1000</td>
<td>3.42</td>
<td>4.68</td>
<td>0.42</td>
</tr>
</tbody>
</table>
The temperature and strain measurements in the raft foundation were recorded at 30 minutes time interval continuously for one month period after the casting of CS1, CS2 and CS3. The equipment embedded to monitor the temperature and strains within the raft foundation was vibrating strain gauges. It was properly tied with reinforcement bar at various depths and positions, prior to pouring of the concrete. The locations were selected for monitoring are along the depth of mid section and near the top and bottom of the raft foundation. The standard model (Model 4200) vibrating strain gauge which has a gauge length of 153 mm was used. The operating principle behind the vibrating wire strain gauges is that the length of steel wire is tensioned between two end blocks that are directly embedded in concrete.

![Fig.5.2 Embedded vibrating wire strain gauges.](image-url)

The relative movements of those two end blocks were caused as a result of thermal strain in the mass concrete due. These deformations altered the tension in steel wire which is directly converted as resonant frequency of vibration using an
electromagnetic coil. Fig.5.2 and Fig.5.3 illustrate the installation of embedded vibrating wire strain gauges at site.

Fig.5.3 Installation of embedded vibrating wire strain gauges at site.

5.3 Laboratory Tests

Various tests were performed in the laboratory under controlled conditions. The time dependent material properties of concrete at early age was essential to predict the thermal stresses accurately such as, setting time, adiabatic temperature rise, compressive strength, elastic modulus, coefficient of thermal expansion, autogeneous shrinkage, thermal conductivity, thermal diffusivity and creep of concrete. Samples were collected from site and the above mentioned tests were preformed. The test details are described as below.
Chapter 5: Early age thermal cracking on massive raft foundation

5.3.1 Setting time

Setting time is important to determine the onset of the development of mechanical stress in the concrete. Before the setting time, it was assumed that concrete was still in plastic state and no stress development takes place in the raft foundation. The test procedure was according to ASTM C 403/C 403 M-99, ‘Standard test method for time of setting of concrete mixtures by penetration resistance’. Fresh concrete was first sieved using 4.75 mm sieve in order to remove coarse aggregate. The mortar was mixed thoroughly and placed in a 150 mm cube steel mould. Penetrometer was used to measure the force required to cause a prescribed penetration of a needle into the mortar. Fig. 5.4 shows the tested sample and Penetrometer apparatus.

Fig. 5.4 Tested sample and penetration resistance apparatus

5.3.2 Compressive strength

Concrete mix was collected from the site and 100 mm cube specimens were prepared accordingly. Fig. 5.5 shows the specimen preparation for various tests. Compressive strength tests were carried out on the cube specimens at 3, 7 and 28
days in accordance with BS 1881: part 116:1983 using Avery Denison compression machine. The cubes were properly moist cured in a fog room until the day of testing. At all the different ages, constant loading rate of 200 kN/min was applied on the cube to determine the compressive strength.

5.3.3 Elastic Modulus

Specimens of diameter 100 mm and length 200mm were used to find the modulus of elasticity at 3, 7 and 28 days. The modulus of elasticity tests were carried out as per ASTM standards. Four transducers were used to measure the compressive strain and each transducer was mounted in the middle 100mm section of the cylinder. Load data and lateral strains were measured through data logger which was connected to computer.

Fig. 5.5 Specimens preparation for compressive, modulus of Elasticity and creep tests.
5.3.4 Creep test

Basic creep tests were also conducted. Cylindrical specimen’s of diameter 150 mm and length 300mm were used and the deformation of specimen was measured under constant compression. The specimen preparation and testing procedure was according to ASTM C512-76. After demoulding, test specimens for basic creep and drying creep were sealed with two layers of adhesive aluminum tape so as to ensure airtight sealed conditions by preventing the moisture exchange between environment and specimen. Openings were provided by removing aluminum tape on each side of prism and a pair of Demec pins was attached. Before loading, specimens were kept under controlled environment at temperature of $30 \pm 2^\circ\text{C}$ and relative humidity of $65 \pm 5\%$. After measuring the compressive strength of the concrete, a compressive stress of approximately 40 % of compressive strength was applied for the creep test. This is in line with the assumption the proportionality of deformation with applied stress is linear up to 40 % of the compressive strength.

5.3.5 Adiabatic temperature rise

Solid state calorimeter was used to measure the rate of heat evolution during the hydration processes. Samples were collected from the site and then tested in the laboratory. Temperature of the concrete was measured using T-type thermocouple and readings were recorded using data logger at regular intervals until the maximum temperature was attained. Measured temperature rise curves ATR1, ATR2 and ATR3 for CS1, CS2 and CS3 concreting, respectively, are plotted and discussed in the subsequent section.
5.3.6 Early age CTE - using Kada et al method

Tests were conducted on fresh concrete and cast in a steel mould of size 100 x 100 x 400mm. Before placing concrete the steel mould, KM 100B (Tokyo Sokki Kenkyujo Co Ltd) embedment strain gauges and thermocouples were installed along the longitudinal axis of specimen centre which was connected to the data acquisition system for monitoring the temperature and strains during testing as shown in Fig.5.6 and Fig.5.7

![Fig.5.6 Installation of KM 100B strain gauges along specimen center.](image)

The testing procedure is described as follows. Both the samples were de-moulded when it begin to set i.e. time to achieve good consistency for de-moulding. Then, the specimens were placed in a precise heat controlling chamber to cycle the specimen temperature at amplitude of 40°C.
Initially the specimen temperature was 30°C and it was gradually increased while the strain is monitored. As soon as the temperature reaches 70°C, the specimen temperature was gradually decreased until it reaches 30°C during which time the strain was continuously monitored. This cycle was repeated many times until the concrete has presumably completed full hydration, after which the concrete properties would not be time-dependent anymore. Fig.5.8 illustrates a typical temperature cycle during the testing. The actual or true strain based of the concrete taking after correcting for the effect of temperature on the embedded strain gauge were computed as follows:

\[ \varepsilon_{\text{u}}(t) = C_\varepsilon \varepsilon_i + C_\beta \Delta T \]  \hspace{1cm} (5.1)
where $\varepsilon_i$ is the actual strain, $\varepsilon_i$ the measured strain, and $C_\varepsilon$ and $C_\beta$ the calibration and temperature compensation coefficient, respectively.

![Fig.5.8 Illustration of temperature cycle of specimen](image)

### 5.3.7 Autogeneous shrinkage

Two prismatic specimens of size 100 x 100 x 400mm were used to measure the autogenous shrinkage. Before preparing the concrete, KM 100B embedded strain gauges and thermocouples were installed as in the case of expansion test along the longitudinal axis of two specimens at the centre to measure strains and temperature variations. The specimens were de-molded after the setting time. Then, the specimens were immediately sealed with two layer of adhesive aluminum tape in order to prevent any hygrometric exchange with the environment. Measurements were recorded using precision data acquisition system and strains were calculated using calibration coefficient $C_\varepsilon$ provided by the strain gauge manufacturer as
\[ \varepsilon_i(t) = C_i \times \varepsilon_i \]  

(5.2)

5.4 Determination of early age thermal properties – Proposed new method

5.4.1 Thermal expansion

Kada et al (2002) method to obtain the early age thermal expansion is by applying cyclic thermal shock while corresponding thermal strains were measured between peaks. As shown in Fig. 5.8 only one thermal expansion coefficient was taken from each thermal cycle despite the value may vary over a wide range during the 6 hours interval. However, it was realised that there are greater limitation with this method. The sudden rate of heating and cooling of the concrete led to drastic change in temperature differential gradient in the concrete specimen. The changes in oven temperature as well as the change of temperature in the concrete were different which caused differential strain throughout the concrete. Thus only the intersection of the concrete and oven temperature could be taken as the reference point to calculate the thermal expansion coefficient. The values computed from this test were widely dispersed and hence large error was obtained.

To overcome this difficulty, the oven temperature was set to \( \pm 5^\circ C \) of the specimen core temperature when the specimen has attained a prescribed minimum or maximum value, respectively.

This method of testing was initially proposed to determine thermal diffusivity and subsequently found to be useful to determine the coefficient of thermal expansion at early ages, simultaneously. This experimental procedure was performed continuously to measure the thermal expansion which varies greatly during early age. In this method, a cylindrical specimen embedded with a strain
gage was subjected to a variable temperature of +5°C of the core temperature in the first cycle and thereafter at -5°C of the core temperature in the second cycle as shown in Fig.5.9.

This would allow for the temperature rise to be uniform across the specimen. These cycles were repeated for 72 hours after which the thermal expansion and diffusivity did not change much with time. The difference of temperature for calculation of thermal expansion coefficient will also be constant thus the thermal expansion will be the same throughout the concrete. Readings were taken after final setting time of concrete, and temperature and strain were measured by T-type thermocouples and embedded strain gangue (KM-100B) respectively. This method allows the thermal expansion coefficient to be taken constantly at any time of the thermal cycle (Fig.5.10), and measured strain (Fig.5.11)

The coefficient of thermal expansion may be given by Eq. (5.3) as

$$\alpha(t) = \frac{d e^{\text{thermal}}}{d T} = \left( \frac{d e^{\text{total}}}{d t} - \frac{d e^{\text{shrinkage}}}{d t} \right) \frac{d t}{d T} \quad (5.3)$$

The true thermal strain of concrete during test was measured by separating shrinkage strain from the total strain. In this analysis autogenous shrinkage of concrete was considered. For thermal analysis of mass concrete structures, usually the effect of drying shrinkage are not taken into account which may be a good approximation for early age concrete of massive structures.
Chapter 5: Early age thermal cracking on massive raft foundation

Fig. 5.9 Cylindrical specimen for proposed method

\[ T_o = T_i + 5^\circ C \]

Fig. 5.10 Temperature cycle obtained - proposed new method
Chapter 5: Early age thermal cracking on massive raft foundation

Fig. 5.11 Corrected real strain reading from strain gauge

Fig. 5.12 Coefficient of thermal expansion of concrete on ages
Chapter 5: Early age thermal cracking on massive raft foundation

Fig. 5.12 shows the thermal expansion coefficient of concrete significantly changing from $17.27 \times 10^{-6}/^\circ\text{C}$ at early age to $9.45 \times 10^{-6}/^\circ\text{C}$ with aging. At very early age of 11 hours, the coefficient of thermal expansion was high and then steadily decreases with age until approximately 50 hours and thereafter it levels off. This high expansion at early ages appears to be affected by free water.

5.4.2 Thermal Diffusivity

From the same experiment, thermal diffusivity of concrete at early age could also be obtained from the rate of change of core temperature with respect to time. Thermal diffusivity was obtained using mathematical model discussed in previous Chapter 4. It was then used for the thermal stress analysis accordingly. Thermal diffusivity is an important parameter for modeling which determines the heat flow and temperature distribution in mass concrete. The time taken to rise $\Delta T$ was observed during test.

5.5 Material properties for temperature and stress analysis

In early age concrete, the coefficient of thermal expansion was observed to be high due to the presence of free water. ACI Committee (517) report mentioned that the coefficient of thermal expansion attains the higher value for very young concrete. And the same was concluded that coefficient of thermal expansion drastically decreases and reaches a constant value when it get hardens (Kada et al. 2002 and Shimasaki et al. 2002). The shrinkage of concrete must be considered to delineate the thermal expansion test. If not, the obtained CTE will be representing both shrinkage and thermal movements. The total free strain development originates
in young concrete from a combination of thermal expansion and shrinkage. The shrinkage strain includes the internal drying due to heat of hydration and temperature and moisture movement.

In the analysis being presented here considered the autogenous shrinkage of concrete. Autogeneous shrinkage occurring in mass concrete structures can be superposed on the volume change caused by temperature (JCI, 1998). It was pointed out the effect of autogenous shrinkage should be included in the thermal stress calculations which contributes to the risk of early age cracking. The reason behind is that the risk of cracking requires the information about the heat of hydration process, to which autogenous shrinkage is closely related. For thermal analysis of mass concrete structures, usually the effect of drying shrinkage is not considered which may be a good approximation for early age concrete of massive structures. The user developed FORTRAN subroutines was used to implement the thermal expansion and shrinkage effect as a function of time in the finite element simulation.

The visco-elastic behavior of young concrete is a crucial phenomenon for thermal stress analysis to describe the real behavior of young concrete. Soon after casting of concrete, it changes from liquid state to solid state. Studies on realistic behavior of young concrete require the knowledge on development of modulus of elasticity, stress relaxation due to creep (Springenschmid, 1996). Creep of concrete is expressed in terms of creep compliance which is calculated under the constant load. The use of creep coefficient instead of creep compliance in the creep formulation requires the value of the E-modulus at the age of load application. The errors will be accomplished if E-modulus is obtained from the various test series. Creep compliance can be included in the analysis through incremental visco-elastic
Chapter 5: Early age thermal cracking on massive raft foundation

phenomena to avoid such errors. Those creep measurement have an assumption that
the rate of creep strain is same for the given applied load and loading time. As result
of that, the creep curves are parallel to each other for all ages. Thus, a single curve is
adequate to calculate creep for all subsequent ages (Neville et al. 1983).

At early ages, relatively higher creep deformations and relaxation of concrete
will reduce the development of thermal stresses (Cusson, 2001) and reduces the risk
of cracking. Most of existing creep models, failed to generalize the formulation of
the creep of young and mature concrete. Bazant proposed the creep response based
on solidification theory which also fails to represent the early age creep behavior. It
has been rectified with reasonable accuracy by applying Linear Logarithmic model
(Larson and Jonasson 2003). This model requires only few test data to describe the
actual behavior of both concrete. The Linear logarithmic model (LLM) approach
was used in the analysis because of the above mentioned advantages. The
development of E-modulus and creep compliance as function of time is shown in
Fig.5.13 and Fig.5.14. The rate of creep strain increments has been calculated from
time dependent deformation $\varepsilon^{cr}(t,t_o)$ which may be expressed in terms of creep
compliance as

$$J(t,t_o) = \frac{1}{E(t_o)} + \Delta J(t,t_o) \tag{5.4}$$

$$\varepsilon^{cr}(t,t_o) = J(t,t_o) \times \sigma_{c}(t_o) \tag{5.5}$$

and creep compliance $J(\Delta t_{load},t_o)$ is described by piece wise linear curve
logarithmic of time span after loading $\Delta t_{load}$.

With effect of early age thermal expansion, autogenous shrinkage and creep,
strains can be calculated and then stress can be calculated from strain history.
Chapter 5: Early age thermal cracking on massive raft foundation

\[ \varepsilon(t) = \int_0^t (\varepsilon^{\text{cr}}(t) + \alpha(t) \Delta T + \varepsilon^{\text{inf}}(t)) \, dt \]  

(5.6)

Fig. 5.13 Development of Modulus of elasticity of concrete varying with age

Fig.5.14 Creep Compliance \( J(\Delta t_{\text{load}}, t_0) \) with varying loading age \( \Delta t_{\text{load}} \)
Chapter 5: Early age thermal cracking on massive raft foundation

5.6 Finite element Analysis - ABAQUS

A time history analysis was performed that closely models the incremental construction processes. The three dimensional thermal stress analyses were performed on the modeled raft foundation to obtain temperature and stress history as a function of time. An incremental, finite element simulation was performed to obtain the thermal and mechanical solutions simultaneously while considering the solutions of each other affecting strongly. Initially, nodal temperature, as function of time was calculated through heat transfer analysis and then the stress distribution was calculated which depends on the temperature distribution and time dependent mechanical properties.

The heat evolution of hydration process increases the temperature in massive concrete substantially. The rate of heat evolution depends on concrete mix and the rate at which heat is dissipated. A constant heat of hydration curve was considered for studying the combined effect of other parameters because the temperature rise mainly depends on the heat generated due to hydration of cement content. It is well known that higher temperature gets accumulated in case of large concrete structures due to high volume to surface area ratio and thereby induces large thermal strains. The heat of hydration as a function of time can be calculated by

\[ Q = C \rho \frac{\Delta T}{\Delta t} \]  

(5.7)

where \( C \) is the specific heat in J/kg K, \( \rho \) the density in kg/m\(^3\), \( \Delta T \) and \( \Delta t \) the change in temperature and time, respectively. Tests were conducted on site concrete under laboratory controlled adiabatic conditions. Fig.5.15 shows the results of adiabatic temperature rise curves of ATR1, ATR2, and ATR3 for CS1, CS2, and CS3 respectively.
Chapter 5: Early age thermal cracking on massive raft foundation

The adiabatic curves as a function of time are input into the numerical analysis through user developed FORTRAN subroutine. It ensures the heat of hydration took place as per construction sequence at job site.

The combinations of parameters for the various load cases to perform the parametric study under constant thermal loads are shown in Table 5.3. The important parameters considered for the thermal analysis of mass concrete structure are adiabatic temperature rise, thermal conductivity, specific heat, ambient temperature, concrete placing temperature, early age thermal expansion, autogeneous shrinkage and creep/relaxation.

Fig. 5.15 Adiabatic temperature rise curve of ATR1, ATR2, ATR3 for CS1, CS2, CS3 respectively
5.6.1 Boundary conditions

The concrete segments were analyzed as being cast at different durations to simulate the different thermal and mechanical boundary conditions. In order to predict the temperature and stress development, the transient material properties, boundary conditions and geometry should be considered as realistically as possible, reasonably. The mean daily ambient temperature as a function of time is shown in Fig. 5.16.

![Mean daily temperature (Singapore)](image)

**Fig. 5.16 Mean daily temperature (Singapore)**

The main difficulty of boundary conditions is how to model the boundary conditions according to construction activities and especially introducing the transient effect of weather conditions on surfaces. The placement temperature of all concreting was taken as initial temperature for heat transfer analysis. It was assumed that placement temperature of concrete does not exceed the average daily
Chapter 5: Early age thermal cracking on massive raft foundation

temperature. The heat transfer between the surfaces of a concrete and its surroundings takes place in the form of convection, radiation and conduction.

The environmental interactions on the top surface include the effects of convection heat transfer to the atmosphere and effects of solar radiation heat gain whereas the bottom surface was treated as surface exposed to conductive heat transfer mechanism only. In the heat transfer analysis, convection coefficient was based on forced convection mode. It refers to dynamic heat transfer at the interface between a fluid (air) and solid surface of Newton’s cooling law. All the concreting were done on a 150 mm thick raft concrete base and not placed directly on soil. The concrete placement at the job site took place at a faster rate by pumping concrete through various terminals. After casting, top surface of the concrete was completely covered with expanded polystyrene board. Other surfaces were surrounded by previously cast concrete except the vertical surfaces upright to construction joints.

Construction joints are exposed to ambient conditions till casting of next segment. The effects of with forms and without forms were considered in terms of convection coefficients in the heat transfer analysis. There was no thermal insulation on the surfaces considered and as a result this, the maximum temperature in the concrete would increase.

Solar radiation was considered in the heat transfer analysis which was governed by the absorptivity of the concrete material. The heat exchange between the surface concrete structure and the atmosphere through radiation can be estimated with the Stefan-Boltzmann constant. It was essential to define appropriate mechanical boundary conditions for the stress analysis. Free movements were allowed along the construction joints and top surfaces of all concreting. Support
Chapter 5: Early age thermal cracking on massive raft foundation

provided by construction joints and the expanded polystyrene board were not included in the simulation. This is due to the complex nature of determining the degree of restraint whereas restraint boundary conditions were defined on the remaining concrete surfaces.

All other boundary conditions, both thermal and mechanical, were executed according to construction processes. Table 5.2 shows the various parameters used for heat transfer and stress analysis.

**Table 5.2 – Various parameters used for thermal stress analysis**

<table>
<thead>
<tr>
<th>Material parameters</th>
<th>Value/Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apparent setting time</td>
<td>11 hours</td>
</tr>
<tr>
<td>Initial concrete temperature of concrete</td>
<td>27.6°C</td>
</tr>
<tr>
<td>Mean ambient temperature</td>
<td>27.5°C</td>
</tr>
<tr>
<td>Convective heat transfer coefficient</td>
<td>35 W/m²/°C</td>
</tr>
<tr>
<td>Net radiation coefficient</td>
<td>0.88</td>
</tr>
<tr>
<td>Stefan-Boltzmann constant</td>
<td>$5.67 \times 10^{-8}$ W/m²/K⁴</td>
</tr>
<tr>
<td>Modulus of elasticity at 28 days</td>
<td>32.23 GPa</td>
</tr>
<tr>
<td>Compressive strength of concrete at 28 days</td>
<td>48.71 Mpa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.22</td>
</tr>
</tbody>
</table>

5.6.2 Load cases considered

Three loadings, namely early age thermal expansion, shrinkage and creep /relaxation of concrete, were considered in various combinations for the stress calculations to understand its effects in mass concrete structures. Table 5.3 shows three load cases considered in the analysis. Finite element simulation was carried out for the three load cases under constant thermal loads. The results were compared with site measurement.
Table 5.3 Load cases considered for thermal stress analysis

<table>
<thead>
<tr>
<th>Load cases</th>
<th>Thermal expansion</th>
<th>Shrinkage</th>
<th>Creep /Relaxation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case I</td>
<td>Constant</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>Case II</td>
<td>Time dependent</td>
<td>Included</td>
<td>None</td>
</tr>
<tr>
<td>Case III</td>
<td>Time dependent</td>
<td>Included</td>
<td>Included</td>
</tr>
</tbody>
</table>

Fig. 5.17 Finite Element Mesh – Raft foundation

A transient coupled thermal-stress analysis was carried out for the above mentioned load combinations under constant thermal loads. The total period of time taken in the transient analysis was 510 hours and a fixed time increment of 1 hour was chosen. 3-D solid plane strains element (Quadratic coupled temperature displacement, C3D20RT) was chosen. The reduced integration process was applied to calculate the temperature and stress distributions. The finite element mesh modeled is shown in Fig.5.17 which has 610 elements and 3395 nodes.

The temperature field was obtained at the beginning of the analysis. Stress calculations started after 11 hours of casting of each segment based of the setting
time obtained from the laboratory experiments. The risk of cracking was evaluated using the fundamental concept which states that if the highest principle tensile stress reaches the actual tensile strength of concrete, cracking would be imminent.

5.7. Results and Discussions

5.7.1 Temperature prediction on raft foundation

The temperature prediction in the raft foundation primarily considered the two basic mechanisms, the heat of hydration and environmental effects. The effects of these mechanisms were reflected in the actual concrete temperature which was measured by vibrating strain gages placed at site. The temperature field in the raft foundation was determined as a result of heat exchange process between the concrete mix and the surrounding environmental effects. The initial temperature of concrete \( T_0 \) of the respective concreting was chosen with assumption that the placement temperature of concrete does not exceed the average daily temperature.

![Fig. 5.18 Measured and predicted Temperature varying with time at midsection A of CS1 concreting](image-url)
Chapter 5: Early age thermal cracking on massive raft foundation

CS2 concreting at midsection B

Fig. 5.19 Measured and predicted Temperature varying with time at mid section B of CS2 concreting

CS3 concreting at midsection C

Fig. 5.20 Measured and predicted Temperature varying with time at mid section C of CS3 concreting
Chapter 5: Early age thermal cracking on massive raft foundation

The temperatures predicted by finite element simulation of the raft foundation were validated with site measured data at locations A, B and C. The result of predicted and measured temperature at location A of CS1 is presented as a function of time in Fig.5.18.

The maximum temperature of 77.8°C was observed after 100 hours of concrete casting and the temperature then decreases slowly due to net heat dissipation. Fig.5.19 and Fig.5.20 shows the temperature development at locations B and C of CS2 and CS3 segments respectively. In these two locations, the temperature development increases steadily and peak temperature of 74.50°C and 69.95°C were predicted after 59 hours and 90 hours of casting. The highest temperature differential at section A of CS1, B of CS2 and C of CS3 are 49.80°C, 46.50°C and 41.95°C respectively.

The effect of casting of concrete in adjacent segments has induced the additional heat transfer to the adjacent concrete which produces additional temperature rise to concrete. This is because of the fact that the temperature cycles are more evenly matched and there is an additional benefit of reduction in the temperature differential. In theory, CS2 generates heat and some of these gets transformed into raising the temperature in adjacent areas of CS1. This reduces the differential temperature in CS1. The differential temperature in CS2 is also reduced because CS1 has an insulating effect along the common boundary and similarly between CS2 and CS3 segments.

Thermal strains corresponding to constant thermal expansion and transient thermal expansion as a function of time was plotted against adiabatic temperature rise of CS1, CS2 and CS3 (Fig.5.21, Fig. 5.22 and Fig 5.23). In Fig.5.21, Fig. 5.22
Chapter 5: Early age thermal cracking on massive raft foundation

![Graph showing thermal strains due to ATR1 and ATR2](image)

**Fig. 5.21** Early age thermal expansion effect on the thermal strains due to ATR1

**Fig. 5.22** Early age thermal expansion effect on the thermal strains due to ATR2
Fig. 5.23 Early age thermal expansion effect on the thermal strains for due to ATR3

and Fig 5.23, an increase in strain occurs due to thermal loads. In the plots, the thermal strains were measured using constant CTE and CTE as function of time, respectively. It shows that higher strains are not only due to the effect of early age thermal expansion, but it also depends on the steeper temperature gradient. At early age, measured expansion was affected by presence of free water. In this case, concrete will experience more compressive stresses and thereby reduces the risk of cracking at later age.

5.7.2 Stress predictions in raft foundation

Stress calculations were performed as explained for the load cases in Table 5.3 where the material models describing the time dependent thermal expansion, shrinkage, strength development and creep/relaxation were explained. The stress
field over time period can be computed as a result of temperature gradients and restraint conditions. The predicted stresses for load cases were compared with the site measurement. The stress development at the specific point in the raft foundation has been computed by setting all the stresses to zero at the setting time of concrete $t_s = 11$ hours. Fig.5.24, Fig.5.25 and Fig.5.26 shows the variation in stresses developed with time obtained from the numerical analysis compared with site measurement. The site measured stresses were obtained from the measured restrained strain. The sign conventions used for the stress analysis is negative for compression and positive tension.

As expected, higher temperature in concrete produces significantly higher compressive stresses rather than tensile stresses in core region. The development of thermal stresses is strongly affected by thermal expansion. The higher thermal expansion at early age contributes to increase in the thermal strain which produces more compressive stress after the placement of concrete. This is mainly due to considering the coefficient of thermal expansion as a function of time. This reduces the risk of cracking at later age.

From the figures, it is observed that the effect of increase in thermal loading causes the element to generate more compressive stresses. Then the compressive stresses decreases slowly. This is due to the decrease in thermal loading in concrete which causes the concrete element to shrink. It is well known that thermal strain is not the only major cause of potential cracking at early ages in mass concrete structures. It may also be due to the contribution of autogenous shrinkage. For concrete having low heat cement or low water/binder ratio, autogenous shrinkage may induce considerable Eigen stresses and cracking. From the plots (Case I and
Case II), it can be concluded that consideration of shrinkage gradually reduces the stresses with time. And it is observed to be less significant because strain due to autogeneous shrinkage of concrete was small compared to the strain due to temperature change. The measured autogenous shrinkage was closer to $100 \times 10^{-6}$. The thermal stress generation will be affected by autogeneous shrinkage when the variation of autogeneous shrinkage strain is larger than that of thermal strain even if the temperature of concrete still increases.

Visco-elastic behavior of concrete was included in the prediction of stresses to describe the stress relaxation due to creep. At early age, mass concrete produces low strength, elastic modulus and high creep rates which results to reduce the stresses. The creep effects reduce the development of stresses considerably in the raft foundation. The prediction of risk of cracking was evaluated with Case III and the maximum tensile stress of $3.79 \, \text{N/mm}^2$ was observed in CS1 segment 92 hours of casting. Similarly, highest tensile stresses of $2.85 \, \text{N/mm}^2$ and $2.93 \, \text{N/mm}^2$ were predicted in CS2 and CS3 segments after 77 hours and 68 hours, respectively.

In comparing with the estimated tensile stress based on CEB-model, the maximum tensile stresses in CS1, CS2 and CS3 exceed the margin (Fig.5.27). And also reference to the “rule of thumb” maximum temperature differential criteria, the maximum temperature differential between core and surface exceeded the limiting temperature differential as expected. This was validated by finite element simulation. The casting of concrete in adjacent segments within few days affects the temperature and thermal stress development. In addition to the reduction in differential temperature, there is an additional benefit because of the adjacent
Chapter 5: Early age thermal cracking on massive raft foundation

concreting. The adjacent concrete is relatively young while casting the other segments and hence is less stiff than if the casting was delayed.

![Fig. 5.24 Stress development at mid section A of CS1 concreting](image1)

![Fig. 5.25 Stress development at mid section B of CS2 concreting](image2)
Chapter 5: Early age thermal cracking on massive raft foundation

![Stress development at mid section C of CS3 concreting](image)

**Fig. 5.26 Stress development at mid section C of CS3 concreting**

![Predicted tensile strength development (CEB- Model)](image)

**Fig. 5.27 Predicted tensile strength development (CEB- Model)**

Therefore, lesser restrain is induced. Based on this study, thermal cracking was expected to develop in the raft foundation. This could have been minimized by
Chapter 5: Early age thermal cracking on massive raft foundation

the use of appropriate construction techniques. A detailed study on optimizing all influencing parameters in mass concrete construction will help to control the potential cracking.
The following conclusions are drawn based on the research study:-

1) **Thermal properties of various concretes:-**

   The measured CTE and thermal conductivity of concrete made of foam, Leca, Liapor and pumice were lower than that of normal weight concrete. Both thermal expansion and thermal conductivity of all concretes are directly proportional to the density but their value depends on the variations of concrete mixes, size of pores and their distribution in the concrete. Based on this study, a general equation was obtained to predict the thermal conductivity ($k$) of concrete from oven dry density of concrete having density ($\rho$) ranging from 800 kg/m$^3$ to 2400 kg/m$^3$

   \[
   k = \left[0.090e^{(0.0012 \times \rho)}\right]
   \]

   The measured CTE of concrete made of Leca and Liapor are approximately 10~15% lower than that of NWC whereas CTE of foam concrete was observed to be approximately 10~50% lesser. Significant increase is observed in the measured CTE and thermal conductivity when sand is added to foam concrete. Increasing the content of ground granulated blast furnace slag (GGBS) and silica fume (SF) in concrete
Chapter 6: Conclusions

decreases the thermal conductivity. CTE of concrete in general increases within the tested temperature range irrespective of type of concrete.

Thermal conductivity of lightweight concrete increases marginally with temperature unlike in NWC where it decreases. Lightweight concretes are made of amorphous aggregates which exhibit low thermal conductivity at room temperature which slightly increases with temperature. But NWC has higher thermal conductivity at room temperature and gradually decreases at elevated temperature because normal weight aggregates are crystalline in nature.

2) **Thermal conductivity system:**

In the new thermal conductivity system, the effect of thermal contact sheet of 1mm, 2mm and 3mm thickness was verified in the numerical analysis for varying thermal conductivity of thermal contact material. The percentage of error increases with increasing thickness of thermal contact sheets. The percentage of error in the conductivity measurements was observed to be high when thermal conductivity of thermal contact sheet has value lesser than that of the test sample. This guides the selection of suitable thermal contact sheet for the thermal conductivity measurement. The developed thermal conductivity system was tested for both the standard reference material PTFE and concrete, and the measured results were found to be acceptable.

3) **Thermal diffusivity at early ages (Numerical and Experimental study):**

In the new method proposed to determine the thermal diffusivity, the coefficient of thermal expansion at early ages can also be determined simultaneously. From the
Chapter 6: Conclusions

Experimental results, thermal diffusivity ($\alpha$) of concrete can be determined from the derived equations

$$\alpha = R^2 \left[ \ln \left( \frac{(T_2 - T_1) + \Delta T}{(T_2 - T(0,t)) + \Delta T} \right) - \frac{\Delta T_h}{4((T_2 - T(0,t)) + \Delta T)} \right]$$

And by finite difference method

$$\alpha = \left( \frac{\Delta r^2}{2\Delta t} \right)$$

The experimental result of early age concrete diffusivity was verified by numerical study. The result shows good agreement and thus justifies the reliability of the experimental technique. Physically, a decrease in diffusivity is expected with time at early age because of decrease in free water and an increase in porosity due to hydration. After hydration is complete, the diffusivity becomes relatively stable.

4) Coefficient of thermal expansion at early ages:

The measured thermal expansion coefficient of concrete significantly changes from $17.27 \times 10^{-6}/{^\circ}C$ initially to $9.45 \times 10^{-6}/{^\circ}C$ with age. At very early age of 11 hours, the coefficient of thermal expansion was high and then steadily decreases with age until approximately 50 hours and thereafter it levels off. This high expansion at early ages appears to be affected by free water. In the thermal stress analysis, the significance of taking into consideration the variable CTE has been demonstrated by computing the free strain under adiabatic condition. This was compared with the result obtained using
constant CTE. The use of constant CTE can underestimate the free strain by as much as 30% and this value depends on the temperature gradient.

5) **Thermal stress Analysis (based on Numerical and experimental studies)**

At early age concrete exhibits high creep which reduces the stresses significantly compared to any other influencing parameters and it assists in mitigating thermal cracking by reducing the thermal stress. An accurate simulation of the evolving temperature field and the visco-elastic behavior of mass concrete can be obtained by means of maturity concept based on the three-dimensional finite element method. Finite element simulation carried out for various load combinations under constant thermal loads compared well with site measurement.

The discrepancies of not considering the time-dependent coefficient of thermal expansion in thermal stress analysis can be significant. But the effect of ignoring creep has been found to be most significant. The casting of concrete in adjacent segments within few days reduces the temperature differential and stress development which also provides additional benefit as the adjacent concrete is still relatively young and less stiff, hence less restraint.

The result of this study demonstrates the importance of implementing time dependent material properties in finite element simulation and also demonstrates its significance for accurate thermal stress analysis. A detailed study requires an optimization of all influencing parameters in mass concreting which will help to control potential cracking.
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