Influence of ambient temperature on early age concrete behaviour of anchorage zones

M. Sofi a,⇑, P. Mendis a, D. Baweja b, S. Mak c

a Department of Infrastructure Engineering, The University of Melbourne, Parkville, VIC 3010, Australia
b School of Civil and Environmental Engineering, University of Technology Sydney, NSW 2006, Australia
c Commonwealth Scientific and Industrial Research Organisation, Clayton South, VIC 3169, Australia

HIGHLIGHTS

• This research treats the anchorage zone behaviour post-tensioned slabs exposed to ambient conditions.
• The conclusions from the analytical study will help better predict bearing stresses in early age concrete members.
• The research findings are particularly important for design of post-tensioned members with early age concrete properties.

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ABSTRACT

Many anchorage zone failures of post-tensioned concrete beams and slabs have occurred in the recent past, during stressing stage, prompting urgent attention to investigate the performance of anchorage zone concrete at early age. The strength properties of early age concretes at the time of tendon stressing is significantly influenced by the fluctuation of ambient temperatures. The existing design equation for allowable bearing stress is a function of concrete compressive strength at the time of post-tensioning. This paper reports parametric studies based on finite element modelling to investigate the effects of variable temperatures on concrete strength. The model was validated against experimental results. Allowable bearing stresses were calculated from compressive strength results between 1 and 7 days. These allowable bearing stresses can then be compared with average bearing stresses from the post-tensioning load to evaluate the bearing capacity of the local anchorage zone. Based on the illustrative example presented, it is shown that the values of allowable bearing stress can be exceeded at the final stressing stage. Inadequate compressive strength at early age can contribute to failures of anchorage.

1. Introduction

Concretes used in post-tensioned (PT) concrete slabs have early age strength requirements for initial and final stressing processes [1,2]. This is typically at 1-day age for initial stressing and 3–4-days age for final stressing. The age at which the final stressing is applied depends upon a confident prediction of in situ strength. The normal practice in many countries is to use a minimum compressive strength of 22 MPa for final stressing. After successfully transferring the post-tensioning load, the ends of the tendons are permanently anchored to the concrete at specified locations in the member.

Despite the use of high early strength concrete as the obvious choice of construction material, many failures have been observed in the floor at anchorage zones. Naturally, the introduction of concentrated loads into the slab section produces highly concentrated stress regions immediately ahead of, and surrounding, the anchorage plate. Anchorage failures generally occur at the time when the post-tensioning load is being transferred. Sometimes, they appear within a day or two after loading, in the form of a crack that propagates along the centreline of the anchorage zone for an arbitrary length.

A typical dead-end anchorage failure is presented in Fig. 1. Observations of anchorage failures in multi-storey apartments during construction in Canberra and Melbourne have indicated that anchorage failures are mostly localised. Given that the failures occur abruptly, it is very difficult to establish the sequence of failure; and, therefore, it is not easy to establish for certain the causes of failures. However, a combination of concrete crushing immediately ahead of the anchor plate (referred to as the “local zone”), shearing and debonding of the strand and wire (for dead-end anchors) appear to be the most plausible causes. In cases where the failures appear in the form of longitudinal cracks along the line of the tendon, they are most likely caused by transverse...
tensile stresses (or bursting stresses) which occur at some distance ahead of the anchorage plate (referred to as the “general zone”). The focus of this paper is on the evaluation of bearing strength at the “local zone”. The evaluation of transverse tensile stresses imposed on the “general zone” is outside the scope of this paper.

The tendon forces are transferred to the concrete using an anchorage device that is often proprietary. They are supplied with special bearing plates that have a complex geometry. Special bearing plates generate very high concentrated bearing pressures on the “local zone”. The behaviour of such special bearing plates is not readily evaluated analytically. Therefore, their adequacy must be established by tests [3].

The design equation (Eq. (1)) according to Post-Tensioning Institute (PTI) [4] for basic bearing plates is based on AASHTO [5]. Only the design provision for basic bearing plates which are used in conjunction with minimum local confinement reinforcement (total volumetric reinforcement ratio \( \rho_s \) greater than 0.2%) is provided herein.

\[
f_{\text{epi}} = 0.75f'_{\text{ci}} (A/A_g)^{0.5} \leq 1.5f'_{\text{ci}}
\]

where \( f_{\text{epi}} \) is the allowable bearing stress under the maximum allowable tendon jacking force \( (P_{\text{ax}} = 0.6A_{ps}f'_{\text{ci}}) \), \( A_{ps} \) is the nominal prestressing steel area, \( f'_{\text{ci}} \) is the characteristic compressive strength of concrete cylinder at the time of tendon stressing, \( A \) is the distribution area, \( A_g \) is the gross bearing plate area, \( f'_{\text{ci}} \) is the nominal minimum tensile strength of prestressing steel.

The compressive strength of the early age concretes at the time of tendon stressing is significantly influenced by the fluctuation of ambient temperatures. It is commonly believed that uncertain weather patterns and fluctuating temperatures influence the curing process of concrete.

This paper investigates the effects of temperature variation on the early age properties of concrete which influences the behaviour of the local anchorage zone at the time of post-tensioning. An experimental program which consists of temperature measurement of a block of concrete cured under laboratory conditions and testing of concrete strength properties at early age is presented in Section 3. The degree of reaction approach was adopted to model the age-dependent strength properties of concrete. A finite element model (FEM) simulating the hydration reaction of concrete and temperature development of a concrete block is presented in Section 4. The temperature model was validated by comparison with the experimental data. Parametric studies of the effects of initial temperature and the variation of ambient temperature at early ages on the temperature and strength development properties were conducted based on ambient temperatures measured on two different construction sites in Melbourne, Australia (Section 5). It is noted that the variation of temperature ranging between 10 and 35 °C used in this study represents the temperature variation in temperate zones. Therefore, the outcomes presented in this paper are limited to those regions, although the approach applied and the methodology developed can be adopted for other regions. The in situ compressive strength of concrete at early age was assessed by comparing the allowable bearing strength defined by Eq. (1) with imposed bearing stress at the time of load transfer (Section 6). This paper focuses on the effects of ambient temperature on bearing strength and the adequacy of concrete immediately ahead of the anchor plate. It is noted that there are other possible failure mechanisms surrounding the anchorage zone such as bursting failure. Discussion of these failure mechanisms are outside the scope of this paper.

### 2. Research significance

As mentioned earlier, catastrophic failures in anchorage regions of post-tensioned slabs and beams have been observed in the recent past. An accurate estimation of strength properties of concrete at early ages is important in assessing its behaviour under post-tensioning load. The strength properties of concrete at early ages depend greatly on the ambient conditions. This study investigates the effects of initial temperature and the daily variation of ambient temperatures on the evolution of strength properties of concrete at early ages. The strength of concrete at early ages is then evaluated by comparison with imposed bearing stress at the time of post-tensioning load transfer. The material presented in this paper should assist designers in assessing the bearing capacity of concrete at the time of pre-stressing in view of the daily ambient conditions. Authors are not aware of any other studies conducted before on bearing capacities of concrete at early age.

### 3. Experimental procedure

#### 3.1. Strength properties

A total of 70 specimens were tested to establish the material properties of concrete at early ages. Preparation of the test specimens for each test was performed in accordance with the relevant Australian Standard for testing of concrete. The tests included compressive, tensile, flexural strength, modulus of elasticity and Poisson’s ratio for the concrete mix. The tests were carried out at 2, 3, 4, 7 and 28 days.

The compressive and indirect tensile cylinders (100 mm diameter × 200 mm height) were cast according to AS 1012.8.1 [6], which sets out the procedure for moulding, compaction and curing of compressive and indirect tensile test specimens. The flexural beams (300 mm × 100 mm × 100 mm) were cast according to the requirements of AS 1012.8.2 [7] for making and curing the test specimens. All concrete samples were cast using ready mixed concrete, left in their moulds for 24 h under laboratory conditions, then transferred to a lime saturated bath until the time of testing. The samples were subjected to constant temperatures of 23 ± 2 °C in accordance with AS 1012 [8].

The concrete used is based on a common post-tension mix designed for winter conditions, with a characteristic compressive strength at 28 days \( f'_{\text{ci}} \) of 32 MPa. The concrete mix details, which were originally presented in Sofi et al. [9], are presented in Table 1.

### Table 1
Concrete mix design.

<table>
<thead>
<tr>
<th>Mix ingredients</th>
<th>Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement content</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Fly ash content</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Total cementitious</td>
<td>kg/m³</td>
</tr>
<tr>
<td>W/C ratio</td>
<td></td>
</tr>
<tr>
<td>Water reducer</td>
<td>ml/m³</td>
</tr>
<tr>
<td>Accelerator</td>
<td>ml/m³</td>
</tr>
<tr>
<td>Air entraining agent</td>
<td>ml/m³</td>
</tr>
<tr>
<td>20 mm aggregate</td>
<td>kg/m³</td>
</tr>
<tr>
<td>14 mm aggregate</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Washed concrete sand</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Slump</td>
<td>mm</td>
</tr>
</tbody>
</table>
The test was performed in accordance with AS 1012.9\textsuperscript{[10]} for compressive strength, AS 1012.10\textsuperscript{[11]} for splitting tensile strength, AS 1012.11\textsuperscript{[12]} for flexural tensile strength, and AS 1012.17\textsuperscript{[13]} for modulus of elasticity and Poisson’s ratio. It is noted that the above Australian Standard methods of testing is consistent with the corresponding ASTM standards\textsuperscript{[14–16]}. Results for 2, 3, 4, 7 and 28 days are presented in Table 2.

### 3.2. Temperature development

In addition to testing of the concrete specimens outlined in Section 3.1, a block of concrete was made and cured under laboratory conditions to monitor the temperature development in the concrete. The block of concrete was cast in a mould constructed using 16 mm black paint coated plywood. The sides and joints of the mould were sealed using a transparent sealant to prevent bleeding effects and to control moisture evaporation. Thermocouples were placed at different depths of the block. The concrete block dimensions and locations of thermocouples are presented schematically in Fig. 2. One side of the block (indicated with a letter “A” in Fig. 2) was exposed to air.

The concrete block was cured under laboratory conditions with a constant temperature of 20 ± 3°C, RH = 70%. The concrete temperature development was monitored over time using three channels at different depths as shown in Fig. 2. The measured temperature profiles are presented in Fig. 3. These temperature profiles will be used for validation of the finite element model presented in Section 4. Only temperature results for Channel 3 are presented in the subsequent sections as its location is the furthest from the surface and it corresponds to an integration point for one of the elements at which the temperature is calculated.

### 4. Modelling early age thermal development in concrete

The temperature of a concrete structural member is influenced by factors such as the fresh concrete temperature (initial temperature), heat of hydration (which depends on the type and amount of the binder), ambient conditions, geometrical properties of the member, and the location and properties of formwork and adjacent structural members that can provide thermal confinement to the hydrating concrete. In this study, only a single concrete block is considered ignoring the thermal confining effects of adjacent structural members.

A finite element model (FEM) simulating the hydration reaction of concrete and temperature development of a concrete block is presented in this section. The temperature model is validated by comparison with the experimental data. The FEM representing the hydration of concrete and temperature development will then form the basis for description of early age effects on mechanically loaded specimens.

#### 4.1. Temperature development in hydrating concrete: theoretical background

The temperature development in a hardening concrete due to hydration may be described by the Fourier differential equation for heat conduction that is for a homogeneous and isotropic body:

\[
\text{div}(q) + \rho c \frac{\partial T}{\partial t} = Q
\]

\[
q = -K \nabla T
\]

where \(q\) is the heat flux, \(T\) is the temperature, \(\frac{\partial T}{\partial t}\) is the rate of temperature, \(\nabla T\) is the temperature gradient, \(\rho\) is the mass density, \(c\) is the specific heat capacity, and \(K\) is the thermal conductivity.

### Table 2

<table>
<thead>
<tr>
<th>Days</th>
<th>Compressive strength (MPa)</th>
<th>Splitting tensile strength (MPa)</th>
<th>Flexural tensile strength (MPa)</th>
<th>Modulus of elasticity (MPa)</th>
<th>Poisson’s ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>27.0</td>
<td>2.7</td>
<td>3.7</td>
<td>15986</td>
<td>0.17</td>
</tr>
<tr>
<td>27.5</td>
<td>2.6</td>
<td>3.9</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
</tr>
<tr>
<td>28.0</td>
<td>2.8</td>
<td>N.A.</td>
<td>15484</td>
<td>0.21</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>30.0</td>
<td>3.1</td>
<td>4.1</td>
<td>18089</td>
<td>0.20</td>
</tr>
<tr>
<td>31.5</td>
<td>3.0</td>
<td>4.0</td>
<td>17570</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>33.5</td>
<td>3.1</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>37.0</td>
<td>3.0</td>
<td>4.3</td>
<td>17921</td>
<td>0.15</td>
</tr>
<tr>
<td>36.5</td>
<td>3.6</td>
<td>4.3</td>
<td>15689</td>
<td>0.17</td>
<td></td>
</tr>
<tr>
<td>36.0</td>
<td>3.6</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>42.0</td>
<td>3.7</td>
<td>4.8</td>
<td>21458</td>
<td>0.18</td>
</tr>
<tr>
<td>41.5</td>
<td>3.6</td>
<td>4.5</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
</tr>
<tr>
<td>43.0</td>
<td>3.3</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>53.0</td>
<td>3.3</td>
<td>3.3</td>
<td>N.A.</td>
<td></td>
</tr>
<tr>
<td>50.0</td>
<td>3.4</td>
<td>5.3</td>
<td>N.A.</td>
<td>N.A.</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 2. Location of thermocouples to measure temperature development with respect to surface (A).

Fig. 3. Temperature development in Channels 1–3.
the specific heat, \( Q \) is the rate of internal heat generation per unit volume and \( K \) is the conduction coefficient.

The temperature development in a newly cast member is determined by the balance between the development of heat due to hydration and the exchange of heat with the surroundings. In finite element analysis, the coefficient of heat transfer represents the heat transfer between a surface and the environment. It depends on the type of material and thermal insulating properties used to make the formwork. In addition, it depends on environmental effects such as ambient temperature, wind velocity and solar radiation. Studies by Van Breugel [17] report that, the effects of wind and solar radiation on temperature developing young concrete are significant and must be taken into account in numerical simulations. For practical reasons, however, the effects of solar radiation and wind speed are often not measured on the construction site. For the common range of temperatures, the solar radiation is usually accounted for together with convection, through a single convection–radiation coefficient [18].

Convection refers to the heat transfer that occurs between the concrete surface and a moving fluid (usually air) when a temperature gradient is installed between both materials. According to Newton’s cooling law, the convective heat transfer can be expressed as shown in Eq. (4) [18]:

\[
Q \cdot n = h(T_f - T)
\]

where \( q \) is the convective heat flux per unit area, \( n \) is the vector pointing outwards normal to the boundary, \( h \) is the heat transfer coefficient, \( T \) is the boundary temperature and \( T_f \) is the temperature of the surroundings.

4.2. Description of the model

The finite element model simulation was undertaken using DIA-NA [19]. The simulation was based on the thermal properties and boundary conditions of the concrete specimen cured under laboratory conditions. The schematic representation of the model is shown in Fig. 2. The FEM has interface elements for potential flow analysis at all boundaries, simulating the heat flow to the outside environment. The BT3HT and CTE30 elements are selected as the interface and solid elements, respectively. Both temperature development and degree of reaction are parts of output from DIA-NA based on thermal analysis of the FEM.

4.2.1. Thermal properties of early age concrete

Specific heat capacity is an important parameter in modelling the temperature gradients in early age concrete. It refers to the capacity of concrete to store heat. The specific heat capacity of concrete during hydration has been the focus of extensive studies [20–25]. The values of the specific heat capacity of early age concrete from the studies range between 0.85 and 1.92 kJ/kg K (2.04 × 10^8 to 4.61 × 10^6 J/m^3 K).

Thermal conductivity is another important parameter when evaluating the temperature gradients in early age concrete. It measures the ability of the material to conduct heat and is defined as the ratio of the flux of heat to temperature gradient [26]. Lofkvist [20] and Byfors [21] reported that there were no significant differences between the conductivity of early age concrete compared with matured concrete. More recent studies [23,27,28] reported an increasing trend of conductivity during the early ages. The studies generally suggested values of thermal conductivity which range from 1.2 to 3.5 W/mK.

4.2.2. Heat transfer through concrete and the external environment

Heat diffusion occurs through boundary mediums such as formwork. In the experimental work, plywood was used as the formwork for curing the concrete. Fig. 4 presents a simple case where concrete is hyrdrating in a plywood box of 16 mm thickness that thermally confines the concrete volume from five sides. The top surface is exposed to air (laboratory temperature).

Thermal conductivity \( (k_{\text{plywood}}) \) varies with timber species; however, an average value of \( k_{\text{plywood}} = 0.1154 \text{ W/(m}^\circ\text{C)} \) is reported in supplier’s datasheet [29]. Another source reports that the thermal conductivity of plywood is largely dependent on its density and is likely to be in the range of 0.09–0.24 W/mK [30].

Based on a nominal 16 mm thickness of plywood, the convection coefficient \( (K) \) through plywood can easily be calculated by dividing the thermal conductivity of the plywood \( (k_{\text{plywood}}) \) by the thickness. The \( K \) values were found to be in the range between 5.6 and 15.0 W/m^2 K.

Thermal conduction between the hydrating concrete surface and the air is modelled by using ‘skin’ type elements (BT3HT) together with a heat transfer coefficient \( (h) \). Different values are reported in the literature. Between the external boundaries and the environment, a heat transfer coefficient equal to 10 W/(m^2 K) is considered reproducing stagnant air condition [18].

As quoted by Faria et al. [18], based on the wind speed \( v \) (m/s), the heat transfer coefficient may be estimated from the following equation [31]:

\[
h_v = \begin{cases} 
5.6 + 3.95v, & v \leq 5 \text{ m/s} \\
7.6v^{0.78}, & v > 5 \text{ m/s} 
\end{cases}
\]

The program DIANA was used to solve the differential Eq. (2) by adopting Galerkin weighting procedure. This weighting procedure was shown to be unconditionally stable [32].

4.2.3. Adiabatic temperature and degree of reaction

To model chemical reactions like cement hydration, the finite element program simulates heat generation based on the degree of reaction. To determine heat production, the characteristic values of the conductivity, capacitance and temperature development over time under adiabatic hydration conditions are required. The program DIANA will then generate the degree of reaction–dependent heat production \( (q_r(T)) \) from this input.

A computer-based program called HYSMOSTRUC was used to generate the adiabatic temperature and degree of reaction for the concrete mix. The program takes into account the effects of physical interactions between hydrating cement particles on the rate of hydration of individual cement particles. It makes allowance for the effects of chemical composition, particle size distribution of the cement, w/c ratio, composition of concrete mix ingredients other than the cement and the temperature regime [28]. The adiabatic temperature representing the concrete mix is presented in Fig. 5.

4.3. Validation of the model

In order to accurately predict the concrete hydration and temperature development, the FEM is validated by comparing the temperature profiles from thermal analysis with the temperature profile recorded during the experiment. The recorded temperature profile is presented in Fig. 6.

As the control samples were cured under laboratory conditions, the heat transfer coefficient was calculated using Eq. (5) to represent the latent heat transfer to the controlled environment associated with laboratory conditions. The effects of solar radiation were not included. The air flow was assumed to be 0.5 m/s in accordance with Faria et al. [18]. Both concrete thermal conductivity and heat specific capacity were obtained from the literature summarised in Section 4.2.1. The activation energy was obtained from Eq. (6) based on studies by Chengju [33]:
The average temperature \( T \) was assumed to be 20 \( ^\circ\text{C} \) in this project.

The thermal properties of concrete and the boundary conditions in the numerical model are presented in Table 3.

The input ambient temperatures were obtained from the experimental data shown in Fig. 6. Fig. 7 presents a comparison between the recorded temperature profiles and those from model prediction at the level of Channel 3 in the concrete block. The temperature profiles are presented only for Channel 3 as its location is the furthest from the surface and it corresponds to an integration point for one of the elements at which the temperature is calculated. The model was shown to be in good agreement with the experimental results. Temperature profiles recorded from the experiment and the analyses were found to be very similar, with a maximum temperature difference of about 2 \( ^\circ\text{C} \).

### 5. Results and discussions

This section discusses experimental and finite element modelling results. The experimental data presented in Section 3 were used to produce predictions of strength based on the degree of

\[
\frac{E_a}{R(K)} = \begin{cases} 
4000 & T \geq 20 \\
4000 + 175(20 - T) & T < 20
\end{cases} 
\]  

(6)

The thermal properties of concrete and the boundary conditions in the numerical model are presented in Table 3.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete thermal conductivity (W/m ( ^\circ\text{C} ))</td>
<td>2</td>
</tr>
<tr>
<td>Concrete volumetric heat specific capacity (J/m(^3) ( ^\circ\text{C} ))</td>
<td>(4.6 \times 10^6)</td>
</tr>
<tr>
<td>Convection-radiation coefficient between concrete and air (W/m(^2) ( ^\circ\text{C} ))</td>
<td>7.5</td>
</tr>
<tr>
<td>Equivalent convection-radiation coefficient between concrete and formwork (W/m(^2) ( ^\circ\text{C} ))</td>
<td>6</td>
</tr>
<tr>
<td>Arrhenius constant ( \frac{E_a}{R} )</td>
<td>4000</td>
</tr>
</tbody>
</table>
reaction (Section 5.1). Section 5.2 presents the outcome of the parametric studies investigating the effects of ambient temperature and time of concrete pouring on the degree of reaction and hence the strength development of concrete.

5.1. Degree of reaction and strength development

Experimental results of the concrete specimens tested at ages 2, 3, 4, 7 and 28-days following the standard method of testing at the curing temperature 23 °C, were presented in Section 3. The mechanical properties of early age concrete include compressive, tensile and flexural strength development, as well as aging elasticity (modulus of elasticity and Poisson’s ratio). In this section, the experimental results were used to establish the degree of reaction-based equations describing various properties of the current mix, based on the models presented by de Schutter and Taerwe [34].

Under adiabatic conditions, the concrete specimen is completely heated. Hence, the heat evolution rate of concrete under adiabatic conditions in terms of time is proportional to the rate of temperature rise. Under isothermal and non-isothermal conditions, the heat evolution rate of concrete is no longer in linear proportion to the rate of temperature rise. Hydration models have been proposed to model the rate of heat development of concrete under these conditions [35–39]. However, these models still rely on experimental data to define their modelling parameters.

In this study, finite element analysis program was used to calculate the temperature development and the degree of reaction of concrete cured under isothermal and non-isothermal conditions. The hydration process of cement in DIANA is described by a variable \( r \), the degree of reaction, which is equal to the cumulative heat production at time \( t \) divided by the cumulative heat of hydration at a reference time of 28 days, as shown below:

\[
r = \frac{\int_0^t q(r, \tau) d\tau}{\int_0^t q(r, \tau) d\tau}
\]

The quantity of produced heat is a function of temperature history. The momentary heat production rate \( q \) can be defined as:

![Fig. 9. Mechanical properties based on degree of reaction.](image)

![Fig. 10. Temperature measurements from field investigation [9].](image)

---

### Table 4

<table>
<thead>
<tr>
<th>Mechanical properties</th>
<th>Values of parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength</td>
<td>( a = 1.00, r_o = 0.160 )</td>
</tr>
<tr>
<td>Splitting tensile strength</td>
<td>( c = 0.40, r_o = 0.035 )</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>( b = 0.12, r_o = 0.623 )</td>
</tr>
<tr>
<td>Flexural tensile strength</td>
<td>( d = 0.15, r_o = 0.586 )</td>
</tr>
</tbody>
</table>

---
where \( r \) is the degree of reaction, \( T \) is the temperature in °C, \( a_{ad} \) is the maximum value of the heat production rate, \( q_r \) is the degree of reaction-dependent heat production under adiabatic conditions and can be defined by the following equation:

\[
q(T) = C_p \frac{dT}{dt} \tag{9}
\]

where \( C_p \) is the specific heat of concrete (in kJ/kg °C), \( q_T \) is the temperature-dependent heat production:

\[
q_T(T) = \frac{C_p}{C_0} \frac{dT}{dt} \tag{10}
\]

where \( C_0 \) is the constant of Arrhenius, which is dependent on temperature and/or degree of reaction.

The program DIANA was used to simulate the hydration for the concrete block presented in Fig. 2. The degree of reaction for the specimens cured at 23 °C is presented in Fig. 8.

The mechanical properties obtained from the experimental program presented in Section 3.1 such as compressive strength \( f_c \), splitting tensile strength \( f_{sp}s \) and modulus of elasticity \( E_{co} \) were used to establish the predictive equations based on the degree of reaction. The normalised experimental results are plotted in Fig. 9.

Regression analyses were undertaken to curve fit the experimental results to degree of reaction-based formulations. The predictive equations based on the degree of reaction are defined by Eqs. (11)–(14). Parameters obtained from the regression analyses are summarised in Table 4.

\[
f_c(r) = \frac{r - r_0}{1 - r_0} \tag{11}
\]

where \( f_c(r) \) is the compressive strength at degree of reaction \( r \), \( f_c(r = 1) \) is the compressive strength at degree of reaction \( r = 1 \), and \( a \) and \( r_0 \) are parameters.

\[
f_{sp}(r) = \frac{r - r_0}{1 - r_0} \tag{12}
\]

where \( f_{sp}(r) \) is the splitting tensile strength at degree of reaction \( r \), \( f_{sp}(r = 1) \) is the splitting tensile strength at degree of reaction \( r = 1 \), and \( c \) and \( r_0 \) are parameters.

\[
E_{co}(r) = \frac{r - r_0}{1 - r_0} \tag{13}
\]

where \( E_{co}(r) \) is the modulus of elasticity at degree reaction \( r \), \( E_{co}(r = 1) \) is the modulus of elasticity at \( r = 1 \), and \( b \) and \( r_0 \) are parameters.

\[
f_{f}(r) = \frac{r - r_0}{1 - r_0} \tag{14}
\]

where \( f_{f}(r) \) is the flexural tensile strength at degree of reaction \( r \), \( f_{f}(r = 1) \) is the flexural tensile strength at degree of reaction \( r = 1 \), and \( d \) and \( r_0 \) are parameters.

5.2. Influence of ambient temperature on temperature history and degree of reaction

In this section, parametric studies of the effects of initial temperature and the variation of ambient temperature at early ages on the temperature development and degree of reaction of concrete are presented. In particular, the effects of curing in different temperature ranges and concrete pouring during different periods

Fig. 11. Variation in ambient temperature ranges – The Sentinel apartments, Melbourne.

Fig. 12. Variation in ambient temperature ranges – Austin Hospital, Melbourne.

Fig. 13. Different pouring times during the day – The Sentinel apartments, Melbourne.

Fig. 14. Different pouring times during the day – Austin Hospital, Melbourne.

Fig. 15. Effects of initial temperature on adiabatic temperature.
of the day are discussed. As the purpose of the study is to investigate the effects of ambient temperature on bearing strength of concrete at early age, only the effects of temperature on the compressive strength development are presented in this section.

Parametric studies were conducted using the FEM presented in Section 4. The ambient temperature profiles were based on the temperature recorded in the field investigation conducted by the authors [9]. The temperature profile in the last cycle was repeated to achieve ambient temperatures with a period of seven days, as presented in Fig. 10.

To investigate the effects of curing temperature on the thermal development in the concrete block (Fig. 2), the temperature profiles presented in Fig. 10 are shifted vertically in order to obtain variable ranges of ambient temperature profiles. Since the mix investigated in this paper is especially designed for winter conditions in Australia, the range of the temperature profiles considered...
was selected based on the range of typical temperatures during the winter time. It is noted that the results of this study are only applicable to countries with similar climates. The curing temperature ranges considered in the parametric studies are presented in Figs. 11 and 12.

The time of pouring can affect the initial temperature of the concrete significantly and could potentially affect the temperature development in the concrete. To investigate the effects of time of pouring on the temperature development in the concrete, the

Fig. 18. Strength development relationships, effects of time of pouring, The Sentinel apartments, Melbourne.

Fig. 19. Strength development relationships, effects of time of pouring, Austin Hospital, Melbourne.

Fig. 20. Slab anchorage showing gross bearing plate and distribution area.
ambient temperature profiles presented in Fig. 10 were shifted in the horizontal direction to produce initial temperatures which range from 10 to 23 °C. The temperature profiles considered in the parametric studies are presented in Figs. 13 and 14. The temperature profiles starting with the peak (initial temperatures 20 and 23 °C) represent pouring of the concrete in the afternoon, whilst those starting with the dip (initial temperatures 10 and 15 °C) represent pouring of the concrete in the morning.

The adiabatic temperatures of the concrete are presented in Fig. 15. These adiabatic temperatures were significantly affected by the concrete mix and the initial temperature. It was shown that the initial slope of the adiabatic temperature curves increases with the increasing initial temperature, indicating accelerated hydration reaction. As expected, the peak adiabatic temperature also increases with the increasing initial temperature.

The temperature development, degree of reaction and compressive strength development are presented in Figs. 16–19 for the ambient temperatures presented in Figs. 11–14, respectively. As the paper focuses on the behaviour of concrete immediately ahead of the anchorage zone, only the compressive strength results are presented in this section. The compressive strength normalised to the characteristic values (the compressive strength values when the degree of reaction equals 1.0) were calculated using Eq. (11).

The effects of ambient temperature on temperature and strength development of concrete are shown in Figs. 16 and 17. As expected, the temperature in concrete increases with the increase in ambient temperature. Concrete cured at high temperature is shown to hydrate much faster than concrete cured at lower temperature. As a consequence, the concrete develops its strength much faster when it is subjected to high ambient temperatures.

The effects of time of pouring on the temperature and strength development of concrete are shown in Figs. 18 and 19. The results from the parametric studies indicate that pouring in the afternoon (initial temperature 20 and 23 °C) would result in a higher degree of reaction and, consequently, a higher strength development. However, the peak temperature could also occur much earlier (at about 10–15 h), and consequently could induce premature cracks on the hydrating concrete.

6. Comparison between allowable and imposed bearing stress

Post-tensioning cables are commonly loaded to 25% of the jacking load when concrete is at 1-day age, and the remaining load is applied when concrete age is between 4 and 7-days age. The required compressive strength according to Post-Tensioning Institute of Australia (PTIA) at the time of loading is 6 MPa at 1-day and 22 MPa at 4–7-days [1] while ACI 318 [2] recommends a higher compressive strength of 27 MPa at final pre-stressing. Results from the parametric studies show that pouring in the afternoon (initial temperature 20 and 23 °C) would result in a higher degree of reaction and, consequently, a higher strength development. However, the peak temperature could also occur much earlier (at about 10–15 h), and consequently could induce premature cracks on the hydrating concrete.
The concrete strength values when the degree of reaction equal to 1.0 were calculated using Eq. (11). The compressive strength values from the parametric studies are shown to exceed the required strength of 6 MPa and 22 MPa set by PTIA for all cases.

The adequacy of slabs subjected to bearing stress can also be assessed by comparing the average bearing stresses with the allowable bearing stress. This is illustrated by an example of a slab anchorage with dimensions shown in Fig. 20. The compressive force imposed by the anchorage plates were determined based on common tendon unit type supplied by VSL (Fig. 21) and a breaking load of 184 kN for a 12.7 mm wire strand. The strands can be loaded up to 80% of the breaking load in accordance with AS 3600 [40] and ACI 318 [2]. It is shown in Table 5 that the average bearing stress assuming standard plate size at the time of loading is about 11 MPa at 1-day and 43 MPa at 4–7-days, respectively.

The allowable bearing stress assuming minimum local zone reinforcement (\(\rho_s > 2\%\)) is defined by Eq. (2) for basic bearing plates. The gross bearing plate area adopted was based on standard dimensions presented in Fig. 21, for tendon unit type H and strand type 12.7 mm 5–5. The distribution area was defined assuming a slab thickness of 250 mm and spacing of 1400 mm between the anchorage plates. The distribution area \((A_g)\) and the gross bearing plate area \((A_g^c)\) for the specimen are shown in Fig. 20.

The allowable bearing stresses \((f_{p,c})\) were calculated based on the lowest and highest values of compressive strength presented in Figs. 16–19 at 1-day and 4-days. The lowest compressive strength values are 8.5 MPa at 1-day age and 30 MPa at 4-day age concrete (referred to as the lower bound case in Table 6). The highest compressive values are 30 MPa at 1-day age and 40 MPa at 4-day age concrete (referred to as the upper bound case in Table 6). The results are presented in Table 6, along with the allowable bearing stresses based on the required compressive strength values according to Post-Tensioning Institute of Australia (PTIA) requirements [1]. It is shown that the allowable bearing stress \((f_{p,c})\) in Table 6 generally exceeds the average bearing stress at 1-day (Table 5). Hence, the concrete slab can be deemed safe when subjected to post-tensioning load at 1-day age. However, the values of allowable bearing stress \((f_{p,c})\) in Table 6 from the parametric studies were generally found to be exceeded by the average bearing stress at 4-days age (Table 5). The compressive strength had not reached the required strength at 4-days age for the temperature ranges investigated. It is noted that, the required strengths as recommended by PTIA have been exceeded by the bearing stresses at both 1-day and 4-days. Thus, the loading requirements in accordance to PTIA can be deemed unsafe.

7. Conclusions

This paper focuses on the effects of ambient temperature and the adequacy of concrete immediately ahead of the anchor plate where anchorage bearing stresses are prevalent. Expressions to describe the mechanical properties of early age concretes, including compressive, tensile, and flexural strength development, modulus of elasticity and Poisson’s ratio have been proposed as a function of degree of reaction. The expressions are based on results from experimental work.

Parametric studies were undertaken using a finite element model (FEM) to investigate the effects of initial temperature and the variation of ambient temperature at early ages on the strength of concrete. The FEM was based on actual measurements. Temperature profiles recorded during the field investigations were shifted vertically and horizontally to simulate variation in the range of ambient temperature and the time of pouring, respectively. The results of the parametric studies were presented in terms of temperature development, degree of reaction and compressive strength development of the concrete at early ages. It is shown that the temperature in concrete increases with the increase in ambient temperature. As a result, the concrete develops its strength much faster when it is subject to high ambient temperature. Similarly, pouring in the afternoon, when the daily temperature is high, could result in a higher degree of reaction and consequently higher compressive strength of concrete.

Based on the illustrative example presented, although the concrete at local zone is adequate in resisting the compressive stress at initial stressing at 1-day, the values of allowable bearing stress can be exceeded at the final stressing stage. Inadequate compressive strength at early age, which can be significantly affected by ambient temperature, can contribute to anchorage failures.

References