# DESIGN OF SKIN REINFORCEMENT FOR CONCRETE PILE CAPS

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Abstract

**The large pile caps of buildings and bridges may get superficial cracks already in the early hours after concreting. Due to the large volume of concrete, the temperature inside the pile cap may reach very high values, as a result of the heat of hydration of cement. Because of the strong temperature gradients, the surface of the pile cap is tensioned and may crack. The employment of skin reinforcement does not avoid the cracking of concrete. However, this reinforcement may reduce the crack width, providing the onset of a large number of small cracks, instead of a single crack with large opening. The object of this work is to address this issue by analyzing the main variables involved and suggest a design methodology for the calculation of skin reinforcement of concrete pile caps.**

Streszczenie

**W dużych oczepach fundametnów palowych budynków i mostów już po kilku godzinach od zabetonowania mogą pojawić się rysy powierzchniowe. Z powodu znacznej objętości betonu, w wyniku ciepła wyprodukowanego w procesie hydratacji cementu, temperatura wewnątrz takiego oczepu może osiągnąć znaczą wartość. Na skutek znacznych gradientów temperatury powierzchnia oczepu poddawana jest rozciąganiu i może ulec zarysowaniu. Zastosowanie zbrojenia przypowierzchniowego nie pozwala uniknąć zarysowania betonu. Zbrojenie to może jednak ograniczyć szerokość rozwarcia rys poprzez zapewnienie powstania licznych, ale małych rys zamiast znacznej pojedynczej rysy. Przedmiotem tej pracy jest odniesienie się do tego zagadnienia poprzez analizę głównych czynników oraz propozycję metodologii obliczania zbrojenia przypowierzchniowego w betonowych oczepach pali.**

Keywords: **Concrete; Cracking; Heat transfer; Pile caps; Reinforcement; Thermal stresses.**

## 1. INTRODUCTION

Cement hydration reactions generate heat in concrete and cause a rise in temperature in the core of the structural element, which depends on, among other factors, the size of the element. The higher the structural element, the higher the temperature reached in its core.

The heat is transferred by conduction from the core to the surface of the element, where it is dissipated to the environment. Because of this heat transfer, temperature gradients rise, which may introduce tension and compression stresses in concrete.

Once the surface cools more quickly, it tends to shorten, while the concrete inside the element is in the heating phase. Thus, the core of the member introduces tensile efforts on superficial layers of concrete. These stresses may cause cracks in the concrete, which may impair its durability.

This is a typical problem of internal or intrinsic imposed deformations and is independent of external loads applied to the structure. Because of the temperature differences between the various points of the structural element, the imposed deformation (thermal deformation) is restrained, which causes compression stresses in the core and tensile stresses on the surfaces of the element. In slender structures, the temperature gradients are small and the tensile stresses are not sufficient to produce cracks in concrete. However, in large elements these surface cracks may be inevitable.

The elements of the framed structure of buildings are usually slender, so there is no need for concern with these cracks. However, pile caps may have sufficiently high dimensions so that this problem is featured in the structural design. In order to minimize this problem, it may be necessary to adopt a set of measures, such as reducing the cement content, using pozzolanic cements, concrete placement in layers of small height, precooling and post-cooling of concrete, protection of concrete to avoid very rapid cooling of the surfaces and prolonged cure to reduce shrinkage, among others.

The employment of skin reinforcement does not avoid the cracking of concrete. However, this reinforcement may reduce the crack width, providing the onset of a large number of small cracks, instead of a single crack with large opening. These skin reinforcement mesh to be arranged on all sides of the pile cap.

The problem of concrete cracking resulting from imposed deformations has been extensively studied for one-dimensional elements and for walls [1÷6].

For three-dimensional structures of large volume, such as dams, there are countless studies on the subject, with approaches aimed at concrete technology and construction techniques, as the concrete placement in layers of small height, precooling and post-cooling the concrete.

However, there is a lack of research to quantify the skin reinforcement of pile caps of buildings and bridges. For these structural elements, skin reinforcement from empirical criteria is adopted, based on experience, but without any calculation methodology. This is mainly due to the omission of concrete design codes in this topic. According to the Eurocode EC2 [7], for example, the top and the lateral surfaces of pile caps may be unreinforced, provided that there is no risk of cracking of concrete, without presenting any criterion for such check. Other design codes, such as CEB-FIP Model Code [8], ACI 318 [9] and FIB Model Code [10] do not even comment on the subject. As a consequence of this lack of normative guidance, it is usual to find discrepant solutions, from the total absence of skin reinforcement to the employment of visibly excessive skin reinforcement.

The aim of this paper is to study this problem and propose a methodology to calculate the skin reinforcement of large pile caps. The thermal analysis is accomplished with the finite element method [11]. The stresses in concrete are obtained through a simplified analysis of the critical section of the pile cap.

The study is limited to two-dimensional heat transfer analysis. However, this analysis may be used with reasonable approximation for pile caps, by defining a width equivalent. This is possible because the heat transfer takes place mainly towards the centre to the top of the pile cap.

Due to the presence of the forms on the sides and bottom of the pile cap, which being made of wood provide thermal insulation, the primary heat flow occurs toward the top of the pile cap. For a prismatic pile cap with height $H$ and dimensions $A$ and $B$ in plan, the area of the upper face is $AB$. A pile cap of same height, but with circular plant of diameter $L$, has its upper face with area $\pi L^2/4$ . To this circular pile cap, the problem is axisymmetric and may be analyzed for a rectangle of width $L$ and height $H$. Equaling these areas of the two pile caps, it is obtained the equivalent width:

$$L = \sqrt{\frac{4AB}{\pi}} \tag{1}$$

## 2. ANALYSIS OF HEAT TRANSFER

The problem of two-dimensional heat transfer in a material with constant thermal properties, is governed by the differential equation:

$$k_x \frac{\partial^2 T}{\partial x^2} + k_y \frac{\partial^2 T}{\partial y^2} - c\rho \frac{\partial T}{\partial t} + \dot{q}_g = 0 \tag{2}$$

where $k_x$ and $k_y$ are the thermal conductivity in x and y directions, respectively; $c$ is the specific heat; $\rho$ is the density; $T$ is the temperature; $\dot{q}_g$ is the rate of heat generation; $t$ is time. For concrete, it is admitted the isotropy, so that $k=k_x=k_y$ .

This differential equation may be solved with the use of the finite element method (FEM) and an integration algorithm step by step. Employing the standard procedure of the finite element method, it is possible to determine temperatures in each point of the concrete pile cap [12,13].

The boundary conditions of the thermal problem are defined in Fig. 1.

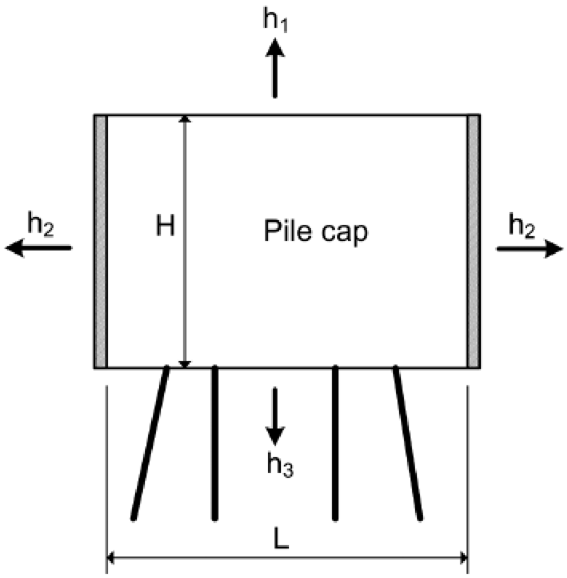


**Figure 1.**
**Boundary conditions of the heat transfer problem**

The boundary conditions are specified in all faces of the pile cap using the coefficients of heat transfer by convection $h_1$, $h_2$ and $h_3$. These coefficients may vary within a relatively wide range, depending on the wind speed, of the material used as forms and of the thermal resistance of the rock or soil at the bottom of the pile cap.

The coefficient of heat transfer has a significant influence on temperature at near the surface of members, and also on the temperature rise in the interior when the thickness of the member is relatively thin. To a wind velocity of 2-3 m/s, the coefficient of heat transfer is between 12-14 W/m$^2$°C. This coefficient increases at a rate of 2.3 to 4.6 W/m$^2$°C per m/s of wind velocity [14].

For the top face, it is considered $h_1 = 13.5$ W/m$^2$°C as being the mean heat transfer coefficient to air. For the lateral faces, the equivalent transfer coefficient, $h_2$, takes into account the insulating effect of forms. This coefficient is obtained by means of the equation:

$$\frac{1}{h_2} = \frac{1}{h_1} + \frac{t_m}{k_m} \tag{3}$$

where $t_m$ is the thickness of the forms and $k_m$ is the thermal conductivity of its material [15].

Considering, for example, wooden forms with $t_m = 18$ mm and $k_m = 0.14$ W/m°C, it results $h_2 = 4.93$ W/m$^2$°C for the lateral faces. The relationship $\delta = h_2/h_1$ has the value $\delta = 0.365$.

After the forms are removed, the value of the coefficient is equal: $h_2 = 13.5$ W/m$^2$°C for the lateral faces.

In this work, it is adopted $h_3 = h_2$ during the entire analysis. Moreover, the removal of forms to obtain the greatest temperature gradient toward the top of the pile cap is not considered.

## 3. THERMAL PROPERTIES OF CONCRETE

According to Eurocode EC2 [7], for temperature of 20°C, the thermal conductivity of concrete varies between $k_{\inf} = 1.33$ W/m°C and $k_{\sup} = 1.95$ W/m°C. The mean value is approximately $k = 1.65$ W/m°C. The specific heat of concrete may be considered equal to $c = 900$ J/kg°C, for temperatures between 20°C and 100°C. The mean density of usual concrete is $\rho = 2400$ kg/m$^3$.

Thermal properties of concrete are strongly influenced by their composition and by the environmental conditions. Thus, the thermal conductivity and the specific heat vary with the type and the cement content, type of aggregate, temperature and moisture content. These values recommended by EC2 may be used for dry concrete made with silicious and calcareous aggregates.

The heat of hydration $Q_h$ is the total amount of heat generated by the complete hydration of the cement. It depends on content and type of cement, as well as the temperature. The rate of hydration also depends on the type of cement. For normal hardening cements, the heat of hydration may reach $Q_h = 350$ kJ/kg at 28 days of age, or a higher value.

The function $T_a(t)$ , which represents the adiabatic temperature rise of concrete, varies with the type of cement, the type of aggregate and the water-cement ratio. In this work, the following expression is adopted:

$$T_a(t) = T_{a,\max}\left(1 - e^{-0.5t^{0.7}}\right) \tag{4}$$

with the age $t$ in days.

This theoretical curve is the best fit to a series of concrete mixtures used in Itaipu dam and Tucurui dam, both located in Brazil [16], where pozzolanic cement, sand as fine aggregate and basalt as coarse aggregate were used. The maximum temperature $T_{a,\max}$ may be obtained from the relationship:

$$T_{a,\max} = \frac{Q_{h\infty} M_c}{c\,\rho} = C_R M_c \qquad (5)$$

where $Q_{h\infty}$ is the total hydration heat per kg of cement, represents the cement content per m$^3$ of concrete and $C_R = Q_{h\infty}/(c\ \rho)$ is the coefficient of thermal efficiency, representing the maximum adiabatic temperature rise per kg of cement per m$^3$ of concrete.

The heat of hydration released until the age $t$ days is given by:

$$Q_h(t) = c\,\rho\,T_a(t) \qquad (6)$$

Finally, considering equations (4), (5) and (6), the rate of heat generation $\dot{q}_g = dQ_h(t)/dt$ may be obtained.

## 4. RESULTS OF THERMAL ANALYSIS

The finite element model was employed to analyze pile caps of width $L$ and height $H$, as shown in Fig. 1. The bidimensional domain $L-H$ is discretized in quadratic isoparametric finite elements of eight nodes. Using polynomial functions to interpolate the temperature $T$ and employing the Galerkin method, equation (2) is transformed into a system of linear differential equations in terms of nodal temperatures. This system is solved with an integration algorithm step by step to obtain the nodal temperatures at each time $t$ [11].

In all the examples, it is considered a concrete hydration heat $Q_{h\infty} = 400$ kJ/kg, which corresponds to the coefficient of thermal efficiency $C_R = 10.19$°C/(kg/m$^3$). The other concrete properties are $k = 1.65$ W/m°C, $c = 900$J/kg°C and $\rho = 2400$ kg/m$^3$. The heat transfer coefficients are $h_1 = 13.5$W/m$^2$°C and $h_2 = h_3 = 4.93$ W/m$^2$°C, with $\delta = h_2/h_1 = 0.365$. It is assumed that the placement temperature of concrete is equal to 25°C and the mean air temperature is equal to 20°C.

Fig. 2 shows the temperature variations at the center, at the top, and the temperature difference between these two locations for a pile cap of width $L = 1.4$ m and height $H = 0.7$ m. Cement content is 350 kg/m$^3$.

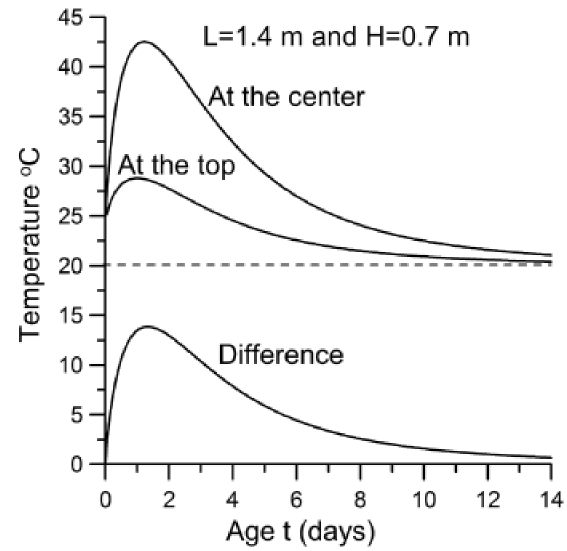


**Figure 2.**
**Temperature variations in concrete (pile cap height: H = 0.7 m)**

It is observed that the thermal equilibrium is reached about two weeks after the concrete placement. The peak temperature occurs at an age $t = 1.3$ day. The maximum temperature reached inside the pile cap is $T_{max} = 42.8$°C. Maximum temperature difference between the core and the top of the pile cap is $\Delta T = 13.8$°C.

Maximum temperatures depend on the size of the concrete pile cap, in addition to other factors involved in the thermal analysis. For the structural design, it is desirable to find an equation that allows to obtain the maximum temperature difference $\Delta T$ between the core and the surface of the pile cap. For this, it is desirable to correlate $\Delta T$ with an equivalent thickness of the pile cap.

The equivalent thickness $H_e$ may be defined as being the ratio between the area of the rectangle $LH$ and the perimeter where the heat is lost. In order to take into account the insulating effect of the forms and the thermal resistance imposed by soil, the equivalent thickness is defined as:

$$H_e = \frac{LH}{(1+\delta_3)L + 2\delta_2 H} \qquad (7)$$

where $\delta_2 = h_2/h_1$ and $\delta_3 = h_3/h_1$ are relationships between the coefficients of heat transfer.

In the numerical examples, it is considered $\delta = \delta_2 = \delta_3$ because $h_2 = h_3 = 4.93$ W/m$^2$°C.

If the insulation is not taken into account, $\delta = 1$ and

$H_e = LH/2(L + H)$. If the insulation is complete, $\delta = 0$ and $H_e = H$. In an actual situation, the equivalent thickness varies between these two limits.

In order to determine the correlation between $\Delta T$ and $H_e$ , they were analyzed pile caps of width $L$ ranging between 0.3 m to 8.0 m and height $H$ ranging between 0.3 m and 2.0 m. By varying these dimensions, various combinations of $L/H$ were obtained. In this numerical analysis, it was considered $\delta = 0.365$. Other data remained unchanged, varying only the cement content.

Based on the results obtained with the FEM, the following equation was found:

$$\Delta T = \frac{(4760 + 90M_c)}{1000} H_e - \frac{(1840 + 9.8M_c)}{1000} H_e^2 \quad (8)$$

where $M_c$ is the cement content in kg/m$^3$, $H_e$ is the equivalent thickness in meters and $\Delta T$ is given in °C.

Equation (8) was obtained from a two-dimensional analysis. It may be used for three-dimensional pile caps with a proper definition for the width $L$. For pile caps with circular base, it may be adopted $L$=$D$, where $D$ is the diameter of the base. For pile caps with a rectangular base, the equivalent width is given in equation (1).

If the heat of hydration of cement $Q_{h\infty}$ is different from 400 kJ/kg, it is possible to employ the equation (8) using the equivalent cement content $M_{ce} = (M_c Q_{h\infty})/400$.

The two-dimensional model for heat transfer analysis was compared with a three-dimensional model presented by Klemczak and Knoppik-Wróbel [17]. The results obtained were very similar, which confirms the validity of the two-dimensional analysis, considering the equivalent width given in equation (1).

## 5. STRESS ANALYSIS

The determination of stresses in concrete resulting from temperature variations, may be carried out using the finite element method, as shown in [12, 13]. In this case, the FEM is used to determine temperature increments and stress increments in each time interval.

However, for this particular problem it is possible to make a simplified analysis, considering only the vertical section passing through the center of the pile cap. Since the stresses depend directly on temperature gradients, it is possible to analyze only this vertical section, because it is the one that presents the greatest temperature gradient.

Fig. 3 shows the temperatures and strains on this vertical section.

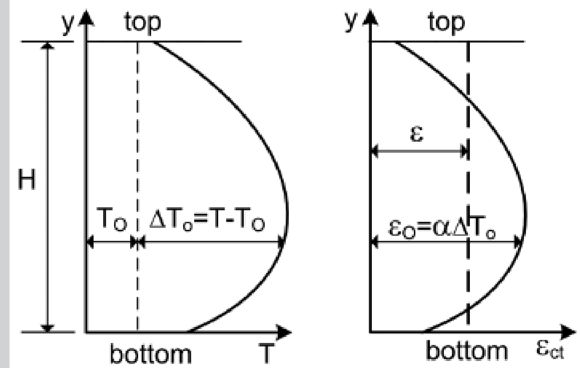


**Figure 3.**
**Temperatures and strains in the vertical section**

The variation of temperature in relation to placement temperature $T_o$, in a point at a distance $y$ from the bottom, is $\Delta T_o = T - T_o$. It is observed that $\Delta T_o = \Delta T_o(t, y)$ is a function of the age and of the distance $y$ to the bottom.

The free thermal strain in this point of coordinate $y$ is given by:

$$\varepsilon_o = \alpha \Delta T_o = \alpha (T - T_o) \quad (9)$$

where $\alpha = 10^{-5}$°C$^{-1}$ is the coefficient of thermal expansion for concrete.

Since the vertical section remains plane and vertical, actual strain $\varepsilon$ must be constant along the height. Due to this difference between the restrained strain $\varepsilon$ and the free strain $\varepsilon_o$, normal stresses $\sigma_c$ arise along the height of the section. These stresses depend on the difference $\Delta\varepsilon = \varepsilon - \varepsilon_o$ between the restrained strain and the free strain.

Stresses in concrete are obtained with the stress-strain diagrams shown in Fig. 4. The diagram for tensioned concrete is proposed by CEB-FIP Model Code [8] and also adopted in FIB Model Code [10]. This diagram takes into account the progressive micro-cracking which starts at a stress of about 90% of the tensile strength. The micro-crack grows and forms a discrete crack at stress close to the mean tensile strength $f_{ctm}$.

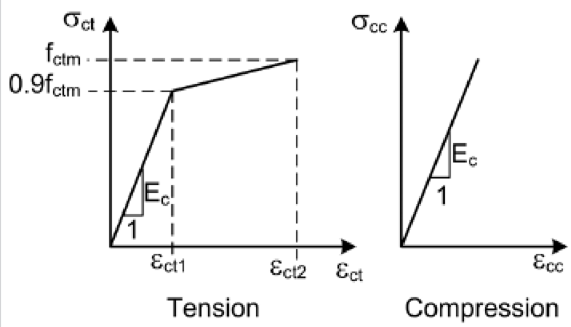


**Figure 4.**
**Stress-strain diagrams for plain concrete**

The strain $\varepsilon_{ct1} = 0.9f_{ctm}/E_c$ is evaluated with the mean tensile strength $f_{ctm}(t)$ and the tangent modulus $E_c(t)$ at an age $t$ days. The strain $\varepsilon_{ct2}$ is constant and equal to 0.00015. Favorable effects of creep are not considered.

Creep has a favorable effect by reducing stresses in concrete resulting from imposed deformations. Creep can be included through the model developed in [12-13], which considers the effects of concrete aging. Alternatively, one can employ the diagrams of Fig. 4, replacing the modulus $E_c$ for an effective modulus. However, creep was not included in this work to provide a solution on the side of safety. The autogenous shrinkage was not considered because it is of secondary importance compared with the thermal strain.

According to CEB-FIP Model Code [8], these properties of concrete at an age $t$ days are given by:

$$E_c(t) = [\beta_{cc}(t)]^{1/2} E_{c28} \tag{10}$$

$$f_{ctm}(t) = \beta_{cc}(t) f_{ctm28} \tag{11}$$

where $E_{c28}$ and $f_{ctm28}$ are the tangent modulus and mean tensile strength at the age of 28 days, which may be obtained from the compressive characteristic strength $f_{ck}$, according to relationships:

$$E_{c28} = 21500\left(\frac{f_{ck}+8}{10}\right)^{1/3} \text{, MPa} \tag{12}$$

$$f_{ctm28} = 1{,}40\left(\frac{f_{ck}}{10}\right)^{2/3} \text{, MPa} \tag{13}$$

with $f_{ck}$ given in MPa.

The aging function $\beta_{cc}(t)$ is given by:

$$\beta_{cc}(t) = \exp\left\{ s\left[1-\left(\frac{28}{t}\right)^{1/2}\right]\right\} \tag{14}$$

where $s$ takes into account the type of cement (CEB 1993).

In order to perform the structural analysis with very young concrete, it is necessary to establish a minimum age from which the mechanical properties of concrete are measurable. In general, it may be assumed that age as 12 hours, which corresponds to a degree of hydration of about 20% for the normal hardening cements [18]. Therefore, the stress analysis is performed only for $t > 0.5$ day.

If the restrained strain $\varepsilon$ is known along the height of the pile cap, the difference $\Delta\varepsilon = \varepsilon - \alpha(T - T_o)$ may be evaluated in each point situated at a distance from the bottom. Using the stress-strain diagrams of concrete, one obtains the stress $\sigma_c = \sigma_c(t, y)$. As the normal effort in this central section is null, one should have:

$$\int_0^H \sigma_c \, dy = 0 \tag{15}$$

Equation (15) allows us to determine the restrained strain $\varepsilon$ through an iterative process. A numeric integration is performed in each iteration. Then, it is estimated $\Delta\varepsilon = \varepsilon - \alpha(T - T_o)$, yielding the stresses $\sigma_c = \sigma_c(t, y)$ along the height of the pile cap, as illustrated in Fig. 5.

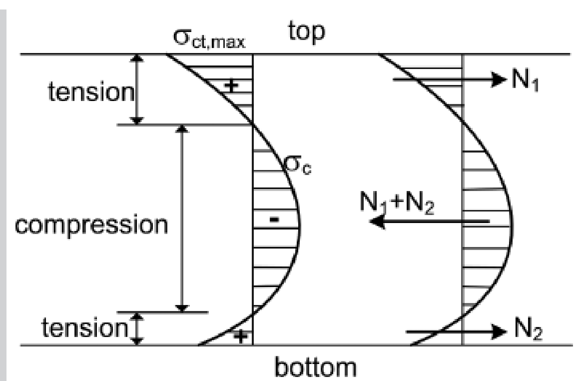


**Figure 5.**
**Normal stresses in the vertical section**

The layers near the top and bottom of the pile cap are under tension, while the central region is compressed. Tensile normal efforts along these faces are $N_1$ and $N_2$, and may be obtained by numerical integration.

The maximum tensile stress $\sigma_{ct,\max}$ occurs at the top of the pile cap. Cracking occurs when $\sigma_{ct,\max} = f_{ctm}$, where $f_{ctm} = f_{ctm}(t_r)$ is the mean tensile strength of concrete in the age $t_r$ days. At this moment, the effort $N_1$ is equal to the cracking normal effort $N_r$. This cracking effort may be written as:

$$N_r = h_o f_{ctm28} \qquad (16)$$

where $f_{ctm28}$ is the mean tensile strength of concrete at the age of 28 days and $h_o$ is the thickness of the surface layer that is relevant to calculated the minimum reinforcement.

## 6. RESULTS OF STRESS ANALYSIS

Results presented below were obtained for a set of pile caps with width varying from $L = 0.3$ m to $L = 0.8$ m and height ranging from $H = 0.3$ m to $H = 2.0$ m. In total 24 combinations were made of these two dimensions. Moreover, the cement content varied between $M_c = 300$ kg/m$^3$ and $M_c = 400$ kg/m$^3$. For the compressive strength of concrete the values of 20 MPa, 25 MPa, 30 MPa and 40 MPa were considered. It is assumed normal hardening cement, with in equation $s = 0.25$ (14). Data for thermal analysis are the same as those adopted previously.

Fig. 6 shows the relationship between the thickness of the surface layer $h_o$, as defined in equation (16), and the equivalent thickness $H_e$. As observed, the values of $h_o$ depend on the cement content. However, there is no a correlation with the compressive strength of concrete or with the equivalent thickness. As previously mentioned, the coefficient of heat transfer has a significant influence on temperature near the surface of member. Therefore, it will also influence the thickness of the surface layer. The results presented in this work are valid for a mean heat transfer coefficient set in the range of 12-14 W/m$^2$°C.

Fig. 7 shows variations of the thickness $h_o$ with the maximum adiabatic temperature $T_{a,\max}$, as defined in equation (4).

As shown in Fig. 7, there is a direct correlation between the thickness of the surface layer $h_o$ and the maximum adiabatic temperature rise. The adjustment equation is shown in Fig. 7. Anyway, it is recommended to consider a minimum value of 10 cm for $h_o$, as a prudent criterion.

In order to calculate the crack width, it is convenient to correlate the maximum tensile stress in concrete

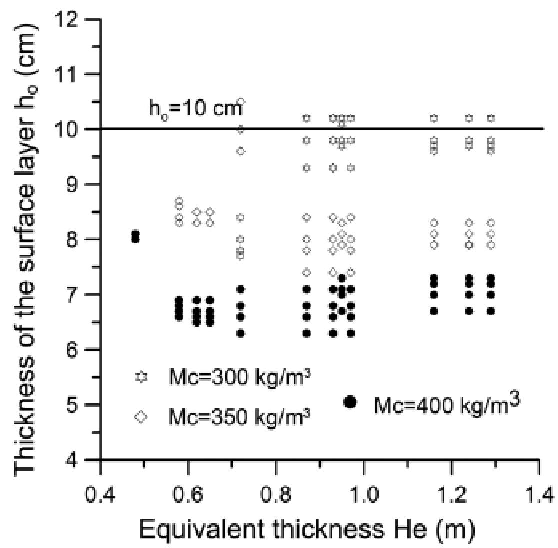


**Figure 6.**
**Thickness of the surface layer to calculate the minimum reinforcement**

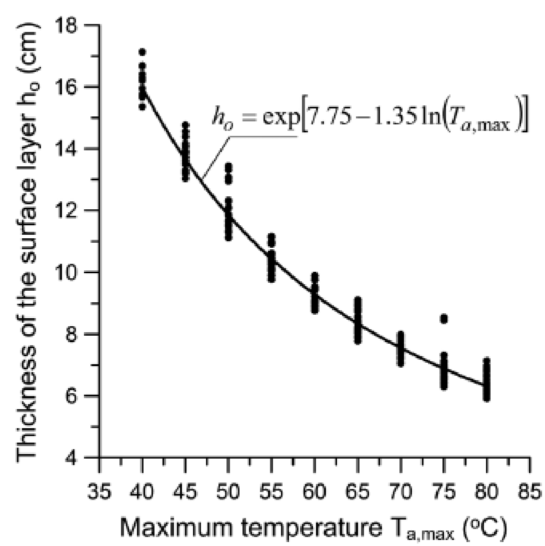


**Figure 7.**
**Variation of ho with $T_{a,\max}$**

with the maximum temperature difference $\Delta T$ between the core and the top of the pile cap. As seen, this temperature difference may be correlated with the equivalent thickness $H_e$ and the cement content $M_c$, by means of the equation (8).

The maximum stress $\sigma_{ct,\max}$ may be written as:

$$\sigma_{ct,\max} = R E_c(t) \alpha \Delta T \qquad (17)$$

where $R$ is the restraint factor for imposed deformations.

As shown in equation (17), the restrained strain $\varepsilon_r = R\alpha\Delta T$ is equal to the product of the restraint factor $R$ by the free thermal strain $\varepsilon_o = \alpha\Delta T$. This definition of $R$ is the same adopted in references [3,6].

At the moment of cracking, $t = t_r$, $\sigma_{ct,\max} = f_{ctm}(t_r)$ and $\Delta T = \Delta T_{cr}$. Thus, equation (17) may be written as:

$$R = \frac{f_{ctm}(t_r)}{E_c(t_r)\alpha\Delta T_{cr}} \tag{18}$$

which makes it possible to determine the restraint factor.

Values of $R$ obtained with the FEM are shown in Fig. 8. It is observed that there is no direct correlation between $R$ and the equivalent thickness $H_e$. The maximum value obtained was equal to 0.32. Assuming a safety factor equal to 1.4, its design value is $R = 1.4x0.32 = 0.45$. Soon, all cases analyzed are handily covered by the usual value $R = 0.5$.

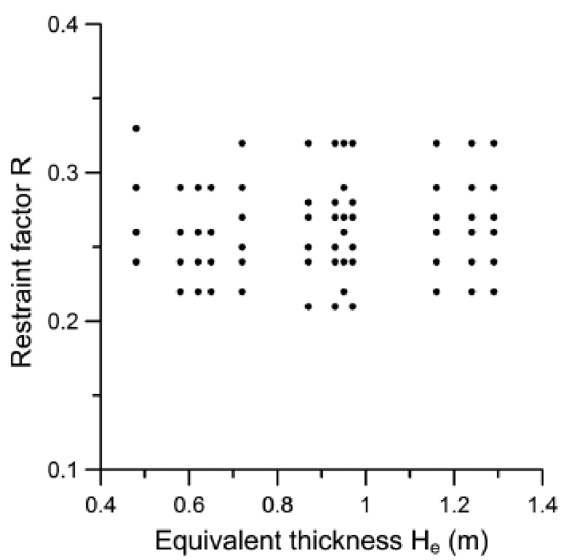


**Figure 8.**
**Restraint factors obtained with the FEM**

Fig. 9 shows the variation of $\Delta T_{cr}$ with respect to the equivalent thickness $H_e$.

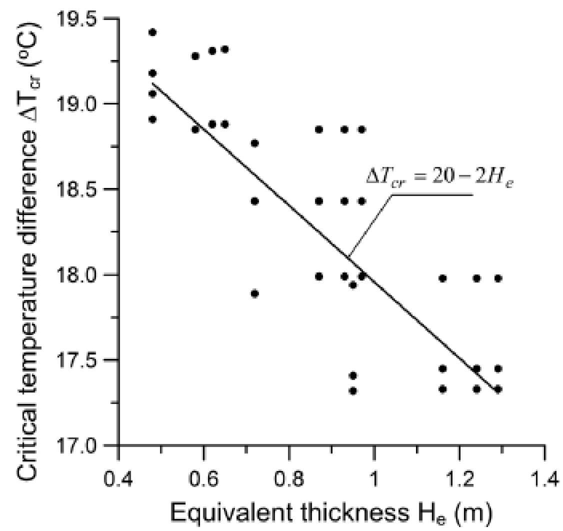


**Figure 9.**
**Critical temperature difference at the moment of cracking**

## 7. DESIGN PROCEDURE

As a result of this study, the following procedure may be recommended for design of skin reinforcement of concrete pile caps:

**Data:**

Pile cap dimensions: $A$ and $B$ in plan and height $H$, in meters.

Concrete: characteristic strength $f_{ck}$ (MPa) and cement content $M_c$(kg/m$^3$).

**Checking the risk of thermal cracking:**

Equivalent width (m): $L = \sqrt{\dfrac{4AB}{\pi}}$

Equivalent thickness (m): $H_e = \dfrac{LH}{(1+\delta)L + 2\delta H}$ ,

where it may be adopted $\delta = 0.365$.

Maximum temperature difference between the center and the top of the pile cap (°C):

$$\Delta T = \frac{(4760 + 90M_c)}{1000} H_e - \frac{(1840 + 9.8M_c)}{1000} H_e^2$$

If the heat of hydration of cement $Q_{h\infty}$ is different from 400 kJ/kg, the equivalent cement content $M_{ce} = (M_c Q_{h\infty})/400$ it may be used in place of $M_c$.

Critical temperature difference (°C): $\Delta T_{cr} = 20 - 2H_e$

- If $\Delta T < \Delta T_{cr}$ , there is no risk of thermal cracking. In order to avoid cracking caused by thermal shock and/or differential shrinkage, it may be adopted a skin reinforcement of approximately 2cm$^2$/m on the pile cap sides. This is the skin reinforcement for deep beams, as recommended by ACI 318 [9].

- If $\Delta T > \Delta T_{cr}$, there is a risk of thermal cracking of the pile caps sides. In this case, it is necessary to calculate skin reinforcement to control the crack width.

**Calculation of the skin reinforcement:**

*A) Minimum reinforcement to support the cracking effort:*

Adiabatic temperature rise: $T_{a,\max} = \dfrac{Q_{h\infty}\, M_c}{c\,\rho}$,

where $Q_{h\infty}$ is the heat of hydration per kg of cement (in kJ/kg), $M_c$ represents the cement content per m$^3$ of concrete (in kg/m$^3$), $c = 0.9$ kJ/kg°C is the specific heat of concrete and $\rho = 2400$ kg/m$^3$ is the density of concrete.

Thickness of the surface layer:

$$h_o = \exp[7.75 - 1.35\ln(T_{a,\max})] \geq 10 \text{ cm}$$

Minimum reinforcement area:

$$A_{s,\min} = \frac{100 h_o f_{ctm28}}{f_{yd}}, \text{ cm}^2/\text{m}$$

where $f_{ctm28}$ is the mean tensile strength of concrete at the age of 28 days and $f_{yd}$ is the design yield strength of steel.

*B) Reinforcement required for crack width control:*

According to the cracking model of CEB-FIP Model Code [8], the reinforcement ratio for crack width control is given by:

$$\rho_{se} = \frac{\phi R \varepsilon_{cn}}{3.6 w_{k,\lim}},$$

where:

$\phi$ is the diameter of the steel bar, in mm;

$\varepsilon_{cn} = \alpha \Delta T$ is the imposed deformation, with $\alpha = 10^{-5}$ °C$^{-1}$;

$R = 0.5$ is the restraint factor;

$w_{k,lim}$ is the limit value of crack width, in mm.

The reinforcement area is $A_s = \rho_{se}\, h_e$, where $h_e$ is the thickness that is relevant for calculation of crack width, given by $h_e = 2.5(c + 0.5\phi)$, where $c$ is the nominal concrete cover of the steel bars.

If: $A_s < A_{s,min}$, it is adopted $A_s = A_{s,min}$.

## 8. CONCLUSIONS

The finite element method (FEM) was employed for two-dimensional analysis of heat transfer. Through the definition of a width equivalent, this method was used to determine temperature rise in concrete pile caps due to heat of hydration of cement.

The temperature distribution obtained with FEM was used to perform the stress analysis of the critical section of pile cap. The main parameters involved were correlated with an equivalent thickness of the pile cap.

Based on the obtained results, a methodology to calculate the skin reinforcement of large concrete pile caps was proposed. This reinforcement is intended to control the crack width resulting from thermal stresses due to hydration of cement.

It should be noted that the skin reinforcement does not prevent cracking. It only limits the crack width, avoiding the emergence of a large crack that might compromise the durability of the structural element.

For massive pile caps, it is necessary to adopt a set of measures, such as reducing the cement content, using pozzolanic cements, concrete placement in layers of small height, precooling and post-cooling of concrete, protection of concrete to avoid very rapid cooling of the surfaces, prolonged cure to reduce shrinkage, among others. The employment of skin reinforcement, solely, does not eliminate the need for such additional cautions.

Results of thermal analysis are very dependent on the concrete properties and boundary conditions. So it should be clear that the results obtained, as well as the proposed design methodology, are limited to the conditions that have been adopted in this work.

The method for checking the risk of thermal cracking and calculating skin reinforcement may be used to pile caps of equivalent height up to 2.0 m. For larger pile caps, it is necessary to perform specific study, considering thermal properties and boundary conditions corresponding to the actual situation.

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