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COMPARISON OF TEMPERATURE RISE PREDICTION METHODS WITH MEASURED TEMPERATURES DUE TO HYDRATION OF CEMENTITIOUS MATERIALS IN CONCRETE

K.C. Tayade[1], J.P. Gupta[2], Kunwar Sushil Kumar[3], Devendra Ramtekkar[4]

[1]*Dy. General Manager (Quality Control), Maharashtra Metro Rail Corporation Ltd., Nagpur.*
[2]*Manager (Reach-1), Maharashtra Metro Rail Corporation Ltd., Nagpur.*
[3]*Addl. General Manager (Reach-1), Maharashtra Metro Rail Corporation Ltd., Nagpur.*
[4]*Executive Director, Maharashtra Metro Rail Corporation Ltd., Nagpur*

Abstract

The paper deals with the experimental study which was carried out to investigate the temperature rise characteristics of the concrete used in a prototype component of foundation for Interchange station of Nagpur Metro Rail Project, Nagpur, Maharashtra State. The actual temperature rise was measured by embedding Thermo Couples – Resistance temperature detector (RTD) in the prototype structure during concreting. The measurements were taken at 5 locations in the component – a pile cap by inserting thermo couples (TC's) in 3 layers at each location totaling to 15 in all. The monitoring of temperature recorded by thermocouples was done by digital recorder manually on hourly basis up to 13 days and then once in 3 hrs. up to next 21 days and beyond but restricted to 14 days in this reporting. The temperature at the time of insertion i.e. initial/starting temperature was noted and maximum temperature reached, the time to reach peak temperatures was recorded.

The plot of time-temperature demonstrated the actual temperature rise, its trend both increasing and decreasing wherein all the thermal effects in and on the concrete such as effects of exothermic reaction of hydration of cement, conduction, convection and thermal properties of concrete such as specific heat, thermal conductivity, diffusivity and so also the external factors such as ambient temperature, humidity, wind speed, solar radiation, curing and form removal have played their role and the resultant was the actual temperature reached at that point where the measurements have been recorded by embedded TC's. The comparison of the actual measured temperatures made with the various methods available to predict maximum concrete temperatures by empirical formulas such as PCA method, adiabatic temperature rise curves as published in ACI committee reports and ASCE method showed that the actual measured temperatures vary on plus as well as minus side over predicted.

***Keywords: -** Heat of hydration; Maximum temperature; Temperature gradient, mass concrete.*

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1. INTRODUCTION

ACI manual of concrete practice committee{1}defines mass concrete as "any volume of concrete with dimensions large enough to require that measures be taken to cope up with the generation of heat from the hydration of cement and attendant volume change to minimize cracking. As per ACI committee {2} in general heat generation should be considered when minimum cross sectional dimensions approaches or exceeds 360mm or cement contents above 356 Kg/m3 are used. The most important characteristics of mass concrete is thermal behavior. Hydration of Portland cement is an exothermic reaction and a large amount of heat is generated during the hydration process in mass concrete elements. Since concrete has low conductivity, a great portion of generated heat is trapped inside the mass concrete and is dissipated slowly. This situation leads to a temperature increase inside and a temperature difference between the center and outer part of mass concrete element. Temperature difference is a cause for tensile stresses which form thermal cracks in concrete structure. Thermal cracking may cause loss of integrity and shortening of service life of the concrete element. The literature shows that the factors most relevant to cracking in massive structures are thermal stresses induced by thermal gradients {3}. The thermal stress is given by the formula as given in Equation 1.

$$\sigma t = Kr \times \frac{E}{1+\varphi} \times \bar{\alpha} \times \Delta t \qquad \text{Eq (1)}$$

σt = tensile stress
Kr = degree of restraint

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E = elastic modulus
$\bar{\alpha}$ = coefficient of thermal expansion
Δt = temperature change
Φ = creep coefficient

The temperature change Δt can be graphically represented as shown in Fig.1

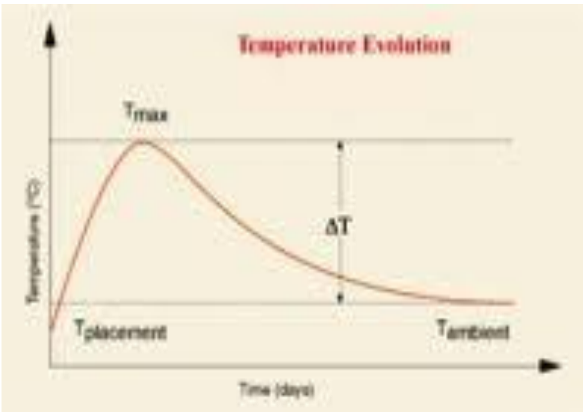

Fig 1: Temperature Evolution in concrete

Hence measurements of temperatures inside concrete is an important tool to know the pattern of rise of temperature, otherwise, the concrete temperature prediction methods such as given in Portland cements associations (PCA) design and control of concrete mixes or using the Schmidt method in conjunction with 207.1R {4} committee documents adiabatic temperature curves has to be resorted to.

In this study the actual measurements of temperature were made in a prototype structure and the predicted maximum temperatures from the empirical methods were compared against actual temperature data obtained during measurements.

2. RESEARCH SIGNIFICANCE OF PRESENT STUDY

The temperature prediction methods commonly referred are empirical and are developed over 50 years ago. However with the change in construction methodology, form types, cement chemistry, cement fines and types/grades, supplementary cementitious materials (SCM's) and chemical admixtures these methods may not be appropriate or may need to be updated. The use of mass concrete elements have also increased in recent years creating a need to accurately predict the temperatures.

In this study, an effort has been made to assess the ability of these methods by checking their accuracy to predict the maximum temperatures by comparing with the actual maximum temperatures measured in a prototype structure.

3. BACKGROUND INFORMATION

3.1 Maximum Temperature and Maximum Temperature Difference

The prediction of maximum temperature and rise of temperature is important in controlling heat of hydration. The maximum in place temperature reached in concrete members can affect the long term performance of structure during its service life. Another effect of high temperatures in concrete is delayed ettringite formations (DEF) and has been shown to cause durability problems. The time to reach maximum temperatures and the maximum temperatures are critical in determining the maximum temperature difference. Predicting the maximum temperatures of mass concrete has always been the main concern of designers and builders of mass concrete structures and one of the earliest efforts to predict maximum temperatures of mass concrete were carried out in late 20's and early 30's during the design phase of Hoover Dam in U.S.A. One of the most popular methods to predict the mass concrete peak temperature rise is using adiabatic temperature curves. The current state of practice is to perform adiabatic calorimetry testing on concrete mixtures and use finite element methods and analysis to predict temperature distribution over time.

Large temperature difference can occur when the concrete core is hot and the ambient temperatures are low or when the forms are removed when the concrete underneath is hot, typically referred to as "thermal shock". The maximum temperature difference causes a change in volume because of thermal expansion/contraction and can induce thermal cracking due to internal restraint or when the member is restrained by adjacent elements or foundations. However the tensile strength of concrete is also a function of concrete's maturity and the thermal gradient required to produce cracking is consequently also a function of concrete's maturity. This means that the a concrete element could in some circumstances have a low risk of cracking while at the same time have a high temperature difference, if this large temperature difference occurs at a late age.

4. TEMPERATURE PREDICTION METHODS

The 3 methods considered for comparison in this paper for the measured maximum temperatures, are PCA method, Graphical method of ACI 207.2R and the ASCE Method.

4.1 PCA METHOD

The Portland Cement Association's (PCA) Design and control of concrete mixtures 14th edition, 2003, {8} gives a quick method for estimating the maximum temperature developed in mass concrete members. This method will be referred to as "PCA method" in this paper. This method postulates that the

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maximum temperature rise of concrete above the placement temperatures will be at 120C (21.60F) for every 100 kg of cement used in concrete. However, method is approximate for concrete containing 300 to 600 kg of cement per cubic meter (506 to 1012 lb per cubic yard) of concrete, also assumes that the least dimension of concrete member is at least 1.8 m (6 ft). PCA method does not provide any information on time of maximum temperature; treats all cements the same and gives no guidelines on how to account for slag cements. ACI committee 207 suggests that the modification to account for supplementary cementitious materials (SCM's) can be made by presuming that they liberate approximately half the amount of heat of cement for a given mass.

The following equation shows the PCA calculations for maximum concrete temperature Tmax when altered to account for the initial temperature and SCM's.

$$T_{max} = T_i + (12 x W_c/100) + (6 x W_{scm}/100) \qquad \text{Eq (2)}$$

where,
Ti = concrete placement temperature in oC
Wc = weight of cement in kg/m3
Wscm = weight of SCM in kg/m3

Due to the consideration that the hydration of a blended cement (PC + GGBS for this study) is not just the simple summary of the hydration of the two individual components, but the co-reactivity effect of PC and additions which has to be taken into consideration and since so called PC reaction and GGBS reaction occurred coincidently, further division of the total heat of hydration into heat from PC and heat from GGBS is difficult{9}. Also as found out in {10} by the finite-element model results, validated by the semi-adiabatic calorimetry tests, which showed a beneficial effect of using GGBS in concrete structures during summer and that when 50% of the CEM I was replaced with GGBS, thermal loading (or T1 values) reduced by 30%. Accordingly, in the present study, it is presumed that GGBS will liberate approximately, 70% of the amount of heat of cement for a given mass.

4.2 Graphical Method of ACI 207.2R

ACI207.2R {5} contains several charts and equations based on empirical data that can be used to estimate the maximum temperature in mass concrete. This is referred to as "Graphical method of ACI" in this paper. Adjustments can be made for member size; cement type; exposure condition, use of slag and /or fly ash and the placement temperature. This graphical method gives adiabatic temperature rise which can be represented by:

$$T_r = T_I \times \frac{T_f}{T_{1800}} \qquad \text{Eq(3)}$$

Where
Tr = Cement turbidimeter (ASTM C115) fineness adjusted adiabatic temperature for 171 kg (377lb) of cement in oC
TI = adiabatic temperature rise for Type I cement from ACI 207
Tf = Heat generation in percent of 28 days heat generation for the measured cement fineness from ACI 207.2R
T1800 = Heat generation in percent of 28 day heat generation for cement fineness of 1800 cm2/gm form ACI 207.2R

4.3 The ASCE Method

As reported in Final report prepared by the department of Civil and Coastal Engineering, University of Florida {7}, the adiabatic temperature rise resulting from the heat of hydration can be calculated using the expression developed and presented by Tanabe et al in seminar proceedings for Finite element analysis of reinforced structures, Tokyo, Japan in 1985 and published by the American Society of civil engineers (ASCE) the following year. This is referred to as "The ASCE method" in this paper. The temperature rise is given by the expression

$$T(t) = K(1-e\alpha t) \qquad \text{Eq(4)}$$

where T = Temperature in 0C
t = time in days.
K & α are constants based on casting temperature (oC)

The values of K & α can be obtained from the plots developed and given. The total amount of heat generated and the rate of heat generation can also be calculated by the equation given in the paper by Tanabe et al. However, they are not given here because this paper deals only with comparative study of temperature rise.

5. SCHEME OF STUDY AND EXPERIMENTAL PROGRAM

The experimental program in the study was to measure and monitor the temperature rise in concrete prototype structure. The temperature measurements were done by inserting an array of thermo couples – Resistance temperature detector (RTD) at five different locations and at three levels. The thermo couples (TC's) used were PT100. The location plan is as shown in Figure 2.

The structure geometry and the locations of thermo couples are as per Figure 3 & 4.

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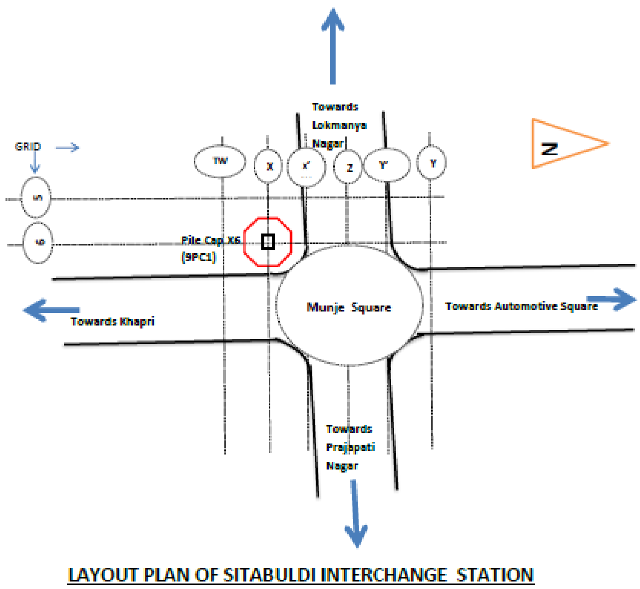

Fig 2: Location Plan

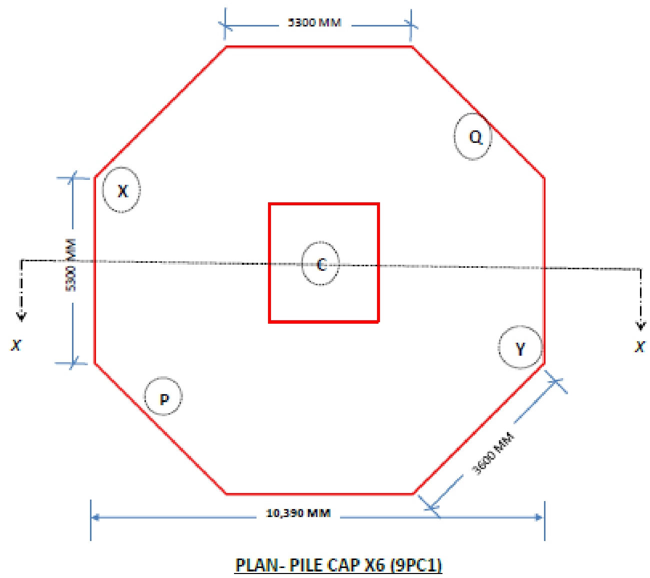

Fig 3: Plan of Pile cap

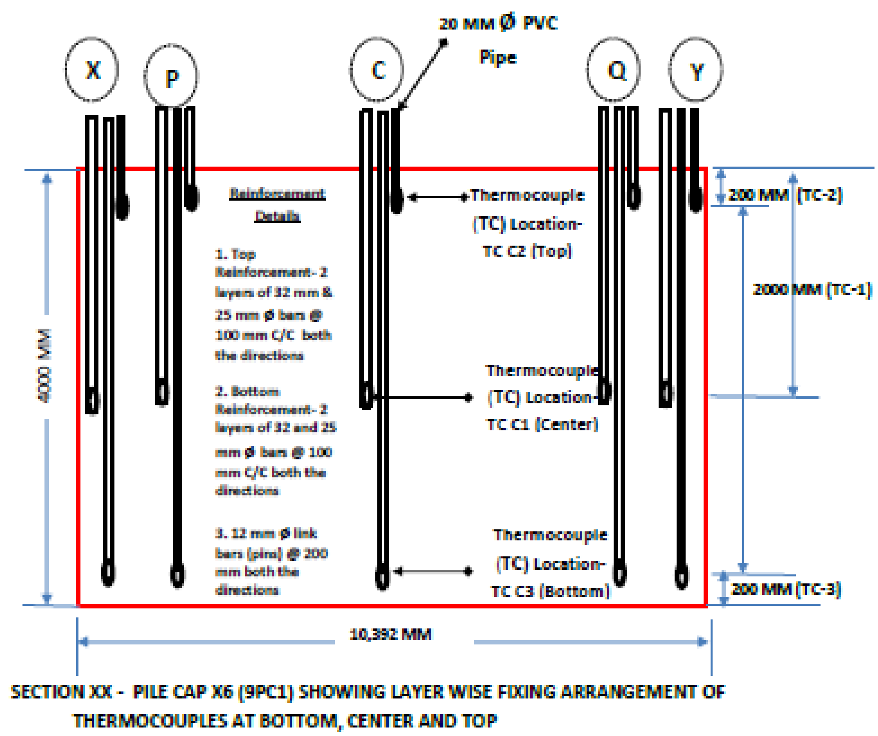

Fig 4: C/s of Pile Cap

The photograph (1), (2) and (3) shows the actual photo of locations of Thermo couples (TC) in pile cap and photograph (4) show the Pile Cap after Concreting wherein PVC Pipes as Casing for embedded TC's are clearly seen.

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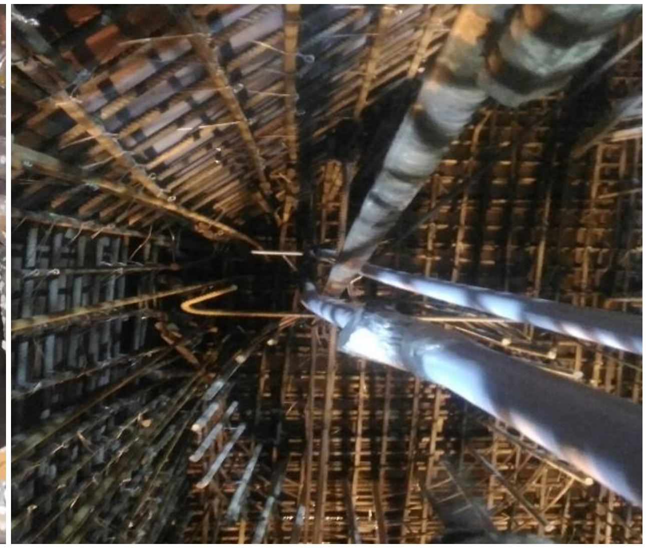
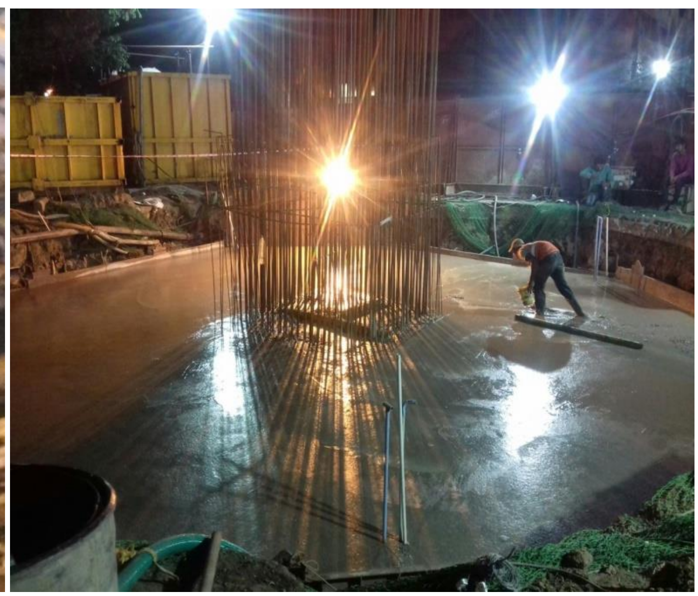


Photograph (1): Top layer **Photograph (2):** Middle layer

Photograph (3): Bottom layer **Photograph (4):** Pile Cap after completion of Concreting.

The scheme of study was to insert the thermo couples at five locations in pier cap namely location X, Y, P, Q, on sides and location C at centre (Figure 3). At all the locations the structure was having same geometry and cross-section. At a particular location, in cross section total three thermo couples were embedded layerwise. The thermo couples in the centre layer are Post fixed as 1, that at the top layer are Post fixed as 2 and that in the bottom layer are Post fixed as 3 (Figure 4). The thermo couples at the top and bottom were inserted 0.20m inside to accommodate the reinforcements. The thermo couples at the location on sides namely X,Y,P,Q were inserted 0.50 m from face of concrete inside the concrete structure, so that the effect of surface cooling is minimized. One TC protected from direct sunlight to measure the ambient temperature and was kept outside the structure.

The scheme of study adopted is as per Table:1 below

Table 1: Table showing the scheme of study

Sr.No.	Location	Number of T.C's	Notation**
1	C	3	C', C2, C3
2	X	3	X2, X3
3	Y	3	Y1, Y3
4	P	3	P3
5	Q	3	Q2, Q3

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* * The Thermo Couples at Location X1; Y2; P1; P2 and Q1 had malfunctioned and so are not considered in this study. The Thermocouple at location C1 malfunctioned during embedment and was replaced with another TC with notation as C’.

Hence, 10 thermo couple readings were available for this study as 5 thermo couples had malfunctioned.

The formwork used was of steel shuttering and it was removed after 3 days.

The test results for Cement and GGBS are as per Table 2.

Table 2: Table showing the test results for cement & GGBS

Cement		GGBS	
Type	OPC -53 Grade	Type	Grade-100
W-M-Y	16-04-2018	% of Silica content (by Mass), (SiO2)	36.50
% of Fly-Ash	Nil	% of Calcium oxide (by Mass), (CaO)	35.34
Fineness(m2/kg)	293	Fineness(m2/kg)	338.6
Normal consistency (%)	29.0%	Normal consistency (%)	35.5%
Soundness (Autoclave)	0.05	Soundness (Autoclave)	0.0%
Setting time (min.) Initial Final	 150 230	Slag Activity Index (%) 7 days 28 days	 74 91
Strength (N/mm2) 3days 7days 28days	41.0 50.4 63.2	Retained on 45 micron wet sieving (%)	4.11

The mix proportion for the concrete Grade M-40 (20MSA) adopted was as per Table 3.

Table 3: Table showing the mix proportion for the concrete Grade M-40(20MSA)

Cement (Kg.)		Water (Kg.)	Sand (Kg.)		Coarse aggregate (Kg)		Plasticiser (kg)
					20mm	10mm	
Cement OPC 53 grade	GGBS JSPL	143 KG	River sand	Crushed sand	626 KG	512KG	3.28KG
246 KG	164 KG		600 KG	257 KG			

The F.M. of R/sand was 2.9 and Crushed Sand was 3.15 which was very coarse sand and categorized as Zone-II and Zone-I as per IS 383.

The monitoring of temperature recorded by thermocouples was done by digital recorder manually on hourly basis up to 13 days and then once in 3 hrs. up to 21 days but reporting is restricted to 14 days in this study.

The graphs (Figure 5 to Figure 10) below represents the plots of time vs. temperatures actually recorded for some thermo couples and are given here as sample.

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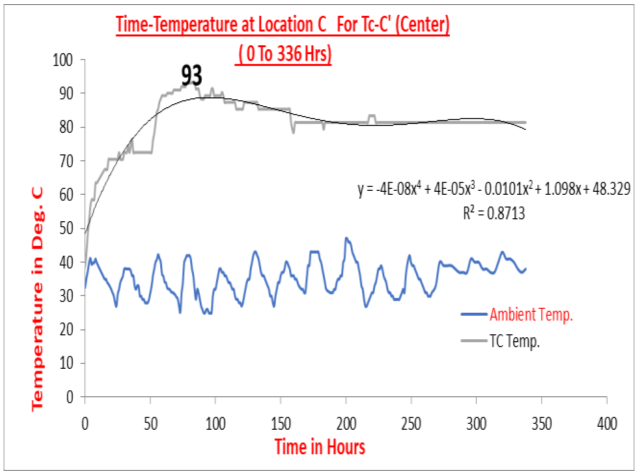

Fig 5: Time – Temperature record for TC- C'

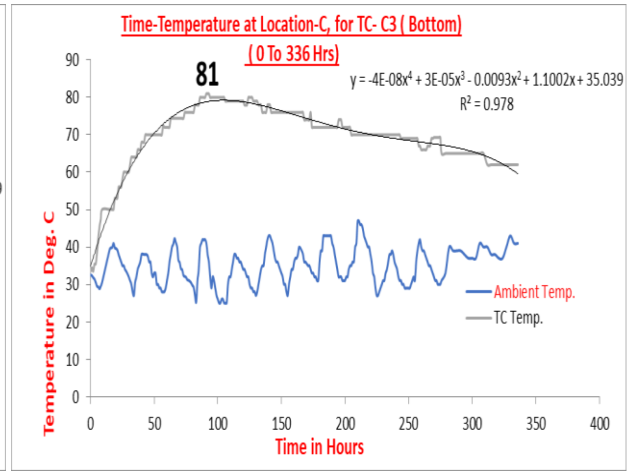

Fig 6: Time – Temperature record for TC- C3

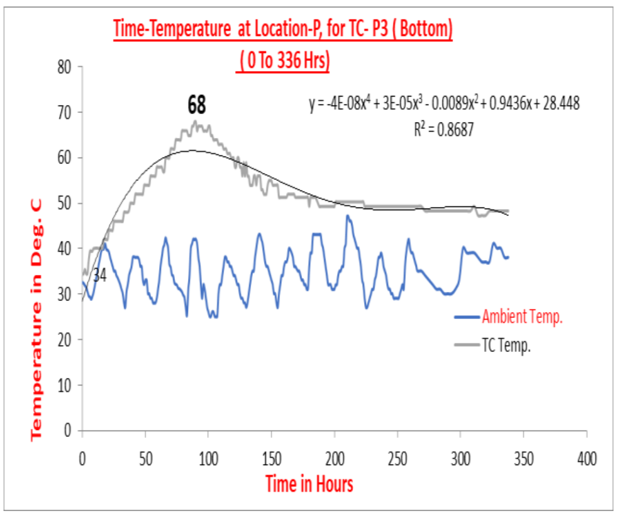

Fig 7: Time – Temperature record for TC-P3

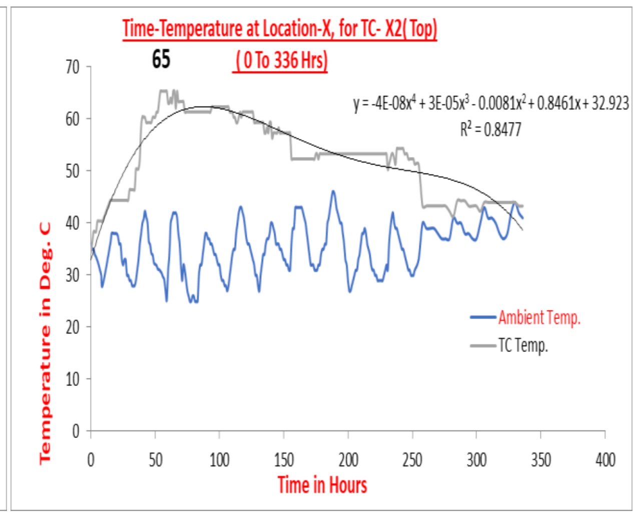

Fig 8: Time – Temperature record for TC- X2

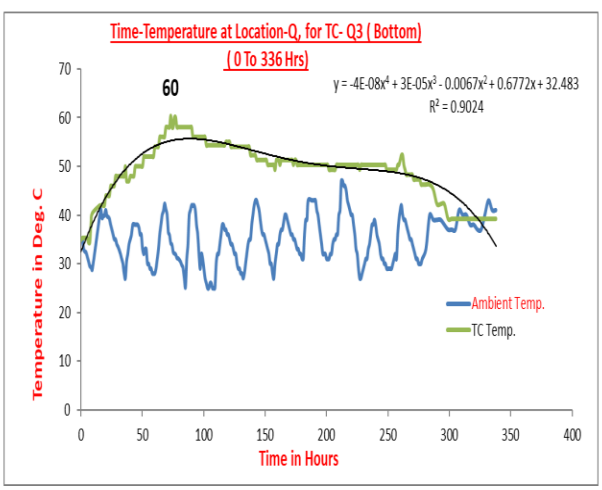

Fig 9: Time – Temperature record for TC-Q3

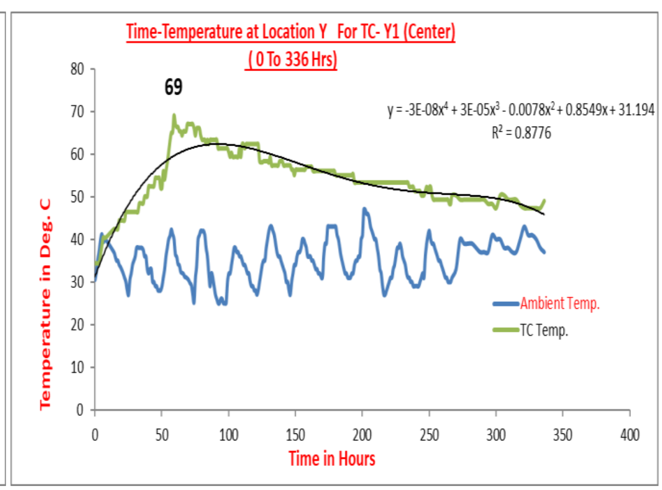

Fig 10: Time – Temperature record for TC- Y1

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Table 4: Table showing the details of temperature recorded by Thermocouples in concrete layer

Location	C			X		Y		P	Q	
Thermo Couple	C’	C2	C3	X2	X3	Y1	Y3	P3	Q2	Q3
Placement Temperatures 0C	31.5	31.5	31.5	31.5	31.5	31.5	31.5	31.5	31.5	31.5
Max.Temp. Recorded. (Tmax) 0C	93	87	81	65	85	69	60	68	68	60
Period to reach Max. Temp. (Hrs)	80	87	91	67	76	59	57	89	56	73
Ambient Temp. (Tam) 0C	42	30	41	41	32	40	29	42	31	34
Diff.of Temp.at peak Δ Tp = (Tmax–Tam) 0C	51	57	40	24	53	29	31	26	38	26

** Thermo Couple at Location X1; Y2; P1; P2 and Q1 are malfunctioned.

6. TEMPERATURE PREDICTION

6.1 PCA Method

The maximum temperatures were calculated by this method for all the ten TC’s. The PCA method is suited in our case as the cement content in mix proportion used was within the cement content range specified and also the least dimension of prototype structure was more than the least dimension of 1.8 m. Cement used was OPC with 40% GGBS; the contribution of this SCM’s was assumed to be 70% of cement as per recommendation of {9} and {10) and as given in equation (2).

6.2 Graphical Method of ACI 207.2R

As Wagner Turbidimeter values are rarely available, equivalent fineness values to the Wagner Turbidimeter method (ASTM C115) were estimated from the Blaine’s fineness values using equation (5)

Sw = 0.56*Sb Eq (5)

Where Sw = Wagner Specific surface in m2/kg
Sb = Blaine’s Specific surface in m2/kg

The cement used was having Blaine’s fineness of 293 m2/kg, so that the Wagner specific surface works out to 164.08 m2/kg. Maximum placement temperatures were calculated with and without adjustments for cement fineness as per equation (3). Values used in the calculations were manually extracted from the charts within ACI 207.2R (Charts 2.1and 2.2)

6.3 ASCE METHOD

The maximum placement temperature (adiabatic) for all the TC’s location were calculated from the equation (4). From the plots developed, for our case with the temperature of casting of 30 0C and unit cement content of 246 kg/m3 the values of constants works out to K = 51 and α = 1.40. The time (t) is taken as the time when measured temperature have reached maximum i.e. the time of T(max).

The measured and calculated Maximum Temperatures in Concrete by various empirical methods are shown in Table 5 below.

Table 5: Table showing the Maximum recorded and calculated Temperatures in Concrete.

Thermo Couple	Maximum Concrete Temperatures 0C				
	Measured	PCA Method using Eq. (2)	ACI 207.2R Method Not Corrected for Fineness	ACI 207.2R Method Corrected for Fineness	The ASCE Method using Eq. (4)
C’	93	74.8 (Since placement	70.3 (Since	73.3 (Since placement	82.0
C2	87				82.2

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C3	81	temperatures are the same, calculated Maximum Concrete Temperatures are same for all TC's)	placement temperatures are the same, calculated Maximum Concrete Temperatures are same for all TC's)	temperatures are the same, calculated Maximum Concrete Temperatures are same for all TC's)	82.2
X2	65				81.5
X3	85				81.9
Y1	69				80.9
Y3	60				80.7
P3	68				82.2
Q2	68				80.6
Q3	60				81.8
Av. of all TC's in 0C	73.6	74.8	70.3	73.3	81.6
% deviation from measured average maximum temperatures	0.0	(+) 1.6	(-) 4.5	(-) 0.4	(+) 10.8

7. RESULTS AND DISCUSSIONS

The table 5 shows the recorded (measured) maximum temperatures at the ten Thermo couple inserted locations in the structure; it also shows the maximum temperatures calculated for the structure, which in this case is the same for all the TC's as placement temperatures and other parameters such as ambient temperatures, curing conditions are the same. Hence, the comparison is made with the measured average maximum temperature of the measured calculated temperatures. The average difference in measured and predicted maximum temperature in PCA method was (+)1.2 oC an error of 1.6%. The structure meets this methods assumptions and the results from this comparison demonstrates that PCA method can be a useful tool for quick estimates to show the effect of cement content on heat development. The avg. difference for ACI graphical method works out to (-)3.3oC(error 4.5%) and (-)0.3oC (error 0.4%) for without and with correction for fineness respectively. Even though, in this case there is a good agreement in measured and calculated temperatures, the reliability of this method is poor because cements have changed significantly since the charts in ACI were developed and due to use of SCM's.

The ASCE method predicted with avg. difference of (+)8.0oC an error of 10.8%, this may be due to the fact that the time (t) in days is taken as the time when measured (recorded) temperatures have reached maximum at that particular T.C. location.

8. CONCLUSION

The actual recorded temperatures were found to be lesser (except ACI methods) than those values obtained from the methods discussed earlier. and this could be attributed to losses due to the combined effects of thermal properties of concrete, environmental and weather conditions and curing and form removal. Hence, these methods have limited usefulness and can be a tool to roughly and quickly estimate the maximum temperature rise based on cement content, fineness, placement temperatures.

The maximum temperature; the time to maximum temperature and the time spent at elevated temperatures were observed to be critical parameters in predicting the possibility of thermal shock susceptibility. This suggests that the time of removal of formwork must be delayed preferably beyond the point of occurrence of peak.

The temperature measurement by inserting Thermo Couples has limited adoptability as such studies could not be done in every structure, however, the results obtained from such experimental studies could be used to validate the results from mathematical models.

The numeral methods/mathematical models need to be developed as a generalized prediction tool for accurate prediction of temperature profiles in concrete structures.

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