Determination of Coefficient of Thermal Expansion (CTE) of 20MPa Mass Concrete Using Granite Aggregate

GO Chee Siang
30, Jalan 1/27D, Section 6, Wangsa Maju 53300, Kuala Lumpur, West Malaysia.
csgo@loh-loh.com.my

Abstract. Experimental test was carried out to determine the coefficient of thermal expansion (CTE) value of 20MPa mass concrete using granite aggregate. The CTE value was established using procedure proposed by Kada et al. 2002 in determining the magnitude of early-ages CTE through laboratory test which is a rather accurate way by eliminating any possible superimposed effect of others early-age thermal deformation shrinkages such as autogenous, carbonation, plastic and drying shrinkage. This was done by submitting granite concrete block samples instrumented with ST4 vibrating wire extensometers to thermal shocks. The response of the concrete samples to this shock results in a nearly instantaneous deformation, which are measured by the sensor. These deformations, as well as the temperature signal, are used to calculate the CTE. By repeating heat cycles, the variation in the early-ages of concrete CTE over time was monitored and assessed for a period of upto 7 days. The developed CTE value facilitating the verification and validation of actual maximum permissible critical temperature differential limit (rather than arbitrarily follow published value) of cracking potential. For thick sections, internal restraint is dominant and this is governed by differentials mainly. Of the required physical properties for thermal modelling, CTE is of paramount importance that with given appropriate internal restraint factor the condition of cracking due to internal restraint is govern by equation, \( \Delta T_{\text{max}} = \frac{3.663E_{\text{ctu}}}{\alpha_c} \). Thus, it can be appreciated that an increase in CTE will lower the maximum allowable differential for cracking avoidance in mass concrete while an increase of tensile strain capacity will increase the maximum allowable temperature differential.

Keywords: CTE, granite aggregate, early-age thermal cracking, thermal shock, temperature differential, tensile strain capacity, internal restraint

1. INTRODUCTION

Experimental test was carried out to validate the actual value of coefficient of thermal expansion (CTE) of 20MPa mass concrete using granite aggregate quarried from borrow hill located within project site for the construction of 265MW TNB Hulu Terengganu Hydroelectric Dam.

“The coefficient of thermal expansion (CTE) of a material is defined as the unit change in length per degree of temperature change. The CTE measures expansion or contraction of a material with temperature. For a given temperature change, the free thermal expansion of the concrete is determined by the CTE, \( \alpha_c \). Thus, concrete, the most important composite construction material, is not an exception. At a simplified level, concrete has paste and aggregate phases. These two main constituents of concrete is dependent on material factors such as type of cement paste, aggregate type, moisture conditions, age, and environmental factors such as temperature fluctuations as well as relative...
humidity; whereby aggregates constitute approximately 75% of the volume of concrete which therefore is a major factor influencing the CTE of concrete. Therefore, thermal properties of the coarse aggregate can have a significant effect on mass concrete. Concrete containing low-thermal-expansion aggregates such as granite and limestones generally permit higher maximum allowable temperature differences than concretes made using high-thermal-expansion aggregates such as gravel, as indicated in below Table 1. This means that selecting an aggregate with a low thermal expansion will reduce the potential for thermal cracking (ACI 207.2R-07, 2007).

“The CTE in concrete is important in numerous engineering applications including design of Dams & Hydropower, Highway Infrastructures such as bridges and pavements. It plays a key role in design, performance, and service life of concrete structures because it affects critical stresses and joint and crack openings. The CTE in aggregates influence the performance of concrete especially mass concrete structure (MCS) mainly because of its effect on dimensional change attributable to the change in temperature condition (ACI 207.2R-07, 2007).”

Researchers (Naik et al. 2011; Jahangirmejad et al. 2009; Mallela et al. 2005; Won M 2005; Ndon & Bergeson 1995) have investigated that “the CTE of concrete with various aggregates and reported a wide variation in the values of CTE among different aggregates depending on type, origin, and geographic location. To minimize the effect of thermal expansion or contraction of concrete on the performance of concrete infrastructures in particularly MCS, the measured value of the CTE should be incorporated in design equations. A higher CTE of concrete may adversely affect the durability of concrete because of thermal incompatibility of its components (Naik et al. 2011).”

A concrete with a low CTE can significantly reduce the risk of early-age thermal cracking as well as the percentage of reinforcement required for the control of thermally-induced cracks. Therefore the selection of aggregate of low αc (coefficient of thermal expansion of concrete) can have a significant impact in preventing, minimizing or controlling early-age thermal cracking. Generally, the definitions of early-age and long-term thermal cracking are defined as i) Early-age: Typically up to seven days and ii) Long-term: Beyond the period of the early-age temperature cycle, typically beyond 28 days (Bamforth 2007).

Table 1. Coefficients of thermal expansion (CTE) for different kinds of aggregates (Bamforth 2007)

| Coarse aggregate/ rock group | Thermal expansion coefficient (microstrain/ °C) | | |
|-----------------------------|-----------------------------------------------|--|---|
|                             | Rock                                          | Saturated concrete | Design value |
| Quartz                      | 10.0-12.2                                     | 11.7-14.6          | 14           |
| Sandstone                    | 4.3-12.1                                      | 9.2-13.3           | 12.5         |
| Marble                      | 2.2-6.0                                       | 4.4-7.4            | 7            |
| Dolomite                    | 3.8-9.7                                       | 8.1-11.0           | 10.5         |
| Basalt                      | 1.8-11.7                                      | Average 9.2        | 9.5          |
| Limestone                    | 4.0-9.7                                       | 7.9-10.4           | 10           |
| Gneiss                      | 1.8-11.7                                      | 4.3-10.3           | 9            |
| Glacial gravel              | -                                             | 90-13.7            | 13           |
| Lying coarse and fine       | -                                             | 5.6                | 7            |
| Lying coarse and natural aggregates fines | -                                           | 8.5-9.5            | 9            |

BS EN1992-1-1:2004 also recommends that unless more reliable information is available, the coefficient of thermal expansion should be assumed to be 10µε/°C. While this is a representative value for a wide range of aggregates there are some commonly used materials in the UK, in particular flint gravels that lead to higher values in the order of 12 µε/°C, as shown in Table 1. The higher value should be assumed if no data are available. In the absence of project data, the value of CTE can be estimated from knowledge of the CTE of the aggregates using aforesaid Table 1 where proposed design values are given. While there is insufficient data to provide values with a defined level of
statistical significance, the proposed values are at the high end of the observed range and represent safe values could be used in design. Precisely, CTE itself is a physical property of the concrete depending on a certain concrete mix designer’s fixed proportions, exclusively on the selected/predetermined age of concrete which corresponds to the peak temperature results derived from the established semi-adiabatic temperature rise test. Therefore, it is customary whenever possible to rely on the physical properties of the concrete such as CTE based on actual tests carried out on the proposed concrete mix rather than pick published values that however still would provide sound reference as to which limits (maximum/minimum) a certain physical property under investigation should lie within.

In other words, once the mix design is assigned and the age of the CTE testing is determined; then the value obtained by any representative testing becomes a ‘constant’ property of the concrete regardless whether it is measured on site or in the laboratory as there is not any known factor to convert the value of CTE as determined in the laboratory to a specific notional size on-site once the conditions for the testing have been fixed. It has been demonstrated and experience by author that in the determination of an accurate value of CTE for the thermal modelling and design purposes by means of large scale mock-up has some limitations in the sense that it require to determine very accurate instruments in a very “gross” environment where any little modification of the test set up and boundary conditions for the strain gauges can amplify errors beyond control. In addition, there is possibility that strain induced by early-age thermal deformations such as autogenous, plastic, carbonation, and drying shrinkage would also sum-up the value of total strain measured by the gauges on-site and therefore a large scale pour used for determining CTE would not be recommended. This is because a certain degree of aforesaid measured thermal induced strains would be captured by the strain gauge as well which should be avoided in order to have an accurate and genuine CTE value.

Thus, there is less volume and time consuming processes thought from which CTE of a certain mix design can be determined in controlled environment by choosing an appropriate value of CTE as determined by testing at the ages where peak/cooling cycles, based on simple method established by (Kada et al. 2002) and correlating with the semi-adiabatic temperature rise test results derived from hot box test. This method is relatively inexpensive but rather accurate way by eliminating any possible superimposed effect of others early-age thermal deformation shrinkages as mentioned in above paragraph. Furthermore, choosing an appropriate range of ages for the determination of CTE on the designated concrete mix will allow (in conjunction with thermal data obtained from large scale volume mock-up tests) the selection of those values of CTE corresponding to the time where concrete is rigid enough to crack and accurate input value for the modelling determined experimentally under controlled conditions on the proposed mix will be eventually available.

Of the required physical properties for thermal modelling, CTE is of paramount importance as it can be proven (Bamforth 2007 and TxDOT 2011) that with given appropriate internal restraint factors (the main type of restraint acting on mass concrete is actually and relaxation factors for the creep of concrete at early ages) the condition of cracking due to internal restraint is governs by equation, \( \Delta T_{\text{max}} = 3.663 \frac{\varepsilon_{\text{cm}}}{\alpha_c} \). From the aforesaid equation it can be appreciated that an increase in the CTE will lower the maximum allowable differential for cracking avoidance in unreinforced concrete while an increase of the tensile strain capacity will increase the maximum allowable temperature differential. BS 8110: Part 2: 1985 Clause 3.8.4 has stipulated that by limiting temperature differential to 20°C has been successful to avoid early-age thermal cracking, but this apply to gravel type of aggregate at the corresponding restraint factor, \( R = 0.36 \). However, when referring to Table 2 and Table 3 of BS 8110: Part 2: 1985, the granite type of aggregate is typically used in local Malaysia construction industry. This granite aggregate has increased limiting temperature differential of 27.7°C at the same corresponding restraint value, \( R = 0.36 \). This can be conservative value as studies have found that massive pour cast into blinding concrete has typical restraint factor of \( R = 0.1 \) to 0.2 (See Table 3 for values of external restraint recorded in various structures).
Table 2. Values of limiting temperature changes to avoid cracking from BS 8110-2: 1985

| Aggregate type | Thermal expansion coefficient (°C⁻¹) | Tensile strain capacity (10⁻⁶) | Limiting temperature change for varying restraint factor (R) | Limiting temperature differential when R = 0.50 |
|----------------|--------------------------------------|-----------------------------|----------------------------------------------------------|-----------------------------------------------|
| Gravel         | 0.18                                 | 30                          | 9.7                                                     | 29.2                                          |
| Lime           | 0.06                                 | 80                          | 15.4                                                   | 40.0                                          |
| Sintered p.f.a.| 0.10                                 | 110                         | 26.2                                                   | 78.4                                          |

Table 3. Values of external restraint recorded in various structures from BS 8110-2: 1985

| Pour configuration | Restraint factor (R) |
|--------------------|----------------------|
| Thin wall cast on to massive concrete base | 0.6 to 0.8 at base |
| Massive pour cast on to existing mass concrete | 0.3 to 0.4 at base |
| Suspended slabs | 0.2 to 0.4 |
| Infill bays, i.e. rigid restraint | 0.8 to 1.0 |

Owing to the acceptable field data of this kind is not available locally i.e. peak core temperature, limiting differentials and temperature gradient were thus be established for conditions at the job site. This was validated basing on trialled hot-box tests at prescribed initial placing temperature as well as the anticipated concrete section thickness, rather than merely follow seemingly arbitrarily values of 50°C maximum peak core temperature and 20°C maximum allowable core-surface temperature requirements as stipulated in various local project contract documents as experienced by author.

Whereby, the maximum temperature differential is a function of concrete mechanical properties such as CTE, tensile strength, elastic modulus, as well as the size and restraints of the element. Therefore, the established maximum allowable core-surface temperature differential can significantly reduce the amount of time that protective measures, such as surface insulation, must be kept in place. Thermal cracking will occur when contraction due to cooling at the surface causes tensile stresses that exceed the tensile strength of the concrete.

It has to be pointed out, these temperature criteria is merely a general guideline based on experience with unreinforced mass concrete placed in Europe more than 50 years ago. In many situations, limiting temperature difference to 20°C is overly restrictive and thermal cracking may not occur even at higher temperature differences as in the case of for the completed Sg. Kinta roller compacted concrete dam under above-mentioned similar temperature restrictions.

Local experience has also shown that for the case of thin wall cast on to massive concrete base, temperature differential of around 12°C to 14°C may lead to early thermal cracking in lower 1m of wall or higher due high restraint factor, R (0.6 to 0.8). For pilecaps and raft foundation slabs, restraint factor, R is generally low (0.1 to 0.4) and with granite aggregate, temperature differential up to around 35°C to 40°C have been recorded without early thermal cracks.

Thermal modelling requires amongst many input parameters and regardless whether it would be made by means of a British oriented approach such as CIRIA C660 (Bamforth 2007 or ACI 207.2R-07, 2007) or through the application of finite element software such as LUSAS, FEMMASSE, MIDAS, ASTEA-MAX and et cetera whereby the foreseeable temperature rise within the concrete as a function mainly of: (i) member shape, (ii) size and thickness, (iii) environmental conditions, (iv) concrete temperature, (v) properties of cementitious materials type and maturity, as well as dosage,
(vi) formwork type and insulation, (vii) degree of restraint, and et cetera. The scope of the thermal modelling will be mainly to predict the temperature and stress development from overlapping/adjacent mass pours along with the prediction of the spacing and width of cracking induced by the thermal (plus hydraulic and autogenous) shrinkage strains (or actions) to which the tensile strain capacity of the concrete will oppose as a reaction (whereby tensile strain capacity could be approximated by the ratio of concrete flexural strength and its elastic modulus). The CTE value established as per procedure highlighted in below Table 4 and presented in this report could be adopted as important input parameter for thermal modelling analysis whereby the pour thickness, placing interval, casting sequence (time-based temperature profiles) and et cetera of mass concrete structure construction could be assessed and established.

**Table 4. Procedure for the determination of CTE**

| Step | Procedure |
|------|-----------|
| 1    | Selection of appropriate CTE testing method |
| 2    | Procedure to experimentally determine the CTE |
| 3    | Actual project concrete design mix & raw materials used for test specimens casting |
| 4    | Laboratory testing: Mould preparation, instrumentation & samples casting in controlled condition |
| 5    | Data acquisition & observation thermal heat cycles (heating/cooling) |
| 6    | Derivation of equation, $\Delta T_{\text{max}} = 3.663 \frac{\text{Ectu}}{\alpha c}$ |
| 7    | Analysis & evaluation of test results corresponds to strength & temperature results derived from semi-adiabatic temperature rise test |
| 8    | Interpretation of results |
| 9    | Conclusions/recommendations |

2. CONCRETE MIX DESIGN

The physical properties and chemical compositions of the Portland-fly-ash (PFA) cement and concrete mix design used in the study are listed in below Tables 5-7 respectively.

**Table 5. Typical physical properties of PFA Cement (CIMA, 2011 & 2012)**

| CEM II/B-V | EN197-1 – CEM II/B - V 42.5N - LH |
|------------|----------------------------------|
| **Typical Properties** | |
| Blaine fineness (m²/kg) | 420 |
| Specific gravity | 2.9 |
| Initial Setting time (min.) | 157 |
| 2d strength (MPa) | 22 |
| 7d strength (MPa) | 37 |
| 28d strength (MPa) | 50 |

**Table 6. Typical chemical analysis of PFA Cement (CIMA, 2011 & 2012)**

| Chemical Properties | Chemical Composition (%) |
|---------------------|--------------------------|
| SiO₂ | 26.57 |
| Al₂O₃ | 8.84 |
| Description                  | Value |
|------------------------------|-------|
| Fe$_2$O$_3$                  | 5.40  |
| CaO                          | 51.89 |
| MgO                          | 0.63  |
| SO$_3$                       | 2.31  |
| Na$_2$O                      | 0.18  |
| K$_2$O                       | 0.72  |
| Na$_2$O + 0.658 x K$_2$O     | 0.65  |

**Table 7.** Class A Mass Concrete C20 concrete mix design (Mass Concrete Class A, 2011)

3. ESTIMATING THE RISK OF CRACKING AND CRACK-INDUCING STRAIN

3.1. Internal restraint

Internal restraint is a result of differential temperature changes within an element. It may lead to both surface cracking and internal cracking that may not be observed from the surface. The development of cracking due to internal restraint is shown in below Figure 1. As heating occurs, the surface is subject to tensile stresses as the centre of the pour gets hotter and expands to a greater extent. At elevated temperature and early-age, creep is high and the stresses generated are relieved, at least in part. As cooling occurs there is a stress reversal and the surface cracks generally reduce in width. At the same time tension is generated at the centre of the pour as it cools more than the surface and internal cracking may occur (Bamforth 2007).
Figure 1. Schematic representation of the development of crack in a massive element due to temperature differentials assuming no external restraint (such cracking may occur in both the vertical and horizontal orientation) (Bamforth 2007).

Where property data are available for a particular mix, the limiting temperature differential, $\Delta T_{\text{max}}$, i.e., the differential at which cracking may occur, can be calculated using equation 7 (see page 6 of 11) in which with the restraint factor $R = 0.42$ and assuming $K_1 = 0.65$ (for creep) and $K_2 = 0.8$ (for strength reduction due to sustained loading). In a previous investigation the value of $R$ was determined to be 0.36 (Bamforth 1982). This value was derived to match the recommended limiting temperature values with those observed in practice and based on property data available at the time. The adjustment in $R$ has been made to maintain these limits with the application of property data from EN1992-1-1:2004 (Bamforth 2007).

3.2. **Internal restraint**

As demonstrated in literature (Bamforth 2007), the CTE can be determined by putting the concrete through relatively sudden temperature shocks and measuring, by mean of embedded strain gauges, the induced strains. Of course, especially at early ages, thermal deformations will not be the only ones measured. Thus, particular care must be put in excluding all the other strain components. The total deformation that can occur at early ages can be expressed as follow:

$$\varepsilon_{\text{tot}} = \varepsilon_{\text{plastic}} + \varepsilon_{\text{drying}} + \varepsilon_{\text{carbonation}} + \varepsilon_{\text{thermal}} + \varepsilon_{\text{autogenous}}$$

Where:

- $\varepsilon_{\text{plastic}}$: is the shrinkage due to the evaporation of water from the fresh concrete or due to the adsorption by the formwork when it is still at the plastic state. This deformation can be neglected since each sample has been firmly wrapped by mean of polystyrene sheets.

- $\varepsilon_{\text{drying}}$: is the shrinkage caused by the evaporation of water from capillary pores in the hardened concrete. This deformation is also considered to be negligible in this study since evaporation was prevented.

- $\varepsilon_{\text{carbonation}}$: this component of the strain is caused by the reaction of the hydrated cement paste with carbon dioxide present in the atmosphere when humidity is present. It can also be neglected considering the PE wrapping and the fact that it is a typical long term phenomenon.

- $\varepsilon_{\text{thermal}}$: this is the component of the strain object of this study. It must be pointed out that this deformation also includes the thermal strain of the strain gauge, which will have to be subtracted to the measured strain.

- $\varepsilon_{\text{autogenous}}$: is the strain due to the consumption of water during the hydration process. As already mentioned, autogenous shrinkage is not expected to occur for high water-binder ratio
concrete. In this research only Grade 20 mass concrete is analysed hence this deformation is neglected (as autogenous shrinkage is an issue solely deals with high strength, very low water/binder concrete and can be neglected at any stage when mass concrete for dams is considered). Therefore, under experimental conditions of this research, the total deformation measured by the strain gauge corresponding to the thermal shock, during which a temperature variation of $\Delta T$ occurs, can be presented in below Equation 1:

$$\varepsilon_{\text{total}} = \varepsilon_{\text{thermal}} = \alpha_c \Delta T + \alpha_e \Delta T$$  \hspace{1cm} (1)

Where:
- $\alpha_c$ = CTE of concrete, to be determined, $\mu \varepsilon/\degree C$
- $\alpha_e$ = CTE of the steel strain gauge, which equals 12.2 $\mu \varepsilon/\degree C$ [Bamforth 2007].
- $\Delta T$ = amplitude of the thermal cycle °C.

Whereby $\alpha_c$ and $\alpha_e$ are the thermal expansion coefficients of the concrete and steel, respectively, thus:

$$\alpha_c = \frac{\varepsilon_{\text{total}} - \varepsilon_e \Delta T}{\Delta T}$$  \hspace{1cm} (2)

The following paragraphs depicting the condition of cracking potential as well as the approach employed in deriving the internal restraint induced strain for cracking avoidance i.e. $\Delta T_{\text{Max}} = 3.7 \varepsilon_{\text{ctu}} / \alpha_c$. In particularly, the maximum allowable core-surface temperature differential will be determined and the parameters that affect it would be analysed. When a concrete member is subjected to thermally induced volume changes under restrained (internal restraint or external restraint) condition, a certain amount of internal strain is built-up and cracking may occur when the tensile strain capacity of the concrete is exceeded. In mathematical terms, external restraint induced thermal strain can be formulated as follow (Bamforth 2007):

$$\varepsilon_r = K_1 \{[\alpha_c T_1 + \varepsilon_{ca}] R_1 + \alpha_c T_2 R_2 + \varepsilon_{cd} R_3\}$$  \hspace{1cm} (3)

Where:
- $T_1$ - Difference between the peak temperature and the mean ambient temperature
- $T_2$ - Long-term fall in temperature which takes into account the time of year at which concrete was cast
- $\alpha_c$ - Coefficient of thermal expansion of concrete
- $\varepsilon_{ca}$ - Autogenous shrinkage
- $\varepsilon_{cd}$ - Drying shrinkage
- $R_1$ - Restraint factor applies for early-age thermal cycle
- $R_2$, $R_3$ - Restraint factors applying to long-term thermal movement and drying shrinkage
- $K_1$ - Coefficient for the effect of stress relaxation due to creep under sustained loading (= 0.65)

As far as massive concrete structures are concerned, as in the case for Tembat Hydropower Dam construction, internal restraint becomes the bottleneck condition for thermal cracking. In this case, it is the core-surface temperature differential that can cause concrete to crack. Indeed, the core is prevented from exchanging heat with the surroundings, thus remaining hot for long time, while the surface tends to reach equilibrium with the environment and thus cooling. At this point, the following assumptions must be pointed out:

- Considering the concrete mix design, autogenous shrinkage can be neglected, as evidences in literature (Kada et al. 2002) that this phenomenon occurs only for low water/binder ratio {typically below 0.4} concrete {i.e. for high to very high strength concrete};
- Drying shrinkage plays a major role in the medium and long-term deformations and can be neglected for early stage determination of total strain;
- Restraint factors for external restraints are negligible for thick sections where – as mentioned above – internal restraint is dominant;
• Ignoring the long-term effects of temperature, T2 in consideration of early-age thermal cracking as in this case.

Taking into above consideration, the internal restraint induced strain is given by:

\[ \varepsilon_r = k_1 \Delta T \alpha_c R \]  

(4)

Therefore, the risk of cracking and crack width should be assessed at both early-age and in the long-term. For the early-age calculation of \( \varepsilon_r \) using above equation 4, include only the terms for early-age thermal contraction at three days, i.e. assume that \( T_2, \varepsilon_{ca}, \) and \( \varepsilon_{cd} \) are zero. Where \( \Delta T \) now becomes the core-surface temperature differential of the concrete element, the condition for cracking is:

\[ \varepsilon_r > \varepsilon_{ctu} \]  

(5)

Where, \( \varepsilon_{ctu} \) is the tensile strain capacity of the concrete under sustained loading. Thus it is assumed that to avoid cracking, the restrained-strain \( \varepsilon_r \) should be less than the tensile strain capacity \( \varepsilon_{ctu} \) of the concrete. The tensile strain capacity of plain concrete represents its ability to elongate before rupture and could be approximated by the ratio between the tensile strength of concrete and its elastic modulus (Bamforth 2007). Rearranging equation 4 and equation 5 for the condition of cracking, \( \varepsilon_r = \varepsilon_{ctu} \), then equation 4 becomes:

\[ T_{1,\text{max}} = \frac{\varepsilon_{ctu}}{k_1 \alpha_c R} \]  

(6)

For the condition of internal restraint it has been estimated that \( R = 0.42 \) and \( K_1 = 0.65 \), it can be defined that the quick check for internal restraint induced strain for avoidance of cracking is expressed by (Bamforth 2007):

\[ \Delta T_{\text{max}} = 3.7 \frac{\varepsilon_{ctu}}{\alpha_c} \]  

(7)

In summary, restrained volume changes can be caused by the superimposed effect of:

• Restricted (internal, external) thermal expansion/contraction cycles;
• Shrinkage (early, late).

Therefore any thermal modelling phase should attribute appropriate input parameters in order to closely model thermal behaviour and maximum strain developed by a certain structure.

3.3. Temperature Rise and Temperature Differential (\( T_{1,\text{max}}, \Delta T_{\text{max}} \))

For the control of early-age thermal cracking, there are two important temperature considerations:

• The temperature rise above that of the adjoining concrete or substrate. It depends on the location of the point being considered and is applicable to conditions of external restraint {i.e. casting a wall onto a rigid mature base};
• The maximum temperature differential and the thermal gradient within the section. This is the condition of internal restraint.

In the calculation of restrained early-age temperature movements in concrete, it is necessary to define the basis upon which the temperature drop from the peak temperature is calculated. This requires a definition of the placing temperature of the concrete, the maximum temperature rise and the temperature to which the element will return after cool down, see below Figure 2.

![Figure 2. Typical heat development’s plot of a concrete member](image-url)
The placing temperature is defined as the temperature of the concrete when placed into the forms. This may differ from the temperature at the mixing plant. In this analysis, the maximum value for the project concrete at the delivery point is assumed to be 23°C {based on data gathered during mock ups} (Mass Concrete Class A 2011). The temperature rise is assumed to be the difference between the placing temperature and the maximum temperature achieved during the early-age thermal cycle.

The temperature rise depends upon the rate at which heat is generated, the total heat output and the rate of cooling. The heat generating characteristics are influenced by both the binder chemistry and its fineness. In thin sections, the rate of heat evolution is dominant in determining the temperature rise. In massive sections, where the heat loss is relatively slow compared with the heat being generated, the temperature rise is more dependent on the total heat evolved. The main factors governing the temperature rise and fall are:

- Binder content & types and sources of cementitious material;
- Concrete placing temperature & section thickness;
- Formwork and insulation and its time of removal;
- Ambient conditions.

The temperature drop, $T_1$, which determines the thermal contraction during cool down, is defined as the difference between the peak temperature and the mean ambient temperature and for the local conditions of average environmental temperature of 27°C of Hulu Terengganu can be also taken to equivalent to the temperature rise with the only difference that during temperature rise heat is built-up relatively quick while it would be desirable to limit to a maximum rate at the cooling phase. The $T_1$ values are based on modelling using semi-adiabatic data (BS EN196-9) derived from earlier test (Mass Concrete Class A 2011). As only the binder produces heat of hydration, the higher the binder content, the greater the heat evolved per unit volume and the greater the temperature rise. Along with the cement, combination and addition types the binder content has a major effect on the temperature rise.

3.4. Determination of Maximum Allowable Core–Surface Temperature Differentials

The mechanical properties of 20MPa mass concrete are shown in below Table 8.

As demonstrated during mock-up tests (Mass Concrete Class A 2011), the core reaches its maximum temperature at about 72 hours. It is therefore reasonable to assume that the maximum core-surface temperature differential occurs in the same period within the actual lifts. Hence, it is necessary to estimate the maximum allowable tensile strain $\varepsilon_{ctu}$ at early ages {in particular, at 3 days}. The tensile strain capacity $\varepsilon_{ctu}$ is the maximum strain that the concrete can withstand without a continuous crack forming. The aforesaid $\varepsilon_{ctu}$ can be measured directly or derived from measurements of the tensile strength and the elastic modulus of the concrete at 3 days. Therefore, these two variables must be estimated as well for correlation. Concrete tensile strength: at 3 days calculated as 68% of the tensile strength at 7 days available in (Bamforth 2007) as shown in below Tables 9-10:

| Table 8. Concrete compressive strength, tensile strength and elastic modulus test results for 20MPa mass concrete (Mass Concrete Class A 2007) |
|---------------------------------------------------------------|
| Description                  | Duration   |
|-------------------------------|------------|
| Stage 1-Hox Box 1             | 7 day      |
| Compressive Strength (MPa)    | 25.25      |
| Tensile Strength (MPa)        | 1.70       |
| Modulus of Elasticity (MPa)   | 19.25      |
| 28 day                        | 33.05      |
| 38.15                         | 2.43       |
| 28.90                         | 2.81       |
| 56 day                        | 25.60      |

This assumption is validated by the percentage increase of concrete tensile strength shown in below Table 9.
Table 9: Values of early-age and long-term tensile strength estimated in accordance with EN 1992-1-1: 2004

| Strength class | Early-age tensile strength $f_{ctm}(3)$ (MPa) | Long-term tensile strength $f_{cm}$ (MPa) |
|----------------|---------------------------------------------|-----------------------------------------|
| C20/25         | 1.16                                        | 2.21                                    |
| C25/30         | 1.30                                        | 2.56                                    |
| C30/37         | 1.52                                        | 2.90                                    |
| C35/40         | 1.72                                        | 3.30                                    |
| C40/50         | 1.83                                        | 3.77                                    |
| C45/55         | 2.06                                        | 4.00                                    |
| C50/60         | 2.27                                        | 4.38                                    |
| C55/67         | 2.47                                        | 4.77                                    |
| C60/75         | 2.67                                        | 5.15                                    |

Table 10. In-situ concrete tensile strength at 3 and 28 days – EN 1992-1-1: 2004

| Item     | Early-age (3 days) | Long-term (28 days) |
|----------|--------------------|----------------------|
|          | In-situ values     | ENI 992-1-1          |
| Strength class | 95% | ENI 992-1-1 | 95% | 99% |
| C20/25   | 1.16             | 1.30                 | 1.16 | 1.27 |
| C25/30   | 1.30             | 1.46                 | 1.33 | 1.49 |
| C30/37   | 1.52             | 1.70                 | 1.73 | 1.99 |
| C35/40   | 1.72             | 1.94                 | 1.98 | 2.24 |
| C40/50   | 1.83             | 2.05                 | 2.12 | 2.38 |
| C45/55   | 1.95             | 2.17                 | 2.26 | 2.52 |
| C50/60   | 2.06             | 2.34                 | 2.39 | 2.65 |
| C55/67   | 2.15             | 2.44                 | 2.51 | 2.77 |
| C60/75   | 2.27             | 2.52                 | 2.62 | 2.94 |

Therefore, it can be seen that the assumed value of 1.16 MPa (0.68x1.7Mpa) for tensile strength at 3 days is reasonable.

Concrete Young modulus: calculated as 90% of the Young modulus at 7 days available in [BS8110-2: 1985]. This assumption is validated by the increase of the concrete Young modulus as shown in below Figure 3.

Figure 3. Increase of the Young modulus with time

In this case, a conservative value of 17.33GPa for the Young modulus at 3 days has been assumed. From the aforesaid determined concrete Young modulus and tensile strength, by applying equation 8, it is thus possible to calculate the maximum allowable tensile strain at 3 days as:

$$\varepsilon_{ctu} = f_{ctm} \times 10^6 \mu e$$

$$\varepsilon_{ctu} (3 \text{ days}) = f_{cm} (3 \text{ days}) / E_{cm} (3 \text{ days}) = 66.96 \mu e$$

Considering the value reported in below Table 11 for granite aggregate ($\varepsilon_{ctu}$ @ 3 days = 75 $\mu e$, as extracted from [Bamforth 2007], the strain capacity for concrete aged at 3 days could then be estimated. This value is in line with the requirements stipulated in the literature (Bamforth 2007).
\[ \varepsilon_{ctu}(f_{ck,cube} = 25) = \varepsilon_{ctu}(f_{ck,cube} = 37) \times \left[ 0.63 + \left( \frac{25}{100} \right) \right] \]  

(9)

By using above equation 9, we obtain \( \varepsilon_{ctu} = 66.19 \mu\varepsilon \). At this stage, by adopting above \( \varepsilon_{ctu} \), \( \Delta T_{max} \) could be calculated by means of \( \Delta T_{max} = 3.7 \frac{\varepsilon_{ctu}}{\alpha_c} \).

Hence, it is possible to calculate the maximum allowable core-surface temperature differential for different values of the coefficient of thermal expansion (CTE). Referring to Table 14 \{(8.648+8.834)/2 = 8.741, Says 9.0 \( \mu\varepsilon/\degree C \)} and by applying above Equation-8, for a CTE of 9.0 microstrain/\degree C (Aged 3 days) as experimentally determined, the maximum differential is 27.53\degree C, as described in below Figure 4.

**Table 11.** Estimated values of \( \varepsilon_{ctu} \) for strength class C30/37 under sustained short-term loading using different aggregate types in accordance with EN 1992-1-1: 2004

| Aggregate type                  | Estimated \( \varepsilon_{ctu} \) under sustained loading for strength class C30/37 |
|--------------------------------|-------------------------------------------------------------------------------------|
|                                | Early-age 3 days | Long-term 28 days                                                                 |
| Basalt                         | 63               | 90                                                                                 |
| Flint gravel                   | 86               | 108                                                                                |
| Quartzite                      | 76               | 108                                                                                |
| Granite, gabroiro             | 75               | 106                                                                                |
| Limestone, dolomite           | 86               | 122                                                                                |
| Sandstone                      | 90               | 155                                                                                |
| Lightweight aggregate          | 115              | 165                                                                                |

The above calculated value of maximum temperature differential is perfectly in line with the published value i.e. 28\degree C as stipulated in BS 8110: Part 2 - Table 3.2 \{See Table 3\} and EN 1992-1-1: 2004 which provides estimates of temperature differentials applicable for a variety of conditions of external restraint and for internal restraint due to temperature gradients as shown in below Table 12.

**Table 12.** Limiting temperature drop \( T_{1,\text{max}} \) and temperature differentials \( \Delta T_{max} \), to avoid early-age cracking, based on assumed typical values of \( \alpha_c \) and \( \varepsilon_{ctu} \) as affected by aggregate type (for \( K_1 = 0.65 \) and \( \varepsilon_{ca} = 15\mu\varepsilon \)} [Bamforth 2007]

![Figure 4. Maximum allowable core-surface temperature differentials as a function of the CTE; For a CTE of 9.0 as experimentally determined, the maximum differential is 27.53\degree C](image-url)
4. EXPERIMENTALLY DETERMINATION OF COEFFICIENT OF THERMAL EXPANSION (CTE)

Even though in literature (Bamforth 2007) it is possible to find data regarding the concrete CTE for different kinds of aggregates {see Table 1}, these are more deem-to-satisfy values provided when an experimental determination of the CTE cannot be performed. The design values in Table 1 may overestimate the actual value of the CTE, causing a large and unnecessary decrease in maximum allowable core-surface temperature differential. In the followings, an experimental procedure to accurately calculate the real CTE value will be proposed based on (Kada et al. 2002).

4.1. Experimental Procedure and Set-up

An identical concrete mix design previously adopted for the hot box trials as shown in Table 7. Class A Mass Concrete C20 Concrete Mix Design was applied on the casting of CTE block samples. The cement content and water/cement ratio were 200 kg/m3 and 0.70 respectively. The CTE was carried out under controlled laboratory conditions in accordance with the established procedure [Kada et al. 2002]. For the establishment of CTE value for grade 20MPa granite concrete, six (6) specimens of 0.2m x 0.2m x 0.4m concrete block {See below Figure 6} was cast whereby two results for each age were be made available so that a mean value will be calculated. The procedure of obtaining an accurate value of the CTE for the mix design under consideration is summarized below:

- Preparation of moulds;
- Samples casting and ageing;
- Subjection of samples to a thermal cycle (cooling/heating/cooling);
- Data acquisition and interpretation;
- Calculation of the CTE value.

Age of testing will be chosen based on the following criteria:

i. Before 24 hours concrete dimensions are dominated by stress relief by creep, hence no cracking would occur.

ii. After 7 days the value of CTE will tend to be constant whereby between 1 day and 7 days CTE is expected to be time dependent (decreasing in value with time)

iii. Based on the thermal data obtained during the hot box test the most critical age of for thermal cracking will be evaluated and at that age a suitable value of CTE will be chosen as input parameter for calculating of maximum differentials acceptable for the concrete under condition of internal restraint which is the dominating restraint for thick sections.
Therefore, the determination of the coefficient of thermal expansion (CTE) will be based on 1-Day, 3-Days, and 7-Days respectively.

![Figure 5. The ST4 Vibrating Wire Embedment Strain Gauge used for measuring strain in mass concrete (ST4 VWESG from itmsoil, UK).](image)

**Figure 5.** The ST4 Vibrating Wire Embedment Strain Gauge used for measuring strain in mass concrete (ST4 VWESG from itmsoil, UK).

**4.1.1. Methodology**

1) Test was conducted on all six specimens using methodology proposed by Kada et al. 2002, for which the specimens were further divided into two specimens each to facilitate three different ages of testing and monitoring.

2) The samples were cast in steel moulds of 200 x 200 x 400mm and a vibrating wire extensometer was installed at the center of the sample along its longitudinal axis before concrete placement in the mould (For this case the height will be about 3 times the maximum aggregate size, hence 3 x 65mm MSA = 195mm, Says = 200mm).

3) Each strain gauge has then been tied to the steel frame by mean of plastic cable ties and connected to a remote data acquiring system. Finally, a thermocouple has been attached to the strain gauge in order to measure the core temperature of the samples.

4) The extensometer is connected to a data acquisition system that monitored the strain and temperature variations within the specimen.

5) Subsequently, concrete has been cast inside the moulds.

6) The specimen is demoulded just as the concrete has been set and reached the right age (right consistency for demoulding).

7) The samples obtained were then wrapped in plastic bags to prevent from hygrometric exchange with environment (prevent water evaporation). Thus allowing excluding different components of the total strain from the measured strain as described in preceding paragraph.

8) Each sample was then immersed in a heat-controlled water bath tank which then subject to alternate heat cycle at temperatures of 50°C±3°C and 10°C±3°C, respectively.

9) Each one of the samples is submitted to a thermal shock in the order of 40°C when stable temperature is reached within samples.

10) The samples from the 50°C±3°C is then switched to the 10°C±3°C water bath and vice versa until the intended duration of test is achieved for all 6 specimens.

11) In order to determine the CTE values at 1 Day, 3 days and 7 days, 2 samples for each aging time have been tested. Identification codes for such samples are shown in Table 13.
12) All the specimens are subjected to hours of alternating heat cycles (cooling/heating/cooling) which separating each thermal shock as illustrated in Table 14, 15 and 16: Duration of Heating and Cooling Cycles for 24 hours (1 Day), 72 hours (2 Days) and 168 hours (7 Days).

### Table 13. Samples identification code and aging time

| Sample Identification | Sample Aging |
|-----------------------|--------------|
|                       | 24 Hour | 3-Days | 7-Days |
| SG1                   | √        |         |        |
| SG2                   |          | √        |        |
| SG3                   |          |          | √        |
| SG4                   |          |          | √        |
| SG5                   |          |          |          |
| SG6                   |          |          |          |

Note:* In order to measure very accurately the deformation induced by thermal cycles, it has been decided to construct steel frames to be firmly screwed to each of the casting mould as a rigid support for the strain gauge (see below Figures 7a-7b).

Figure 7. Front and top view of the system composed by steel frame and strain gauge inside the mould [7a and 7b showing the installation of ST4 vibrating wire embedment strain gauge. Size of specimen is 200mm x 200mm x 400mm]

Subsequently, the samples obtained were then wrapped in plastic bags to prevent from hygrometric exchange with environment (prevent water evaporation) as illustrated in below Figures 8a-8b.

Figure 8. CTE specimens using Grade 20MPa [8a and 8b indicating the casting of CTE specimens using Grade 20MPa granite concrete and wrapped up with multilayer plastic sheet prior to CTE testing.(Samples as casted and after wrapping)]

### 5. THERMAL CYCLE

At the end of each thermal cycle step, the CTE can be determined as follow:
From above expression, it can be noted that it is possible to determine one CTE for the cooling phase and one CTE for the heating phase. These two values should coincide, at least theoretically. Experimentally, the closer these two values are, the more accurately the test has been performed. The below Figure 9 showing sample that have been put through a thermal cycle.

Thermal cycle shown in above Figure 9 refers to sample SG3. Anyway, all the other samples have been put through an identical thermal cycle. To reach the temperatures of 10°C and 50°C a chilled bath (chilled with ice cubes) and a hot bath (with a thermostat able to keep temperature constant) have been used, respectively (see below Figures 10a-10b).

An alarm (see below Figure 11) has been set to ring when the core temperature of the sample (measured by the thermocouple fixed to the strain gauge) reaches 10±3°C in the cooling phase or 50±3°C in the heating phase). When this happens, the sample is immediately removed from one bath and put in the other one, depending on the thermal cycle period according to above Figure 9. Finally, to prevent water to flow inside the sample, samples themselves have been further wrapped with thick plastic bags (black colour) as shown in below Figures 11a-11b.
6. INTERPRETATION OF TEST RESULTS & ANALYSIS

The results for test performed on samples aged 24 hrs, 3 days and 7 days (Raw Data for CTE Test 2013), are shown in below Table 14. It must be pointed out that, because the test duration is about 24 hours, the age of samples at the time of CTE measurement will be of 2 days, 4 days and 8 days, respectively.

Table 14: The values of CTE recorded for heating and cooling cycles for each specimen during the effective age of concrete at 2 days, 4 days and 8 days

| SG   | Heating (με/°C) | Cooling (με/°C) | CTE average (με/°C) | Effective Age of Concrete | CTE average (με/°C) |
|------|----------------|----------------|---------------------|--------------------------|---------------------|
| SG1  | 8.882          | 8.764          | 8.823               | 2-Days                   | 8.858               |
| SG2  | 8.825          | 8.958          | 8.892               |                          |                     |
| SG3  | 8.764          | 8.531          | 8.648               | 4-Days                   | 8.741               |
| SG4  | 9.116          | 8.552          | 8.834               |                          |                     |
| SG5  | 8.954          | 8.349          | 8.652               | 8-Days                   | 8.924               |
| SG6  | 9.570          | 8.822          | 9.196               |                          |                     |

From the above analysis, it is possible to deduce that the CTE remains virtually constant after 48 hours from samples casting. For the design purpose, the value of the CTE will take as 9.0με/°C [(8.648+8.834) = 8.741, Says = 9.0με/°C] between the period of 2-days and 8 days. Hence the CTE value at 3 days to be used for calculation.

6.1. Interpretation of r and r2 for Concrete at Effective Age at 4-Days

As per analysis shown in above Table 15, it could be concluded that the r & r2 for heating and cooling cycles for respective SG1, SG2, SG3, SG4, SG5 and SG6 for concrete aged at 3-days is of about 99.48% versus 99.87% and 98.96% versus 99.73% respectively, in agreement is achieved between the recorded temperature and strain values which implied strong linear relationship.
Table 15. Coefficients, r & r² at Age of 4-Days

| Item | Cooling r | Heating r | Cooling r² | Heating r² |
|------|-----------|-----------|------------|------------|
| SG1  | 0.9876    | 0.9959    | 0.9754     | 0.9919     |
| SG2  | 0.9966    | 0.9980    | 0.9932     | 0.9960     |
| SG3  | 0.9985    | 0.9989    | 0.9971     | 0.9979     |
| SG4  | 0.9973    | 0.9995    | 0.9947     | 0.9989     |
| SG5  | 0.9934    | 0.9998    | 0.9869     | 0.9996     |
| SG6  | 0.9950    | 0.9998    | 0.9901     | 0.9996     |
| Mean | 0.9948    | 0.9987    | 0.9896     | 0.9973     |
| %    | 99.48     | 99.87     | 98.96      | 99.73      |

7. DISCUSSIONS
1. It has to be pointed out that variation of coefficient of thermal expansion occurred on all six specimens during the initial hours of strain measurements while concrete is having effective age of 1-day and whereby the strain is significantly dependent on the age of testing following concrete casting. The CTE values will gradually decreasing until the end of concrete setting and remain constant thereafter which implied that appropriate crack control and protection measure need to be taken in order to avoid early-age thermal cracking.
2. From the CTE analysis, CTE values remains virtually constant after 48 hrs from samples casting. The average CTE value for samples SG3 and SG4 cured for 72 hours and tested during a 72 hours cycle (hence for a resulting concrete effective age of 4 days) can be taken as 8.741 με/°C. For design purpose, the CTE value of granite aggregate could then be assumed as 9.0 με/°C between 2 and 8 days.
3. Besides thermal deformation, this proven test method is rather accurate way in eliminating any possible superimposed effect of others early-age thermal deformations that can occur during early-age strain measurements such as autogenous, plastic, drying and carbonation shrinkages.
4. Therefore, experimental determined value of CTE of a concrete mixture is of paramount importance in avoiding variation in CTE owing to a change in aggregate types. In contrast, any investigation aiming at evaluating early-age strain development in high strength concrete should into consideration of the sensitivity of other thermal deformations such as autogenous, drying and plastic shrinkages to the total thermal expansion coefficient variations during concrete hardening.
5. From the established statistical analysis of r and r², both coefficients showing very strong positive linear correlation as r and r² is very close to unity i.e. 1. This could be observed that CTE value is virtually identical for both samples (SG1 and SG2) tested and regardless of the heating/cooling phase, meaning that the test is repeatable and test conditions have been controlled accurately.

8. CONCLUSIONS
6. Based on the actual CTE value of granite aggregate of 9.0 με/°C, it could be inferred that a simple CTE test method [Kada et al. 2002] is proven effective in determining early-age value of CTE, as compare to CTE value of concrete made with granite aggregate which varied from 8.1-10.3με/°C [CIRIA C660 and BSEN 1991-1-1:2004]. Therefore, the CTE value derived from this test is perfectly coherent (consistent/harmonious) with data available in scientific literature for concrete containing large volumes of granite aggregates [BSEN 1991-1-1:2004 and TxDOT2011].
7. Maximum allowable core-surface differential for the present concrete mix design with design CTE of 9.0 με/°C → 27.53°C, which is perfectly in accordance with the published values as stipulated in Bamforth 2007 and BS 8110-2: 1985.

8. Recommendations on optimum fresh concrete placing temperature, core-surface temperature, and maximum peak concrete core temperature, could be further verified and validated during further thermal modeling studies using suitable finite element software such as LUSAS and MIDAS FEA for heat of hydration and thermal stress analyses, with the experimental determined input parameters established in this paper along with other key parameters such as maximum adiabatic temperature rise data, convection heat transfer coefficients, environmental temperature and et cetera.

References

[1] ACI Committee 207.2R-07, 2007, ‘Report on Thermal and Volume Change Effects on Cracking of Mass Concrete.’

[2] BS 8110-2: 1985, Structural Use of Concrete – Part 2: Code of Practice for Special Circumstances. BSI, London

[3] BS EN 1992-1-1:2004 Eurocode 2: Design of concrete structures - Part 1-1: General Rules and Rules for Buildings. BSI, London

[4] BS EN 196-9: 2010, Methods of Testing Cement – Heat of Hydration - Semi Adiabatic Method. BSI, London

[5] Bamforth P. B, 2007, ‘Early-Age Thermal Crack Control in Concrete’, CIRIA C660.

[6] CIMA Cement Test Certificates on NS Ecocrete-LH, CEM II/ B-V 42.5N-LH (January 2011 and June 2012) for 265MW TNB Hulu Terengganu Hydroelectric Project, Malaysia.

[7] Jahangirmejad, S Buch, and Kravchenko A, 2009, ‘Evaluation of Coefficient of Thermal Expansion Test Protocol and Its Impact on Jointed Concrete Performance’, ACI Material Journal, Vol 106 (1), 64-71.

[8] Kada et al 2002, ‘Determination of the Coefficient of Thermal Expansion of High Performance Concrete from Initial Setting’, Materials and Structures, Vol.35: 35-41.

[9] Mallela, J Abbas, Harman T, Rao, C Liu and Darter, M I, 2005, Measurement and Significance of Coefficient of Thermal Expansion of Concrete in Rigid Pavement Design', Transportation Research Record 1919, Transportation Research Board, Washington DC, 38-46.

[10] Mass concrete Class A/Grade III Concrete Mix Trials and Hot Block Monitoring, Overview of Results, Report 1202/11/7322 (2011) for 265MW TNB Hulu Terengganu Hydroelectric Project, Malaysia.

[11] Naik. T.R. et al 2011, ‘Influence of Types of Coarse Aggregates on the Coefficient of Thermal Expansion of Concrete’, Journal of Materials in Civil Engineering, Vol.23: 467 & 472.

[12] Ndon U J, Bergeson K L 1995, Thermal Expansion of Concretes: Case Study in Iowa, Journal of Materials in Civil Engineering, Vol 7(4), 246-251

[13] Raw data for Coefficient of Thermal Expansion Test Result (from 2/7/2013 to 9/7/2013) for 265MW TNB Hulu Terengganu Hydroelectric Project, Malaysia.

[14] ST4 Vibrating Wire Embedment Strain Gauge from itmsoil, United Kingdom for 265MW TNB Hulu Terengganu Hydroelectric Project, Malaysia.

[15] TxDOT Designation, Tex-428-A (June 2011). Test Procedure for Determining the Coefficient of Thermal Expansion of Concrete.

[16] Won M 2005 Improvements of Testing Procedures for Concrete Coefficient of Thermal Expansion, Transportation Research Record 1919, Transportation Research Board, Washington DC, 23-28.