CONCRETE TEMPERATURES, A REVIEW OF REQUIREMENTS, SPECIFICATION, AND RECENT CASE STUDIES

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SUMMARY

High concrete temperatures during the hardening process can pose short and long term risks to the integrity of the concrete. Guidance on the requirements for concrete temperature monitoring and temperature limits in New Zealand is limited and there appears to be no defined standard on what is an acceptable temperature limit. Engineers are often required to seek specialist guidance within New Zealand to try and understand what standard practice is and establish their own understanding and basis for concrete temperature requirements.

This paper focuses on mass concrete temperatures and discusses why concrete temperature and temperature monitoring is important, it outlines the key risks associated with high concrete temperatures during concrete hardening, reviews New Zealand and international literature; covers recent case studies in New Zealand, including the methods and results of large concrete pours and includes key criteria to assist engineers in specifying requirements for future mass concrete projects.

INTRODUCTION

Recent project experience has identified a lack of clear guidance on what are deemed to be acceptable temperatures in concrete which occur as a result of heat of hydration and the hardening process of concrete. Research has been undertaken within New Zealand and International literature to determine an acceptable industry standard which could be adopted by others to apply in practice.

It is well known that concrete temperatures rise occurring the during hardening process, generally within the first 4 days of the concretes life, as a result of the heat generated by the hydration process. This paper reviews commonly available literature which most engineers would be expected to use in practice, and identifies their approach to temperature control in concrete, and more specifically mass concrete. The key risks associated with high temperatures are then discussed, and five case studies in civil structures are reviewed and the results of the temperature monitoring discussed and compared to predicted temperatures using theoretical methods. Finally, a summary of key findings and recommendations is presented which with aim of assisting future concrete designers in New Zealand.
CAUSES OF HEAT DURING THE HARDENING PROCESS

The temperature of concrete during the hardening process is critical to the performance of the concrete. The primary source of temperature rise in concrete is hydration of the cement which produces heat. The other primary source of heat that directly effects the maximum temperature in concrete is the ambient temperature of the fresh concrete. A higher fresh concrete temperature results in higher peak temperature in the hardening process. The temperature of concrete at which it is placed and cured has a direct effect on the rapidness of strength gain, concrete strength properties, durability of the concrete; and the short term and long term strength of the concrete. As the temperature of concrete directly effects the above listed key performance outcomes it is critical that designers know what the acceptable limits are when specifying concrete to ensure the desired outcome.

REVIEW OF LITERATURE

New Zealand Literature

Three main concrete design guides and standards in New Zealand were reviewed to establish a baseline of the requirements of temperatures for concrete construction. The standards were; NZS 3109 Concrete Construction, NZS 3101 Concrete Structures, and the CCANZ New Zealand Guide to Concrete Construction. A summary of the review is presented below, and focuses around how the literature address concrete temperature requirements during the hardening process.

NZS 3109 – Concrete Construction addresses temperature requirements in clause 7.2, unfavourable conditions, and states that concrete should not be placed in temperatures below 5degrees Celsius, nor in excessively hot and dry conditions. The commentary additionally adds that hot weather concreting results in rapid hydration of the cement, and hence early stiffening of the concrete and so on, reduced strength, and a number of other detrimental effects high temperatures can have on the concrete. These detrimental effects will be elaborated on later on in this paper. It recommends an upper practical limit of 30degrees Celsius as the maximum ambient temperature for placing fresh concrete.

The NZS 3109 commentary highlights the importance of temperature rise due to heat of hydration in mass concrete, however it leaves determination of what is an acceptable temperature rise to the designer. It does indicate thermal shock (i.e. large difference in temperature between different areas in mass concrete) can cause detrimental effects on the concrete if not considered.

The standard does not propose that temperatures need to be limited to any certain value and does not give a reference or guidance on where to look for what is deemed acceptable.

Upon review of NZS3101 it is noted that the standard does not cover the design for the effects of temperature change or differential temperature. It also doesn’t specify requirements for designing for the effects of heat of hydration. Not designing for the effects of heat of hydration is identified as a particular issue when thick concrete elements are cast as second-stage construction and their thermal movements are restrained by previous construction. NZS 3101 does require that heat of hydration effects to be considered in the control of cracking that could lead to a loss of serviceability, but does not give a method for determining this. A review of AS5100.5 also identified it does not outline procedures for analysing heat of hydration effects (Kirkcaldie and Wood, 2008).
The Cement and Concrete Association of New Zealand (CCANZ) published a New Zealand Guide to Concrete Construction (CCANZ, 2010). The guide covers the design, supply, and construction practices for concrete in New Zealand. Upon review the guide gives information on types of cements, and indicates low heat rise cements should be used in massive structures, but it does not mention temperature limits, or give guidance on what limits should be specified for mass concrete. More broadly the guide specifies, ‘Unless otherwise specified, concrete at the point of delivery should have a temperature of not less than 5°C nor more than 35°C’. This value is relatively high compared to international guidance, especially considering mass concrete. The guide mentions limitation of temperature rise during hardening as having an effect on the compressive strength, but does not cover any requirements for it. Additionally, the guide covers construction considerations for hot and cold weather concrete pouring, but does not propose any temperatures limits, or discuss and the effects of differential and peak temperatures, and the effects temperatures can have the on long term performance of the concrete (CCANZ, 2010).

The above mentioned guidance documents do not appear to provide clear guidance on what is required with respect to temperature limits to ensure concrete performs as the designer is expecting it to. It simply points out it should be considered, but no New Zealand Standard appears to give sufficient guidance to easily enable a designer to be confident in specifying requirements.

International Research

Research of international best practice relating to concrete temperature limits was conducted to determine what acceptable temperature limits should be imposed on concrete during the hardening process.

Research identified that there are three major international concrete research groups which are the Portland Cement Association in the USA, the Concrete Society, and the Construction Industry Research and Information Association (CIRIA), in the UK. The key documents and research organisations which were identified in the research were Ciria guide C660, ACI 207, ACI 301, and a number of United States Department of Transportation (DOT), in particular Texas DOT and Georgia DOT.

The above mentioned publications all define mass concrete and the need for temperature limits and elaborate further on this topic, which New Zealand guidance does not. An appropriate definition of ‘mass concrete’ is defined by the American Concrete Institute as: “any volume of concrete in which a combination of dimensions of the member being cast, the boundary conditions, the characteristics of the concrete mixture, and the ambient conditions can lead to undesirable thermal stresses, cracking, deleterious chemical reactions, or reduction in the long-term strength as a result of elevated concrete temperature due to heat from hydration.” (ACI 207.1R). Each of the publications reviewed, proposed a limit for peak and differential temperatures. Each were in agreement within a few degrees of the limit specified below. In the author’s opinion, the Texas DOT requirements are most appropriate and are summarised as below, ‘Specification 420’ limits mass concrete placement temperature to 24 °C the maximum in-place temperature to 71 °C with the purpose of avoiding DEF, and the maximum temperature difference in the concrete to 20 °C. The research review highlighted that recent amendments, (within the past two years), to the Texas and Georgia DOT standards were amended to specify that “Mass concrete shall conform to the concrete acceptance criteria and the temperature requirements….to prevent delayed ettringite formation (DEF) and thermally induced stress cracks.” TxDOT (2014),
The UK ‘Concrete Society’ defines Delayed ettringite formation (DEF) as “expansion and cracking of concrete associated with the delayed formation of the mineral ettringite which is a normal product of early cement hydration. DEF is a result of high early temperatures (above 70°C – 80°C) in the concrete which prevents the normal formation of ettringite.” (UK Concrete Society).

DEF-induced damage is a complex and uncommon phenomenon in concrete and is not well understood by the concrete industry currently. There have been many papers published regarding the phenomenon, and most indicate to imposing a 70°C concrete temperature limit to address the issue. Papers worldwide document research on factors which contribute to the occurrence of DEF, and these indicate that temperatures above 70°C may be acceptable provided the chemical make-up of the concrete meets proposed limits and other specific details are addressed. However, for the purposes of this paper, this will not be discussed in detail, as there does not appear to be a defined requirement to fully suppress the phenomenon. Reference is made to two papers by (Bruno Godart and Loïc Divet, 2013), and (Mak, 2012) for proposed DEF prevention strategies. It is clear that DEF can be prevented by limiting the internal concrete temperature to 70°C during its very early life.

**CONCRETE PROPERTIES WHEN SUBJECT TO HIGH TEMPERATURE**

The temperature of concrete during the hardening process has a direct impact on many of the properties which effect the short and long term performance and integrity of the concrete. When concrete is subjected to high temperatures during the hardening process, two physical effects are directly observed, a reduction in workability, and the concrete begins to set faster. It is common to heat cure precast elements to increase the rapidity of strength gain in concrete. Workers using fresh concrete directly observe these two effects on the concrete when the concrete is subject to elevated temperatures. However, effects which are not directly observed during concrete placing are not as widely understood and these other effects become apparent with time. Engineers and specifiers of concrete need to be aware of these non-direct visual effects. Concrete temperatures over 70°C have the possibility of causing DEF. High differential temperatures can cause thermal shock. High curing temperatures can increase the permeability of the concrete, thereby impacting long term durability. High peak and differential temperatures in concrete their effects are reviewed in more detail below.

**Peak Temperature**

As discussed earlier, a more recently internationally recognised negative outcome to the performance of concrete associated with high temperatures during the concrete hardening process is DEF. DEF has also been shown to cause durability problems when concrete is cured at elevated temperatures. The rate of temperature increase, duration of the induction period, maximum temperature, and cooling rate are all factors that may determine the extent of damage from DEF. (Riding, 2006) Temperatures of less than 70°C are deemed to be acceptable.

**Maximum Temperature Difference.**

Large temperature differences can occur when the concrete core is hot and the ambient temperature is low or when the forms are removed when the concrete underneath is hot, and the concrete surface is exposed to the environment, typically referred to as “thermal shock”. The maximum temperature difference causes a change in volume of the concrete, due to thermal expansion and contraction and can cause thermal cracking in concrete when the member is restrained by adjacent elements or foundations.
Some research papers have reviewed the maximum differential temperature limits during the
ever life of concrete, and some suggest that the maximum allowable temperature differential
should increase with the compressive strength (Gajda and VangGeem, 2002). A differential
temperature limit of 20 to 25 degrees is generally specified by many international standards
mentioned in this paper. However, by adopting this limit, it does not give certainty that thermal
cracking will not occur, as it not purely a function of temperature difference. The properties of
the concrete, such as the coefficient of thermal expansion, tensile strength, and modulus of
elasticity, are all time and temperature dependent and, thus, may affect the concrete’s cracking
susceptibility as the concrete matures. (Riding, 2006)

It is recommended ACI 207 or C660 should be used by designers for considering the effects
of restraint on hardening concrete by adjacent elements to ensure sufficient measures are
taken to limit cracking of the concrete.

**CONTROLLING CONCRETE TEMPERATURES**

**Reducing Peak Temperatures**

The primary means for limiting temperature rise is controlling the type and amount of
cementitious materials. The use of low heat cement in concrete and use of pozzolan, such as
fly ash, as a replacement further delays and reduces heat generation. The use of fly ash is the
most common method adopted in New Zealand to reduce heat generation in concrete. Typically, 20% to 35% fly ash replacement for GP cement is made.

Other means to reduce peak temperatures are:
- Reduce the fresh concrete temperature, by adding flaked ice or chilled water to the concrete mix.
- Choosing the appropriate time to make the concrete and pour it. i.e. Pour in winter, pour at night when ambient temperature is low.
- Adding cooling pipes within the concrete section.
- Possibility of shading the area.
- Fog spraying the steel with water.
- Shading the aggregates in bins.
- Larger aggregate size.
- Reducing mixing time without jeopardizing the quality and uniformity.
- Spraying cold water on the drum of agitator.
- Split concrete pours into thinner pours, by adding construction joints, need to be mindful of restraint and thermal cracking.

**Methods for Reducing Differential Temperatures**

Differential temperature can be reduced in a number of ways: Typically leaving insulated forms
on as long as possible is the best way to reduce differential temperatures as the form provides
an insulating barrier to the environment. Sprayed foam on the formwork can be an efficient
means of insulation. Providing coverings over the concrete to prevent heat from escaping. The
use of heating equipment to control the temperature. All other methods of reducing peak
temperature as listed above will also impact the temperature differential.
TEMPERATURE PREDICTION METHODS

There are a number of methods available for temperature prediction, the common methods used in NZ Industry are the ACI 207.2R Method, and Ciria C660 method. In the USA the PCA method is also used and specified by DOTs. Temperature in concrete is governed by two main criteria, exterior ambient temperature, and the heat of hydration, which is the exothermic reaction of the cement in the concrete. The authors’ general rule of thumb to estimate the peak temperature which is on a typical temperature increase based on the initial temperature of the fresh concrete, the heat gain due the amount of cementitious material. The estimate is a follows: Fresh concrete temperature (°C) + (13.5 x (mass of Cement /100)) + (8.5 x (mass of fly ash/100)) ± 2°C, and this rule of thumb is similar to the methods as discussed below.

There are many variables which impact the actual insitu concrete temperature. Key variables include: ambient temperature, temperature of the fresh concrete, heat gain due to cementitious materials (e.g. cement, fly ash), aggregate type, exterior environmental conditions, and type of formwork on the concrete. Three international temperature prediction methods are outlined below.

ACI 207.2R method

The first method reviewed is the ACI 207.2R method, which considers the Effect of Restrain, Volume Change, and Reinforcement on Cracking in Mass Concrete. This method reviews limiting cracks in structural members that occur principally from the restraint of thermal contraction. The report covers a detailed approach which is aimed at addressing thermal cracking, and designing reinforcement to limit cracking to an acceptable level.

The method in the standard includes calculations using the rate and magnitude depending on the amount of and types of binder. The method also allows for the fines of the cement to be included in the analysis. Also included are placing temperature, surface exposure, moisture conditions and ambient air conditions.

The method uses a number of graphs based on American types of cement and is all in Imperial units so becomes cumbersome to use with the New Zealand metric system.

CIRIA C660 Method

The Ciria C660 guide, ‘Early-age thermal crack control in concrete’ provides a method for estimating the magnitude of the risk of cracking due to thermal issues. Included in the analysis for this method are; binder content and type, the size of the element, ambient temperatures, formwork, insulation and time of its removal and active forms of temperature control such as internal cooling pipes. The guide provides a spreadsheet based calculation method that has several variables that allow the user to best estimate peak temperatures and differentials. The Ciria method only estimates differential temperatures located on the same horizontal plane, and does not assess 3D effects. Whilst this method is based on European materials it does allow a quick and easy estimate of predicted temperatures and differentials and is commonly used in New Zealand. The below figures show the spreadsheet. (P.B.Bamforth, 2007)
The Portland Cement Association’s (PCA) Design and Control of Concrete Mixtures gives a quick method for estimating the maximum temperature developed in mass concrete members. This method will be referred to in this paper as the “PCA Method.” This method calculates the maximum temperature rise above the concrete placement temperature, as 12 °C for every 100 kg of cement. The PCA Method is only appropriate for concrete containing between 300 and 600 kg of cement per cubic meter of concrete and assumes that the least dimension of the concrete member is at least 1.8 m. The PCA Method provides no information on time of maximum temperature and does not allow the quantification of temperature differences. (Riding, 2006)

TEMPERATURE MONITORING

It is recommended that on all concrete members which have a minimum dimension of 1.5m or greater, temperature monitoring is undertaken to verify that the concrete mix proposed meets specified temperature limits. The concrete specifier should be aware of the potential scenarios where a section size less than 1.5m which could cause temperature problems, such as high cement content. The specifier should discuss with the contractor and concrete supplier as to why temperature monitoring is needed and ensure the mix proposed and construction practices proposed are expected to meet the temperature limits imposed. A heat control plan should be established and agreed between all parties.

Measuring Requirements for Temperature Monitoring

For mass concrete with a minimum dimension of 1.5m or greater, the maximum temperature in the concrete usually occurs between 2 to 5 days after placement depending on the mix and size of pour. As a minimum the temperature shall be recorded every 1 hour for the first 72 hours then every 3-4 hours for at least 5 days and until advised by the engineer to when the peak temperature and differential are at acceptable limits to cease monitoring. Typically, when the maximum temperature is around 35-40 degrees or less and the temperature is declining is an appropriate time to stop monitoring.

Positioning of Temperature Monitoring Probes

Before concrete is placed, probes which can measure temperature and send a signal back to a receiver should be placed at the required locations on the structure. These probes are to be placed at maximum depth, in the centre or core of the structure, and on the edge of the
structure in the cover zone, typically on the inside of the reinforcing cage and at a depth of 75 mm. Depending on the size of the pour there could up between 2 and 10 probes required.

CASE STUDIES

Five concrete pours case studies are detailed below. Each case study outlining the size of the pour, cementitious content, any special measures taken, and temperature monitoring data obtained during the concrete hardening process. The first three case studies exceed the minimum dimension of 1.5 m. It is important to note that concrete temperatures can be problematic in smaller size concrete pours with high cementitious contents, or high strengths, and hence two case studies are presented which have a smaller minimum dimension than 1.5 m.

Case Study 1

Piles for a new bridge over a river in north Waikato are 1.8 m in diameter. The concrete mix required a 50 MPa strength at 28 days, and a 19 mm maximum aggregate size, with a target slump of 180 mm. The binder content included 362 kg GP Cement and 168 kg fly ash. The mix was also retarded for 6 hours as the pile was a tremie pour. The actual average 28 day strengths were 66 MPa. The pile used a steel form. The C660 method was used to estimate the maximum temperature, and was estimated to be 58°C with a maximum differential of 18°C. The estimated adiabatic peak was 74°C. Probes were placed in the centre of the pile at depths of 3.6 and 5.6 from the top of the piles. Probes were placed at the edge of the pile at the same depths. The graph shows the results. Time intervals shown are daily. The level of the river was between 4 and 5 m from the top of the pile and the effect of the flowing river water can be seen very clearly. Average river temperature for the river in April is 18°C.

Case Study 2:

This case study is a 2.6 m diameter solid reinforced concrete column in the Waikato area. Maximum temperature limits were set at 70°C peak temperature, and a maximum differential of 20°C. Due to access issues a 30 MPa 13 mm self compacting concrete mix was proposed. For the first pour the cement content was 345 kg GP. The concrete column form was uninsulated steel. The Ciria C660 method estimated a peak of 72 °C and a differential of 35°C although this model was not used at the time. From the data we can see that the maximum
temperature was exceeded as well as the differential limit. The effect of the sun can be seen clearly on the cover probe even though this probe was 100 mm deep.

The first mix did not meet the desired project limits, and for subsequent pours the mix was adjusted to include fly ash to help reduce the maximum temperature, additionally the steel formwork was also then insulated with a sprayed foam to reduce the differential temperatures between the core and the surface. The subsequent mix used contained 247 kg GP cement and 133 kg fly ash per m3. The recorded 28 day average strength results was 58.1 MPa. A probe was also placed at the top edge of the column, 100mm below the surface and 100mm from the face. Geotextile was used as a blanket on top of the column but once this got wet, it provided no benefit to reducing differential temperatures. With the revised fly ash mix, the Ciria method predicted a maximum temperature of 60°C and a differential of 30°C. The change to 35% fly ash gave a peak temperature reduction of about 7°C and the introduction of insulation reduced the estimated temperature differential to 17°C.
Case Study 3

Case study 3 is a 2.5 deep concrete beam in the Waikato Area. The beam required a 40MPa concrete mix and a mix design comprising 260 kg GP cement and 135 flyash was used to reduce peak temperatures. Plywood formwork was used. The Ciria method estimated 58°C peak temperature and 42°C differential. The plywood forms acted as a good insulator and the pour occurred around mid-winter which is why the peak temperature is estimated to be 58°C. Actual peak temperature measured was 68°C, and temperature differential was 18°C. Verification was not undertaken on the final temperature probe location for the cover locations and given the low differential temperatures recorded, it is expected these would likely be 300mm deep into the concrete, and the timber formwork provided better than expected insulation.

Figure 5. Case Study 3 – Temperature Monitoring Data

Case Study 4

Case study 4 is beam located in Auckland and had a maximum temperature specified of 70°C and a maximum differential of 20°C. The formwork used was 18mm plywood. The concrete required a 30 MPa 19 mm mix and had a 30% flyash content. The beam was 2.6 m by 1.3 m by 8.5 m long. The mix contained 243 kg GP and 105 kg fly ash per m3. The Ciria model predicted a maximum of 62°C with a differential of 37°C. This beam was poured in March so the effect of the sun is clearly visible. The 28 day average strength was 42.4 MPa. It can be seen even though this beam has a minimum dimension of 1.3m, and only a 30MPa strength requirement, the use of 30% fly ash was still required to bring the peak temperature down to 62 degrees. This case highlights the need for fly ash and temperature monitoring, in particular in summer months, when the fresh concrete temperature is over 25°C. It is thought the summer temperatures reduced the actual temperature differential compared to what was estimated.
Case Study 5

This case study is for a 1.1m thick foundation pad with a 12.5 m diameter base. The concrete was a 30 MPa 37mm mix containing 235 kg GP and 105kg fly ash per m3. The 28 day average strength result was 41.8 MPa. The Ciria model predicted a maximum temperature 68°C and a maximum differential of 16°C. Probes were placed in the centre of the thickest section, 75mm from the bottom, the centre and then 75mm from the surface. The top was insulated with a layer of soil which is why the differential observed was low.

SUMMARY OF CASE STUDIES

The following thoughts are documented as a result of the case studies. Fly ash is a suitable replacement to GP cement, and is effective at reducing the peak temperatures. Fly ash is commonly used in ‘mass concrete’ structures to reduce peak temperatures. The fresh concrete and ambient temperatures have a direct impact on the peak and differential temperatures in concrete. Plywood is an effective insulator, and sprayed foam is also an exceptional insulator which can be used to reduce differential temperatures. The Ciria temperature prediction method was used for all case studies is simple and easy to use and provides a relatively accurate estimate of the expected temperatures, however actual monitoring results varied in a couple of the case studies. This is due to the actual conditions of the pour not being able to be
modelled in the calculation. The Ciria method needs specific calibration for local materials and actual conditions for the output to yield greater accuracy when compared to actual results.

Table 1. Summary of Case Study Concrete Mixes

<table>
<thead>
<tr>
<th>Case Study</th>
<th>Minimum Dimension</th>
<th>Grade MPa</th>
<th>Max Agg size mm</th>
<th>Max probe depth mm</th>
<th>GPC kg/m³</th>
<th>Fly ash kg/m³</th>
<th>28 Day MPa</th>
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<td>900</td>
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<td>168</td>
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<td>13</td>
<td>1300</td>
<td>345</td>
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<td>49.6</td>
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<tr>
<td>2b</td>
<td>2.6m dia</td>
<td>30</td>
<td>13</td>
<td>1300</td>
<td>247</td>
<td>133</td>
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<td>3</td>
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<td>40</td>
<td>19</td>
<td>1250</td>
<td>260</td>
<td>135</td>
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<td>4</td>
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<td>19</td>
<td>650</td>
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Table 2. Summary of Temperature Monitoring Data

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<th>Ciria maximum ºC</th>
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<td>5</td>
<td>1.1m</td>
<td>68</td>
<td>16</td>
<td>52</td>
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EXISTING CHALLENGES IN THE NZ INDUSTRY

There appears to be no peak or differential temperature documented in any widely used New Zealand literature, making it an extensive research task for a concrete specifier/engineer to make an informed decision on what is or is not appropriate for temperature requirements in concrete, and in particular mass concrete. Higher strength concretes of 50MPa and 60MPa are becoming more common in mass concrete structures, meaning temperature related problems are becoming more prevalent in the industry. Heat curing is becoming common practice with precast elements to accelerate concrete strength gain, and designers may not be aware of the detrimental effects which could be caused by high temperatures, particularly if they have not needed to address the issue before. If there is no limit in the design specification the concrete supplier may not be aware of the limit, or the size of the pour that the concrete is to be used for. Concrete suppliers only supply the actual concrete and cannot control the construction method, types of formwork used or other factors on site, and therefore the contractor, concrete supplier and designer/specifier need to co-ordinate a plan to achieve the desired end result. A competent concrete supplier can predict peak and differential within acceptable limits, but they cannot supply concrete that will conform to a specification without the contractor using sufficient measures to control factors which effect the temperature of the concrete.
SUMMARY AND CONCLUSIONS

On mass concrete pours, with a minimum dimension greater than 1.5m, or when the engineer deems it necessary, a heat control plan should be established and agreed between all relevant parties to ensure peak and differential temperature limits are met for the reasons outlined in this paper. It is recommended that concrete temperature monitoring on all pours with a minimum dimension greater than 1.5m is undertaken to identify and mitigate the risk associated with high differential or peak temperatures.

The designer should specify that mass concrete is special concrete in accordance with NZS3109, and mandate temperature monitoring is undertaken, with appropriate action by the contractor to ensure firstly, an appropriate mix design, and secondly, a construction method which will address the concrete temperature requirements specified. Designers should also specify the maximum peak and differential concrete temperature limit, with proposed acceptable limits being 70°C as the maximum peak and 25°C maximum temperature differential, and reference be made to either CIRIA C660 or ACI207 for further guidance. In some cases, temperatures higher than the limits proposed may be considered acceptable, depending on the concrete mix design and specific details associated with the concrete pour in question, although not recommended.

The author’s recommend clearer guidelines and requirements are established around temperature limit requirements, temperature monitoring, and implementing the use of heat control plan when dealing with mass concrete. These guidelines and requirements should be presented in an appropriate standard, with the aim of ensuring all parties involved in concrete design, supply and construction, produce satisfactory outcomes when it comes to the integrity of concrete.

The authors also highlight future research could be undertaken to establish and correlate international temperature prediction methods for use with typical New Zealand Concrete mixes.

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