Thermal control systems to enable rapid concrete placement at Yusufeli Dam

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ABSTRACT:

The construction of a large mass concrete arch dam is as much about the management of thermal effects as it is about lifting of formwork and pouring of concrete. Arch dams are three-dimensional structures, but in conventional mass concrete, they are constructed vertically as a series of cantilever blocks. With hydration of concrete being an exothermal reaction, surface and mass gradient temperature effects develop and must be controlled during the process of construction.

Unlike mass concrete gravity dams, the individual cantilever blocks of an arch dam will often be unstable when constructed to full height if not supported by the adjacent, down-slope block. Consequently, it is necessary to cool the monolith blocks of an arch dam to a pre-determined closure temperature and to grout the joint between adjacent blocks before the limiting state of stability is reached. This process must be achieved sufficiently slowly to avoid surface gradient cracking and sufficiently quickly to allow the required concrete placement rates to be achieved.

In this paper, the authors discuss the thermal analyses and temperature control systems applied to achieve an average placement rate of 150 000 m^3 per month at the Yusufeli Dam on the Çoruh River in north-eastern Turkey.

1 PROJECT DESCRIPTION

The Yusufeli double-curvature conventional concrete arch dam is currently under construction on the Çoruh River in the Black Sea region of north-eastern Turkey. The associated hydropower plant will contribute 558 MW of electricity generation capacity to the national grid. Concrete placement for the dam structure was initiated in July 2018 and is scheduled for completion at the end of 2020. Comprising 4 million m^3 of concrete, the 275 m high dam will be the sixth highest dam in the world and the highest dam in Turkey.

2 BACKGROUND & INTRODUCTION

2.1 *Arch dam construction stability requirements*

The design analysis requirements for a large arch dam are generally more complex than is the case for most other dam types, with each stage of the design development requiring progressively more detailed analyses and various thermal and structural analyses being required specifically to manage behavior during construction.

A conventional mass concrete gravity dam will require that each individual monolithic construction block must indicate adequate lateral stability on its foundation to be constructed to full height, while an RCC gravity dam is constructed horizontally and consequently inherently restrained against lateral movement during construction. When constructed without formed joints, a similar situation will generally exist for an RCC arch dam. The joints between the monolithic construction blocks of a conventional mass concrete arch dam, on the other hand, must be grouted to allow support to be provided to each block on the abutment by the adjacent downslope block, before a limiting state of stability is reached. Correspondingly, after concrete placement, the hydration heat must be removed, the concrete further cooled to the requisite temperature and the joint must be filled with grout before concrete placement on the particular block exceeds the allowable elevation.

The associated cooling requirements must be tempered against a conflicting requirement not to exceed the tensile strain capacity of the constituent concrete through the development of excessive thermal gradients during the post-cooling process.

2.2 *Mass concrete thermal control*

As presented in Figure 8-9 of the USACE Engineering Manual EM 1110-2-2201 Arch Dam Design (1994) and Figure 1 below, the temperature within the core of an arch dam structure indicates 4 important temperature points and 5 primary thermal cycle periods.

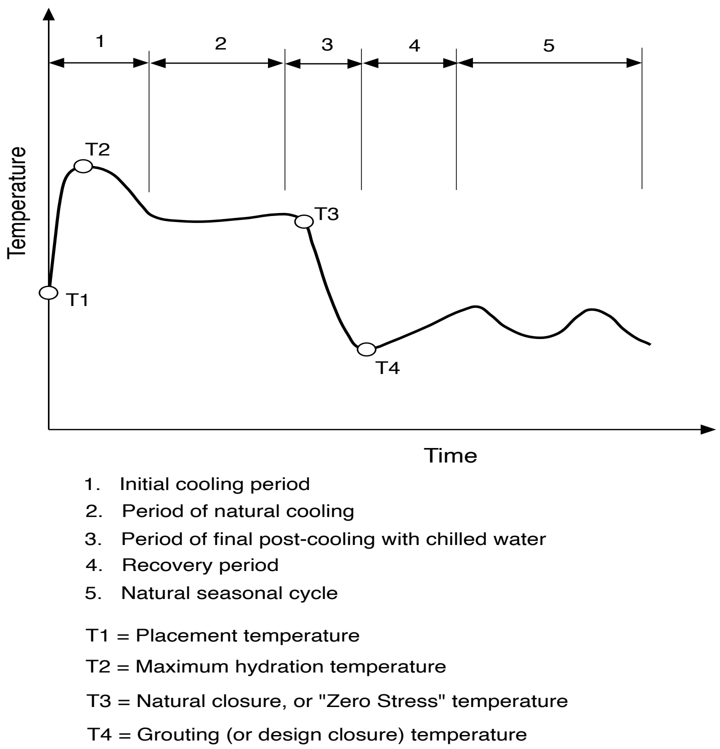

Figure 1. Concrete Temperature-Time graph (USACE, 1994)

The key elements illustrated in Figure 1 relate to mass-gradient thermal effects and beyond the basic concept of ensuring that T4 (the concrete temperature at the time of grouting) represents a sufficiently low concrete temperature to allow the structure to adequately accommodate all future cold-season temperature conditions, an important aspect of the concrete behaviour is demonstrated in the temperature difference T3 – T1. At the natural closure, or “zero stress” temperature, T3, during the cooling cycle, the concrete will indicate the same volume as at placement and the difference between the two temperatures, T3 – T1, represents an effective shrinkage in the concrete volume during the course of the hydration cycle.

In EM 1110-2-2201, it is suggested that T3 should either be equated to T2, or taken as between 1,5 to 2,5˚C less than T2, depending on the applicable placement lift height. In either case, the implication is a shrinkage in the concrete volume that is approximately equivalent to the thermal expansion that could have been expected in mature concrete for a temperature rise equivalent to the full hydration temperature rise. Although the related effects have been managed through temperature loads in mass concrete dams technology, the applicable shrinkage is actually a stress-relaxation creep that occurs as the immature concrete attempts to expand thermally, while partly retrained/contained.

Conversely, problematic effects that occur during both natural and forced cooling, and in the process of managing the mass-gradient thermal issues, relate to surface-gradient thermal phenomena. Concrete will obviously expand and contract with rising and reducing temperatures and surface-gradient effects are developed through the occurrence of differential heating and cooling, or thermal gradients, which can be caused by cooler external conditions, or cooler internal conditions immediately around a cooling pipe. With a coefficient of thermal expansion of the order of 1×10^{-5}, temperature changes of 10˚C in concrete will incur volume changes of the order of 100 microstrain, which represents a strain of a similar order as the tensile strain capacity of mass concrete. Consequently, thermal gradients in large concrete masses can easily develop tensile strains that can result in the development of cracking.

Referring to Figure 1, during the “initial cooling period”, water is circulated through cooling pipes in an effort to draw out some of the hydration heat and limit the peak temperature reached during hydration. In reality, this exercise can only suppress the maximum hydration temperature by 1,5 to 2,5˚C, although the reduced total temperature increase in the immature concrete can beneficially reduce the total stress-relaxation creep incurred during hydration.

As the concrete temperature continues to increase during hydration, containment from adjacent concrete experiencing the same hydration will imply, on a macro scale, only compression stresses will generally be experienced in the core of a large concrete mass and consequently cracking in the immature concrete is less likely to develop during the first part of the cycle. It must be understood, however, that the hydration process can actually be slowed by cooling, particularly in the concrete immediately surrounding the cooling pipes. In the second part of the temperature cycle, once the maximum hydration temperature has been reached, any subsequent forced, or rapid reduction in temperature will give rise to the development of thermal gradients; whether due to a cooler external temperature, or due to cooler temperatures immediately surrounding the cooling pipes. With concrete having gained strength during the second cooling period, a more intensive cooling can be implemented during the third period, circulating chilled water at a more rapid rate through the cooling pipe system to bring the concrete temperature down to the target temperature for joint grouting (T4).

During the development of mass concrete dam construction, a number of generic “rules of thumb” were established in respect of allowable thermal gradients and maximum daily rates of concrete cooling. According to the USACE (USACE, 1994), the rate of cooling of the concrete core during the final (and intermediate) cooling stages should not exceed approximately 0.3˚C per day, while an equivalent figure of between 0,3 and 0,6˚C is suggested by the USBR (USBR, 1977). Many specifications for mass concrete restrict the maximum internal/external temperature difference to between 15 and 20˚C in order to control cracking.

In his publication Thermal Stresses and Temperature Control of Mass Concrete (Bofang, 2014), Zhu Bofang concludes that to assure that no dangerous tensions are created through thermal gradients during the process of post-cooling of mass concrete in dams, the temperature of the post-cooling water should ideally be no more than 10˚C less than the temperature of the concrete being cooled. Considering a typical concrete hydration temperature rise of more than 20˚C, this requirement would typically necessitate the initial use of warmed water, which is not usual practice, and would imply waiting periods before arch joint grouting of years, rather than months. As impractical as this idealization might be, it realistically implies that some level of thermal cracking will always occur consequential to mass concrete post-cooling in dams and the priority consideration can realistically be limited to ensuring that any such cracking does not compromise the structural capacity, or response of the dam under load.

2.3 *Paper objectives*

At the Yusufeli double-curvature concrete arch dam, currently under construction on the Çoruh River in north-eastern Turkey, an initial target of placing approximately 4 million m^3 of concrete in 26 months was set, implying the requirement for an unprecedented average concrete placement rate exceeding 150 000 m^3 per month. As much as this target represented a challenge for the concrete manufacture and conveyance system, the achievable construction rate would also be determined by the achievable safe rate of concrete post-cooling.

Typically, the final stage of post-cooling with chilled water to allow joint grouting for a concrete arch dam will only be initiated between 2 to 12 months after concrete placement (USBR, 1977) and the average rates of concrete placement on the super-high arch dams in China have not generally significantly exceeded 100 000 m^3 per month, despite a similar rate of placement being achieved at Hoover Dam in the 1930s. While the site geometry and geology will give rise to unique conditions in respect of critical cantilever block stability for every arch dam and information related to the impact of cooling rates on concrete placement for arch dams is not generally available, few mass concrete arch dams have seen average vertical placement rates exceeding 10 m per month. To achieve average rates of vertical rise of 10,6 m per month and concrete placements averaging 150 000 m^3 per month would require very rapid concrete cooling in certain critical areas of the Yusufeli dam structure and a period of 2 to 12 months for natural cooling would not be feasible. In the realities of large-scale construction, careful planning to avoid deleterious concrete cracking was necessary and a system to proactively adjust and accommodate variations in schedules and other practical realities was an essential component of construction quality control.

In this paper, the authors address the analysis system and model that was developed to interactively predict, monitor and control the concrete post-cooling process during the construction of the Yusufeli double-curvature concrete arch dam

3 PREPARATORY STEPS

3.1 *General*

Before the development of the thermal-grouting model could be initiated, it was first necessary to establish the target thermal condition of the dam structure for grouting of the joints between the monolith blocks and to determine the limiting height to which each of the cantilever monoliths could safely be constructed without lateral support. The associated studies are subsequently described in very basic detail. The primary concrete, representing 80% of the dam concrete, was designated C20/120 and contained 130 kg/m^3 CEM1 cement, 70 kg/m^3 fly ash, 110 litres/m^3 water, 2183 kg/m^3 aggregates with a maximum size of 120 mm, plasticizing/set retarding and an air entraining admixtures and indicated a slump of approximately 3 to 5 cm.

3.2 *Long-term thermal analyses*

The adiabatic temperature rise for the core C20/120 concrete was measured as 23 to 24˚C in concrete placed in the first block of the dam constructed at the top of the left abutment to support the temporary works. Using this information, the available climate data, estimations of seasonal water temperature profiles, the intended construction schedule and the specified maximum allowable concrete placement temperature of 18˚C, the critical temperatures in the core of the dam, across the full height of the structure were determined through finite element (FE) thermal analysis. Target (T4) grouting temperatures of 10˚C in the main body of the dam and 12˚C against the foundations were subsequently assumed and these were demonstrated to be appropriate through structural FE analysis under the coldest conditions predicted for the concrete structure.

3.3 *Construction stability analyses*

Each of the 29 monoliths comprising the dam structure was analyzed as a "stand-alone" structure, determining the respective limiting height with a factor of safety against sliding of 1,5 and/or a maximum lateral cantilever displacement of 5 mm.

Figure 2. Yusufeli Dam site on completion of excavation

In the same process, the stress state of each monolith was evaluated at various critical stages during construction, to ensure no significant vertical tensions were developed in the structure as a consequence of its geometry. Determining the maximum "stand-alone" height for each cantilever, unsurprisingly, the analyses indicated the critical situation to exist on the steep right abutment, with only the lowermost and uppermost two monoliths indicating adequate unsupported stability to full height. Translating the proposed concrete placement schedule into a schedule of vertical rise for the dam structure, the findings of the stability analysis were interpreted against the necessary grouting times for each of the 13 horizontal grouting compartments.

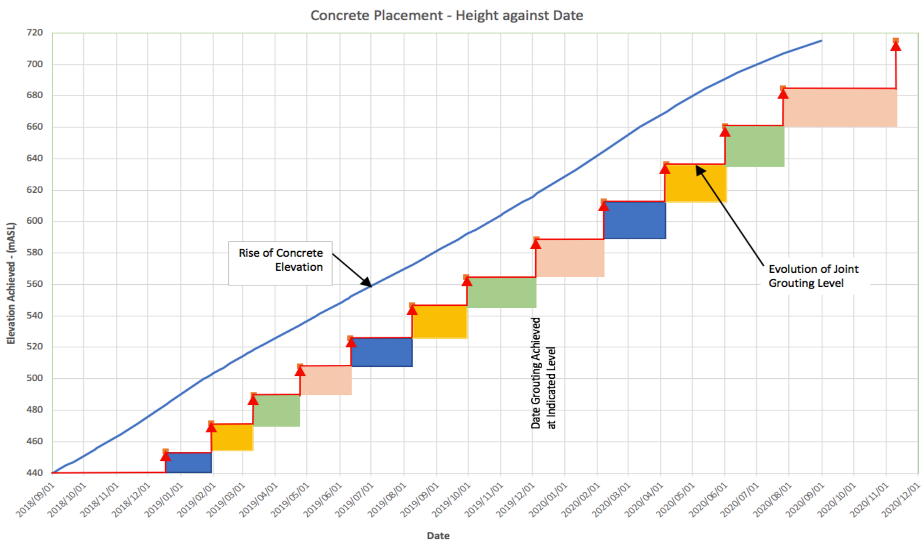

Figure 3. Anticipated evolution of joint grouting

This analysis revealed the timing of grouting and the rate of post-cooling to be critical at several different stages during the dam construction, requiring that post-cooling at the maximum possible rate must be achieved and sustained.

3.4 *Concrete behavior measurements*

The first block of the dam constructed at the top of the left abutment in advance of the main dam body was instrumented with temperature, stress and strain gauges in order to gain the best possible understanding of the early behavior of the constituent concrete during the hydration cycle. Using significant percentages of fly ash in the cementitious materials significantly changes the chemical and autogenous shrinkage characteristics of the paste during hydration, with a consequential influence on the concrete autogenous shrinkage (Bofang, 2014 & Shaw, 2010. This characteristic can in turn impact the level of stress-relaxation creep during the hydration cycle, or the T3-T1 temperature (Shaw, 2017).

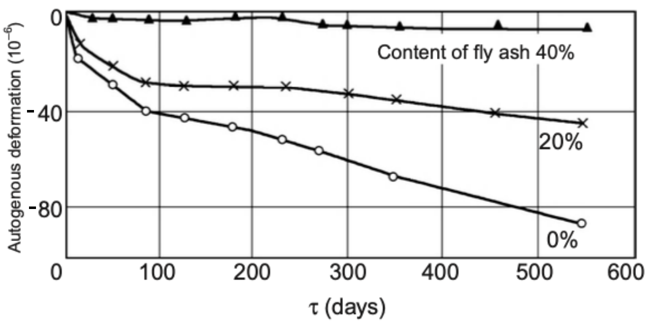

Figure 4. The influence of fly ash content on the autogenous deformation of concrete (Bofang, 2014)

While reduced stress-relaxation creep is typically beneficial for mass gradient thermal effects, it can be more problematic with respect to increasing sensitivities to thermal gradients, as hot core concrete experiences increased levels of expansion and tensions are developed in the cooler surface concrete. Conversely, any consequential cracking is more likely to remain localised.

With a fly ash content of 35%, it was considered that the early construction of the first block (see Figure 5) provided an important opportunity to gain an understanding of the associated early concrete behaviour and while such measurement using temperature, stress and strain gauges cannot be considered to provide absolute quantitative values, a very useful indication of typical behaviour can be gained. The installed instruments provided a strong indication of linear behaviour, with stress relaxation creep apparently limited to below 40 microstrain, which is very substantially less than the 180 to 200 microstrain that would be suggested on the basis of common practice (USACE, 1994).

Figure 5. The first concrete block supporting temporary works at the top of the left abutment

3.5 *Model conceptualization*

Having demonstrated the safe post-cooling of the constituent concrete to 10˚C to be as critical in achieving the overall target rate of concrete placement as the concrete mixing and conveying system capacity, it was necessary to establish the maximum allowable rate of concrete cooling during the various stages of concrete strength development, to review the practicalities of rapid cooling and to establish a model through which a coordination could be maintained between the concrete placement, the concrete cooling and the contraction joint grouting.

4 THERMAL-GROUTING MODEL

4.1 *Model objectives*

The model developed accordingly served a number of purposes. In providing control for the post-cooling processes, the model was able to identify those specific elevations in the arch and cantilever monoliths with the greatest sensitivity in respect of post-cooling requirements and consequently the control of concrete placement rates. Additionally, the model would allow the prediction of the impacts of any delays in placement, indicating the consequential impact on the date of joint grouting, subsequent concrete placement and dam completion. Necessary adjustments in the rates of cooling can be made, should temperatures drop faster than the allowed rate during particular cooling periods. In principle, the model provided a means to safely control and record the process of cooling and placing concrete as rapidly as possible.

The function of the model was predictive, but also reactive. It was initially set up on the basis of the maximum allowable cooling rates, but was subsequently adjusted to accommodate the actual maximum achievable cooling rates, to give a warning when measured data indicated cooling at a faster rate that allowable and to adjust the joint grouting date and concrete lift placement dates on the basis of the measured concrete temperatures in each placement lift.

4.2 *Allowable cooling rates*

As discussed above, the allowable concrete cooling rates are determined by the requirement to limit the consequential thermal gradients at a particular time to a level at which the simultaneous tensile strain capacity of the concrete is not exceeded and consequently, the first related requirement was a determination, or estimation of the time-development of the concrete tensile strain capacity. Testing of the Yusufeli C20 /120 concrete indicated 7, 28 and 90 day tensile strengths of 1,3, 2,5 and 3,4 MPa, respectively and the equivalent slow-load tensile strain capacities were estimated as 110, 150 and 170 microstrain.

Modelling two 3 m placement lifts, with cooling pipes at 2 m centres on the top surface of the first lift, several thermo-mechanical analyses were undertaken in an attempt to model the strain and stress development in the concrete through the full hydration heating and cooling cycle, assuming a zero stress-relaxation creep. Finally, the development of an adequately credible model for the full cycle was found to be too complex within the available time frame and a model that assumed a starting temperature distribution at the peak of the hydration cycle was used, taking measurements from actual blocks in the dam, and only the subsequent post-cooling cycle was modelled for a series of different cooling rates. The criteria of not exceeding a tensile strain of 150 microstrain at 28 days and 170 microstrain at 90 days within 400 mm of the cooling pipes was used to determine the maximum allowable rates of cooling.

With measurements on the dam indicating the maximum temperatures to be reached within 30 to 45 days of concrete pouring, it was found that the immediate application thereafter of chilled water quickly induced problematic levels of strain and the analyses confirmed that it was significantly preferable to use river water (11 – 23˚C, depending on time of year) for the initial post-hydration peak cooling. Cooling with river water should be continued for around 50 to 60 days, with a maximum cooling rate of approximately 0,15˚C per day (max 0,17˚C), subsequently changing over to chilled water, with a maximum cooling rate of approximately 0,3˚C per day.

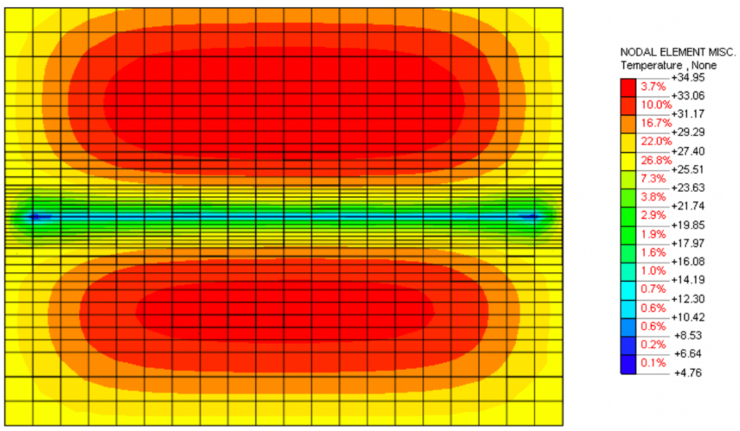

Figure 6. Illustration of excessive temperature gradients during rapid post-cooling with chilled water

At Yusufeli Dam, post-cooling was achieved using 19 mm diameter steel cooling pipes in coils at 2 m centres in the main dam body, and at 1,5 m centres in the blocks immediately adjacent to the abutments and the coils were placed at the bottom of each 3 m lift. The pipe systems were tested under a 10 bar pressure and river water was circulated as soon as concrete placement was initiated and continued for a period of approximately 8 - 9 weeks, after which time the concrete had typically reached a temperature of between 20 and 27˚C. Subsequently, water chilled to between 4 and 5˚C was circulated for a period of between 1,5 and 2 months, until the target grouting temperature was achieved.

4.3 *Model inputs*

The Yusufeli arch dam structure comprises 29 cantilever monoliths, one of which was constructed separately and ahead on the main dam structure at the top of the left abutment. The remaining 28 monoliths comprise a total of 1901 placement lifts, each of 3 m in height. For each of these placement lifts, a date of concrete pouring was established on the basis of the proposed construction schedule. Considering the requirements of the construction stability analysis in respect of maximum unsupported cantilever heights and the proposed concrete placement schedule, the necessary programme for arch joint grouting was established. Consequently, the Contractor set himself a formidable challenge to counterbalance the achievement of a very rapid target concrete placement rate with the process of safely and effectively post-cooling the concrete in each placement lift to the required grouting temperature in sufficient time to allow his concrete placement rate to be sustained.

A critical input of the model was the rate of cooling and while this was initially established on the basis of the maximum allowable, after a number of months of monitoring the actual rates of cooling achieved, it was subsequently based of these rates. Operating the post-cooling system on the basis of water circulated at a flow of 0,29 litres/s, analysis of the available data indicated a coefficient of cooling (K_{ce}) of essentially between 0.045 and 0.007, where:

$$k_{ce} = \frac{1}{t_2}\ln\left(\frac{\theta_{peak}}{\theta_{Gr}}\right) \qquad t_2 = \frac{1}{k_{ce}}\ln\left(\frac{\theta_{peak}}{\theta_{Gr}}\right)$$

Where Θ_{peak} = peak hydration temperature; Θ_{Gr} = average joint grouting temperature; and t2 = time taken to reach grouting temperature.

It was also found that the rate of cooling was partly influenced by the volume of the lift placed, as a consequence of smaller blocks having greater surface area exposure and shorter cooling loops, as illustrated in Figure 7, and a best fit line of Kce = -5 x 10^{-7}x Lift volume + 0.0075 was established. The equivalent average concrete cooling rates are between 0,1 and 0,15˚C per day.

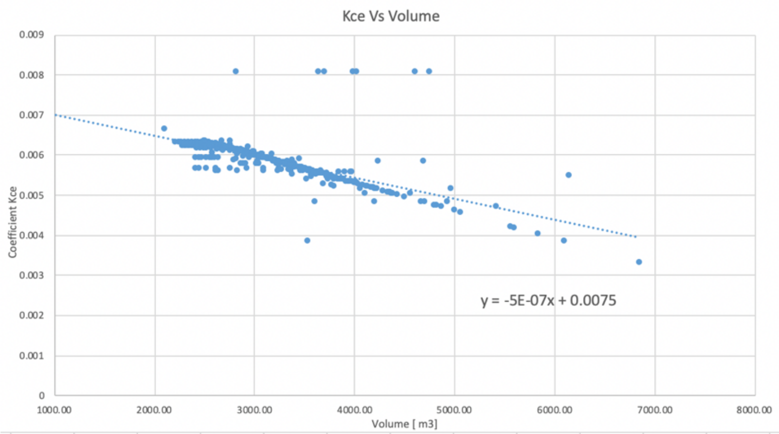

Figure 7. Plot of cooling coefficient (K_{ce}) against lift volume

4.4 *Model operation*

The model was initiated with a planned start date for each of the concrete placements and an associated necessary grouting date for each grouting compartment. After concrete casting, temperatures on all installed instrumentation were measured every one to three days and the associated information was input into the model, monitoring temperature development and dissipation against the initial assumptions.

The first three grouting compartments were grouted at the end of April, in mid-June and in September 2019, respectively. According to the original project programme, grouting of the second compartment had been scheduled for mid-January 2019 and the third by the end of February 2019. Certainly during the early concrete placement operations it was clear that the post-cooling would not be able to achieve sufficiently rapid cooling to realise the initial target grouting dates. While this situation created significantly reduced risks in terms of concrete cracking, it implied that the model must be revised to highlight particular risk areas in terms of limiting cantilever block stability on a number of occasions.

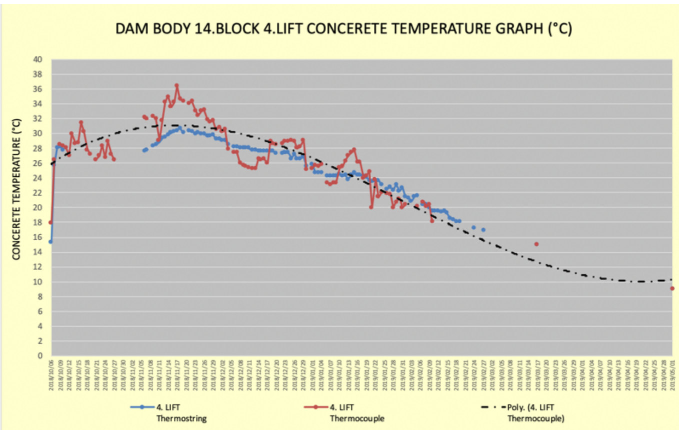

Figure 8. Example of measured concrete cooling

The pre-cooling could consequently be seen to be the controlling factor on the achievable rate of construction, rather than the mixing and delivering of concrete.

The model allows a comprehensive review of whether the concrete temperatures in a particular grouting compartment, as well as in the one above, have reached an appropriate level to allow joint grouting to be initiated, while also enabling a proactive prediction of when compartments will be ready for grouting, which in turn establishes the requirement for an acceleration, or deceleration of cooling water flow.

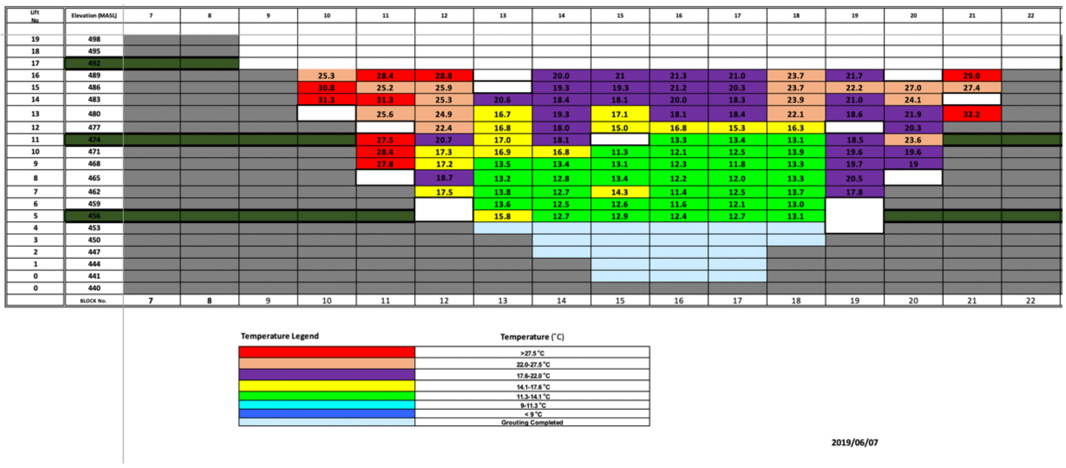

Figure 9. Average concrete temperatures at 07 June 2019

5 DISCUSSION & CONCLUSIONS

As the construction of Yusufeli Dam continues, the cooling/grouting model is continuing to evolve. To date no problematic stress levels, or evidence of thermal cracking has been measured, or observed and it is clear that a denser arrangement of cooling pipes, larger diameter pipes, or more rapid water circulation would in fact be necessary to achieve a rate of concrete cooling that would incur sufficiently steep temperature gradients to incur consequential cracking. As the dam structure section thins higher up the arch, however, and with greater constructional efficiency in respect of the cooling systems, more rapid rates of concrete cooling are likely to be achieved and these will need to be monitored closely through the thermal model. The model will also be of particular use in monitoring and managing critical stability situations on particular cantilever blocks, as related levels become more, or less critical with variations in achievable cooling and actual concrete production, etc.

6 ACKNOWLEDGEMENTS

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7 REFERENCES

ACI. 2007. Report on Thermal &Volume Change Effects on Cracking of Mass Concrete. ACI 207.2R-07. Committee 207. American Concrete Institute. Farmington Hills, Michigan, USA.

Bafang, Z. 2014. Thermal stresses and temperature control of mass concrete. The China Institute of Water Resources and Hydropower Research and Chinese Academy of Engineering.

Shaw, Q.H.W. 2010. A new understanding of the early behaviour of RCC in large dams. *PhD Thesis.* University of Pretoria*.* South Africa..

Shaw, Q.H.W. 2017. The influence of stress-relaxation creep on the design of large RCC dams. *Symposium – Knowledge-based Dam Engineering.* 85th ICOLD Meeting. Prague, Czech Republic. July 2017.

USACE. 1994. Arch Dam Design. Engineering Manual. EM 1110-2-2201. Department of the Army. United States Corps of Engineers. Washington, DC. USA.

USBR. 1977. Design of Arch Dams. Design manual for concrete arch dams. United States Department of the Interior. Bureau of Reclamation. Denver, Colorado, USA.