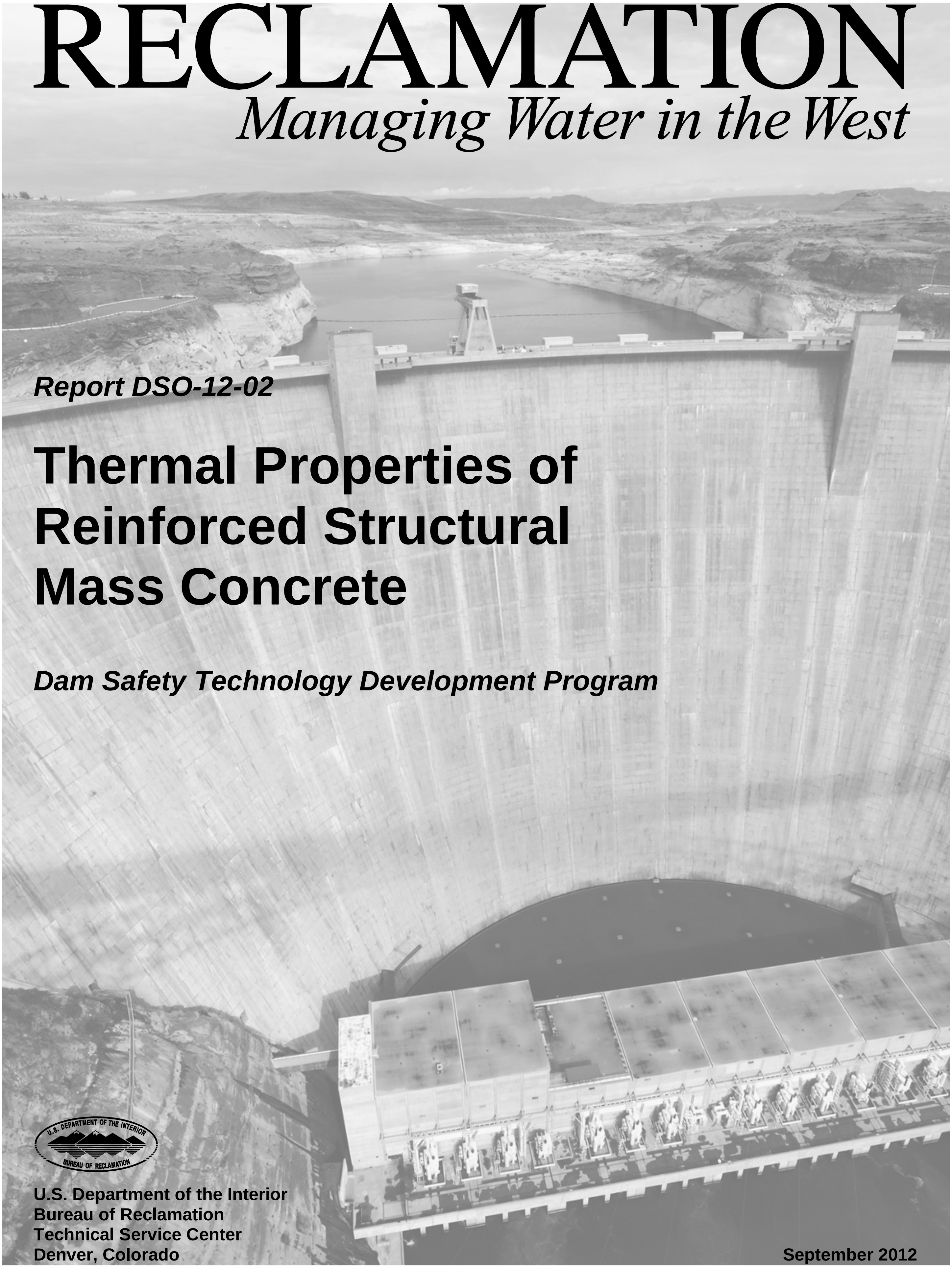

RECLAMATION

Managing Water in the West

Report DSO-12-02

Thermal Properties of Reinforced Structural Mass Concrete

Dam Safety Technology Development Program

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

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The in-situ temperature rise of structural reinforced mass concrete (RSMC) differs significantly from traditional unreinforced mass concrete, such as dams. The compressive strength of RSMC structures is often higher and required at earlier ages. This leads to higher cementitious materials contents and subsequent higher potential temperature rise. Two RSMC placements were monitored for temperature rise and potential harmful temperature gradients, one with a 4,000 psi design strength and the other with a design strength of 7,000 psi. The temperature rise was monitored at the center, near the surface, and at mid-point locations for up to 90 days. One mixture was also tested in the laboratory for compressive strength, elastic properties, and thermal properties under standard curing conditions and under a simulated in-situ temperature regime. This report will highlight the in-situ properties of RSMC and compare methods taken to reduce the temperature rise.

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Thermal Properties of Reinforced Structural Mass Concrete

Dam Safety Technology Development Program

prepared by

Katie Bartojay, P.E.

Mission Statements

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BUREAU OF RECLAMATION
Dam Safety Technology Development Program
Materials Engineering and Research Laboratory, 86-68180

DSO-12-02

Thermal Properties of Reinforced Structural Mass Concrete

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Abstract

The in-situ temperature rise of structural reinforced mass concrete (RSMC) differs significantly from traditional unreinforced mass concrete, such as dams. The compressive strength of RSMC structures is often higher and required at earlier ages. This leads to higher cementitious materials contents and subsequent higher potential temperature rise. Two RSMC placements were monitored for temperature rise and potential harmful temperature gradients, one with a 4,000 psi design strength and the other with a design strength of 7,000 psi. The temperature rise was monitored at the center, near the surface, and at mid-point locations for up to 90 days. One mixture was also tested in the laboratory for compressive strength, elastic properties, and thermal properties under standard curing conditions and under a simulated in-situ temperature regime. This report will highlight the in-situ properties of RSMC and compare methods taken to reduce the temperature rise.

Introduction

Currently, Safety of Dam modifications are being constructed with portions containing reinforced structural mass concrete (RSMC). RSMC is different from the typical unreinforced mass concrete that Reclamation is familiar with. The concrete mixtures have higher compressive strengths and elastic properties requirements at much earlier ages, smaller nominal maximum size aggregate (NMSA), higher cementitious materials (cement plus pozzolan) contents, higher initial and peak curing temperatures, and significantly more reinforcing steel than typical mass concrete. Also, Type IV cements are less available and overall cement fineness has increased over the years to result in more heat output per unit of cement.

For comparison, the total cementitious contents and design strengths of some Reclamation projects are shown in Table 1. Note an almost 600 lb/yd^{3}overall increase in total cementitious materials content from Hoover Dam in 1935 to the RSMC placed in 2008 during the Deer Creek Dam Spillway Modification.

Table 1 – Various Reclamation projects.

Reclamation Structure	Cement Type	Cement, lb/yd^3	Pozzolan, lb/yd^3	Total Cementitious, lb/yd^3	Design Strength (at 28 days), lb/in^2
Hoover Dam	Low Heat (Pre-Type IV)	380	0	380	3000
Grande Coulee Dam	Low Heat (Pre-Type IV)	377	0	377	4750
Glen Canyon Dam	Type IV	188	111 Class F Fly Ash	299	2550
Stony Gorge Dam Modification	Type II	529	176 Class F Fly Ash	705	3700
Deer Creek Dam Modification	Type II	735	183 Class F Fly Ash 46 Silica Fume	964	7000

RSMC is being used in commercial construction for large mat foundations, bridge footings and piers, and for large containment structures. The recent Reclamation applications reviewed in this report include construction of massive concrete placements for seismic resistance at Stony Gorge Dam and Deer Creek Dam Spillway. The design of the structural modifications at Deer Creek Dam Spillway required higher compressive strengths to resist seismic loadings. Additional Reclamation projects where RSMC has been used recently include pumping plant walls, water conveyance structures, thrust blocks for pipelines, and ogee crests for spillways.

Thermal cracking, possible delayed ettringite formation (DEF), and potential strength reductions have all been concerns that arise from the high internal temperature and the temperature differentials between the internal and near-surface concrete. Most of the original work on thermal cracking of mass concrete is related to mass dams or mass structural concretes in power and pumping plants. These concretes normally had cementitious contents ranging from about 300 to 500 lb/yd^3 and the internal temperature rise normally was controlled by traditional means.

Thermal Cracking

Mass concrete is defined in ACI 207 as: "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change, to minimize cracking" [1]. There is no limit to the largest or smallest size dimension of the concrete section to be considered "mass concrete." However, as mass concrete sections increase in section size, thermal cracking can become a problem because the thermal volume contraction of the exterior concrete is restrained by the high temperature expansion of interior concrete. When thermal stresses exceed the tensile strength of the concrete at any given time thermal cracking occurs. To lessen the internal stresses, the temperature gradients of un-reinforced mass concrete sections are normally limited to about 35 ^{o}F. Thermal shock is avoided by preventing rapid surface temperature drops.

The internal temperature of mass concrete is traditionally controlled by (in order of precedence):

1. limiting the total cementitious materials content of the mixture through use of the largest practicable maximum size aggregate
2. using the highest possible percentage of pozzolan
3. lowering the initial placing temperature of the concrete
4. embedding cooling coils in the concrete to dissipate the temperature.

In some cases, the test age for the design strength is changed from 28 to 56 or even 90 days' age (using traditional fog-cured test specimens). Large-sized, reinforced structures in power and pumping plants have historically also controlled internal temperature rise by the same traditional means as mass concrete dams, such as lowering the placing temperature, reducing the cementitious contents, and using 1.5- to 3-inch NMSA structural concrete.

Reinforced structural mass concrete presents a more difficult problem for controlling the interior temperature of mass concrete. Higher design compressive strengths lead to an increase in the cementitious content of the mixture and early strength needs can limit the pozzolan content of the mixture to less than 25 percent by mass of total cementitious materials. Reinforcing steel congestion and pumps used to transport the concrete prevent use of larger NMSA. Unlike traditional mass concrete projects where the volume of concrete predicates the erection of an onsite batch plant, concrete for many RSMC projects are supplied by local ready-mix concrete plants that have smaller aggregates sizes available and limited ability to control the temperature of concrete as batched. In addition, reinforcing steel and formwork make it difficult to embed cooling pipes. High strength or high performance concretes only exacerbate the problems of thermal heat generation due to even higher cementitious contents.

Other Effects of High Internal Concrete Temperature

Delayed Ettringite Formation (DEF)

In addition to thermal cracking, higher internal temperatures may cause deleterious effects in concrete. DEF is associated with some concretes exposed to higher curing temperatures and is caused by the melting of ettringite, a cement hydration product, at temperatures above about 158 ^{o}F. Reformation of the ettringite occurs upon cooling, and when introduced to water, can cause internal volumetric expansion, which could lead to cracking of the concrete at the paste to aggregate or reinforcing steel interface. DEF has been attributed to gypsum contaminated aggregates or cement with high sulfate contents [2]. In Robert Day's 1992 paper he summarized the work of Heinz and Ludwig who concluded that the silica trioxide (SO_3) to di-aluminia oxide (Al_2O_3) ratio (SO_3/Al_2O_3) of the cement may be a critical factor in determining the extent of damage due to DEF. Cementitious materials combinations which contain less than 0.7% $\overline{S}/A$ and less than 2.0% $\overline{S}^2/A$ were less susceptible to DEF[1] [3].

Loïc Divet concluded that DEF is possible and has been observed in many mass concrete structures, but that it is less prevalent than other deterioration mechanisms because up to five or six factors would need to occur simultaneously for DEF to occur. The contributing factors found in the research were; temperature, alkalis in the cement, SO_3 and C3A contents of the cement, aggregate mineralogy, and high humidity conditions [4]. Supplementary cementitious materials, often found in reinforced structural mass concrete to lessen the overall heat signature, also have a mitigating effects on DEF. If used in enough volume, these supplementary cementitious materials can reduce the overall $\overline{S}/A$ and $\overline{S}^2/A$ ratios of the combined cementitious materials and prevent the formation of the damaging expansive product.

Although controlling DEF was one of the initial reasons for limiting the maximum temperature of RSMC to 155 ^{o}F, performance issues may also arise due to high curing temperatures.

Reduction of Strength Potential

Another noteworthy effect of high internal concrete temperatures is that concrete cured at higher temperatures often has a higher early, but lower ultimate

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[1] The Al_2O_3 content in $\overline{S}^2/A$ from the C3A. Common abbreviations for SO_3, Al_2O_3, and $CaO_3Al_2O_3$ in the concrete industry are $\overline{S}$, A, and C3S respectively and are used from this point forward.

compressive strength compared to concrete cured at lower temperatures. The difference in long-term strength also increases with higher curing temperatures [5]. Concrete specimens used for mixture pre-qualification and field acceptance of strength are typically based on specimens fog cured between 70 and 77 ^{o}F. There is a possibility that the strength of in-situ concrete curing at high temperatures may be overestimated by the ideally cured acceptance specimens.

Research Objective

The objective of this research was to determine how much heat is being generated by the higher cementitious contents of the RSMC concrete; if it is contributing to a greater potential for thermally induced cracking in the concrete, and if so, to determine methods to control this cracking in these structures.

Two primary field studies were performed by the Bureau of Reclamation's Materials Engineering and Research Laboratory (MERL) of the Technical Service Center (TSC) for this research program. In-situ temperature monitoring was performed on RSMC placements at two sites undergoing safety of dam modifications; Stony Gorge Dam and Deer Creek Dam Spillway. The internal and near surface temperatures of the mass concrete placements at each dam were evaluated to determine the peak temperatures and thermal gradients within the structures during the first month after placement. Thermal histories and temperature gradients were compared for the two different mixtures. The Deer Creek Dam Spillway seismic buttress was more thoroughly instrumented to evaluate the thermal history of three consecutive vertical placements at the same station, placed about ten days apart.

A laboratory simulation of one RSMC mixture cured at temperatures that matched the in-situ thermal cycle was also performed at the MERL. For the laboratory study, the materials used for the Deer Creek Dam Spillway Modification were obtained from the project concrete supplier for laboratory evaluation. The concrete mixture used for the seismic buttress was replicated in the laboratory to determine thermal properties and to evaluate the compressive strength and modulus of elasticity development for three curing conditions; in-situ cure, adiabatic (no heat loss) cure, and fog cure. The in-situ cure set was cured using temperature controlled environmental chambers programmed incrementally to achieve the same internal concrete temperatures recorded in the field from the center of one placement.

The field and laboratory studies were then compared to two past Reclamation projects that had internal concrete temperature monitoring, and to a simplified numerical method for determining the theoretical adiabatic heat rise of concrete.

Preliminary results of this study were presented by the author at the 2011 Fall American Concrete Institute national convention in Cincinnati, Ohio.

In-Situ Temperature Monitoring

This research program involved monitoring several RSMC placements in the field, followed by laboratory testing of one concrete mixture. The mass concrete placements at Stony Gorge Dam and Deer Creek Dam Spillway modifications provided an excellent comparison for thermal heat generation of RSMC.

Both RSMC sites were monitored at several locations for in-situ temperature rise using embedded temperature monitoring instrumentation. Multiple placements were monitored at the Deer Creek Dam Spillway Modification. The in-situ temperature rise was measured from the time of placing until about 90 days' age. The temperature rise and subsequent cooling history were monitored for both structures.

Temperature monitoring devices were embedded in the RSMC at specific locations to determine the internal temperature, the near surface temperature, and the resulting thermal gradients between the interior and exterior concrete. The Materials Engineering and Research Laboratory (MERL) of the Technical Service Center (TSC) provided all temperature monitoring equipment. Close coordination was maintained between the TSC and field construction staff. The Provo Area Office and Mid-Pacific Regional Construction Office assisted with instrumentation installation and initiated some of the monitoring.

About one dozen temperature monitors were installed at Stony Gorge Dam and about three dozen monitors were installed at the Deer Creek Dam Spillway modification (in three placements). For the Deer Creek Dam Spillway, three consecutive placements were monitored to investigate how heat generated by the lower lift contributed to the temperature rise of the next lift placed above.

The temperature monitors were installed just prior to concrete placement and protected by placing them on the underside of reinforcing bars. MERL coordinated with the field construction inspection staff to access the heavily reinforced sections and locate exit points for instrumentation. The monitors were embedded in the center of the RSMC placements and at about 6- and 18-inches from the exposed faces at the top, and bottom of the placements.

The temperature recording devices provided continuous data monitoring. The intelliRock II™ - Concrete Maturity, Temperature, and Moisture Measurement System manufactured by Engius was used. The proprietary temperature loggers recorded time and temperature automatically through a microprocessor and the data was downloaded with a proprietary data reader [6]. Temperature data was downloaded several times, beginning at about 7 and 28 days, and some records were obtained after about 6 months. However, due to a limited battery life, the readings were eventually terminated.

Stony Gorge Dam Temperature Monitoring

Stony Gorge Dam is an Ambursen slab and buttress dam located near Willows, CA. The dam has forty five buttresses with a maximum height of about 139 ft. The dam was modified in 2007 to 2009 to reinforce the buttresses to resist earthquake loadings as part of the Stony Gorge Dam Modification, Orland Project. Horizontal struts and diaphragm walls were constructed to laterally brace the buttresses. The diaphragm walls were about 6 ft thick, 14 ft wide, and 12 ft high (Figure 1). Eleven concrete temperature monitors were installed in one RSMC placement between buttresses 28 and 29. Reclamation chose the diaphragm wall at this elevation because it provided access to the existing walkway and it minimized interference with the contractor.

Figure 1 - View looking upstream. Three arched and three solid diaphragm walls shown. Formwork installed on three additional diaphragm walls above. Stony Gorge Dam Modification.

The design compressive strength for the Stony Gorge Dam diaphragm walls was 4,000 lb/in^2, however, designers allowed for average 28 day strength of 3,700 lb/in^2 in the specification anticipating the later age strengths would reach the desired strength. The specifications also required that 90 percent of all tests exceed the design strength resulting in a required average strength in excess of about 4,500 lb/in^2.

In the field however, the 4,000 lb/in^2 strength was enforced at 28 days resulting in a total cementitious content of 705 lb/yd^3 with 25 percent replacement of Class F fly ash by weight cement and a NMSA of 1.5-inches. Mass concrete placement temperatures were limited to a maximum of 70 ^{o}F [7]. Concrete mixture proportions and average quality assurance test data are shown in Appendix A.

Stony Gorge Dam Sensor Installation Plan

Reclamation attached sensors to reinforcing steel after the contractor finished installing the reinforcing steel and formwork. Figures 2 through 4 show the typical reinforcing steel layout and placement. Sensors and sensor wires were tied to the underside of reinforcing steel with zip ties. The sensor locations are shown in Appendix B.

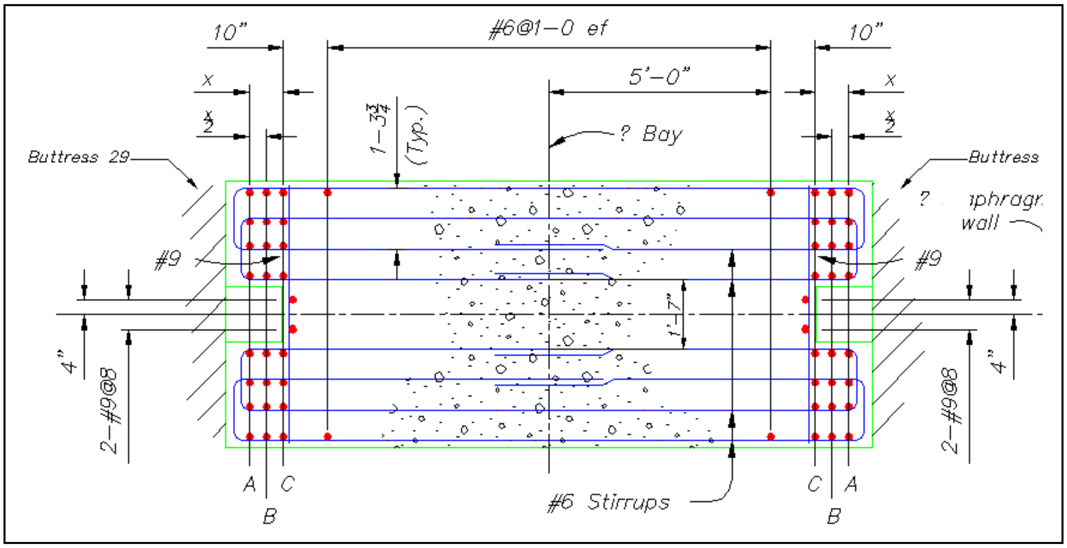

Figure 2 - Plan view of reinforcing steel layout. Stony Gorge Dam Modification.

Figure 3 - Diaphragm wall reinforcing steel prior to formwork installation. Stony Gorge Dam Modification.

Figure 4 - Diaphragm wall prior to concrete placement with temperature sensors installed (yellow wire). Stony Gorge Dam Modification.

Stony Gorge Sensor Results

Temperature monitoring of the diaphragm wall between buttress 28 and 29 started around 9:00 AM on April 24, 2008. The concrete placement temperature was around 52 ºF. Temperature data was downloaded from the sensors at 11 days and

180 days after placement. Maximum temperatures and temperature differentials between crucial locations were then calculated from these values. Table 2 summarizes the temperature data collected within the first week after placement. Values in orange indicate the time period with the highest recorded temperature for each sensor. A maximum temperature of 156 ºF was reached at the center of the diaphragm wall placement at 36 to 51 hours after placement.

Table 2 - Temperatures recorded within 7 days after concrete placement. Stony Gorge Dam Modification.

Sensor ID	Location	12 hrs	1 day (24 hrs)	2 days (48 hrs)	3 days (72 hrs)	7 days (168 hrs)	Max. Temp ºF
SGT8[1]	center of block	113	143.6	156.2	147.2	102.2	**156.2**
SGT2	21" from buttress 28	107.6	131.0	132.8	123.8	89.6	134.6
SGT3	21" from buttress 29	107.6	132.8	134.6	125.6	93.2	136.4
SGT10	6" from top face	95	113.0	105.8	96.8	75.2	113.0
SGT11	18" from top face	105.8	132.8	129.2	116.6	82.4	134.6
SGT1	18" from bottom face	109.4	134.6	140.0	129.2	93.2	140.0
SGT6	6" from u/s form	93.2	113.0	107.6	100.4	78.8	113.0
SGT7	18" from u/s form	107.6	136.4	138.2	129.2	93.2	140.0
SGT4	6" from d/s form	98.6	114.8	114.8	107.6	86.0	116.6
SGT5	18" from d/s form	111.2	138.2	143.6	134.6	98.6	145.4
SGT13	Ambient [2]	53.6	60.8	69.8	73.4	60.8	89.6 [3]

[1] Sensor SGT9, also located at the center, was omitted for clarity due to nearly identical temperatures.
[2] Measured at construction trailer.
[3] Maximum ambient temperature for the first 7 days. Average ambient for this time period was 63 ºF.

Figures 5 and 6 show the temperature and temperature rise, respectively, of each sensor for the first 300 hours (12.5 days) after placement.

The maximum placement temperature of 156 ºF was not high enough above the 155 ºF, the theoretical maximum temperature recommended to prevent DEF, for DEF to be a concern.

Figure 7 shows the calculated temperature gradients from the center of the placement to the other sensors. Concrete 6 inches from the surface exceeded the maximum specified 35 ºF temperature differential when compared to the center of the placement for all three sensors. The maximum differentials calculated were between 41 to 52 ºF. The differential limit was also exceeded for 6 inches from the surface to ambient temperature depending on the time of day. During the cold evenings in the first 6 days after placement the differential between the ambient temperature and concrete 6 inches from the surface was as high as 73 ºF. However, during this time period the formwork remained in place and provided some minimal insulation of the placement.

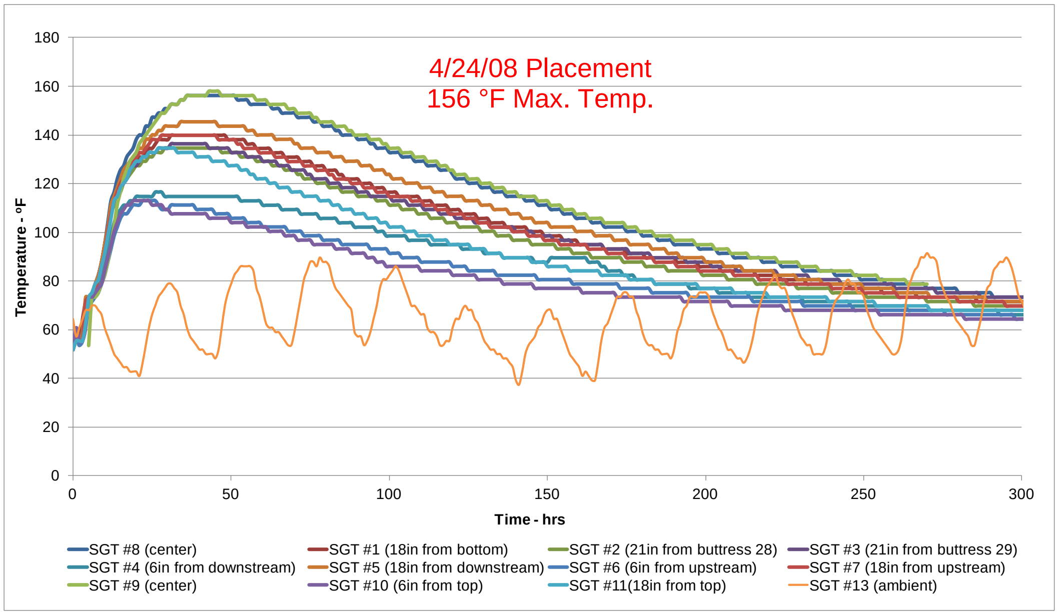

Figure 5 – Recorded temperatures for the first 300 hours after 4/24/08 concrete placement. Stony Gorge Dam Modification.

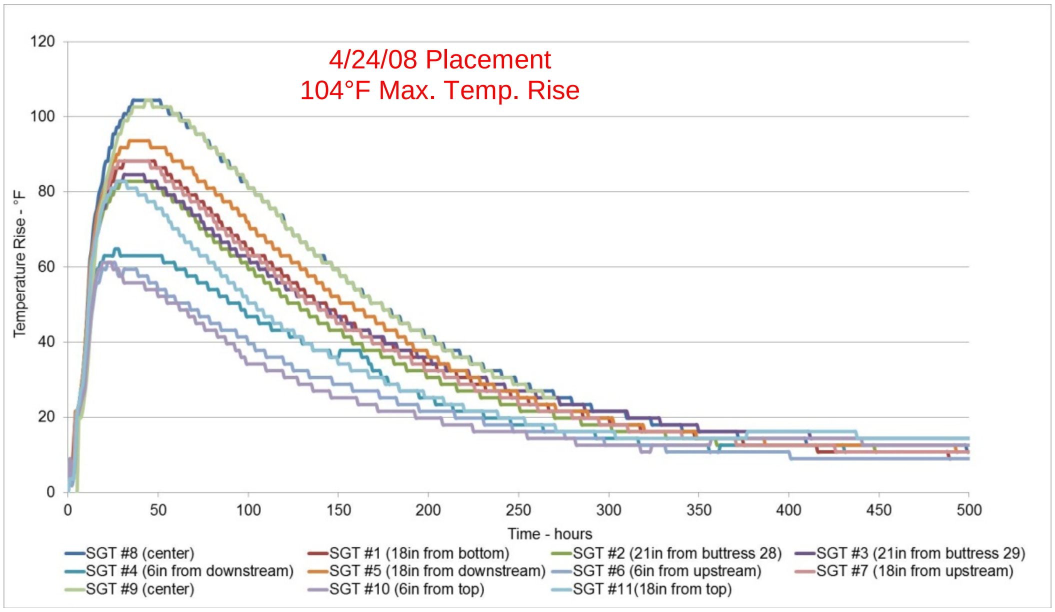

Figure 6 - Temperature rise of sensors in 4/24/08 concrete placement up to 500 hours. Stony Gorge Dam Modification.

Figure 8 shows the calculated temperature gradients for 6 inches from the surface to ambient, 6 inches to 18 inches, and for 18 inches to the center. The difference from the center to the sensors at 18 inches did not exceed the 35 °F. This differential was not exceeded for sensors at 18 inches to the sensors 6 inches from the top on the upstream or downstream face either.

One theory to be further evaluated is whether this maximum temperature differential of 35 °F could be increased as the overall placement size increases in any particular direction. If a thermal analysis approach is used, the application of an incremental temperature differential may be appropriate for very large placements.

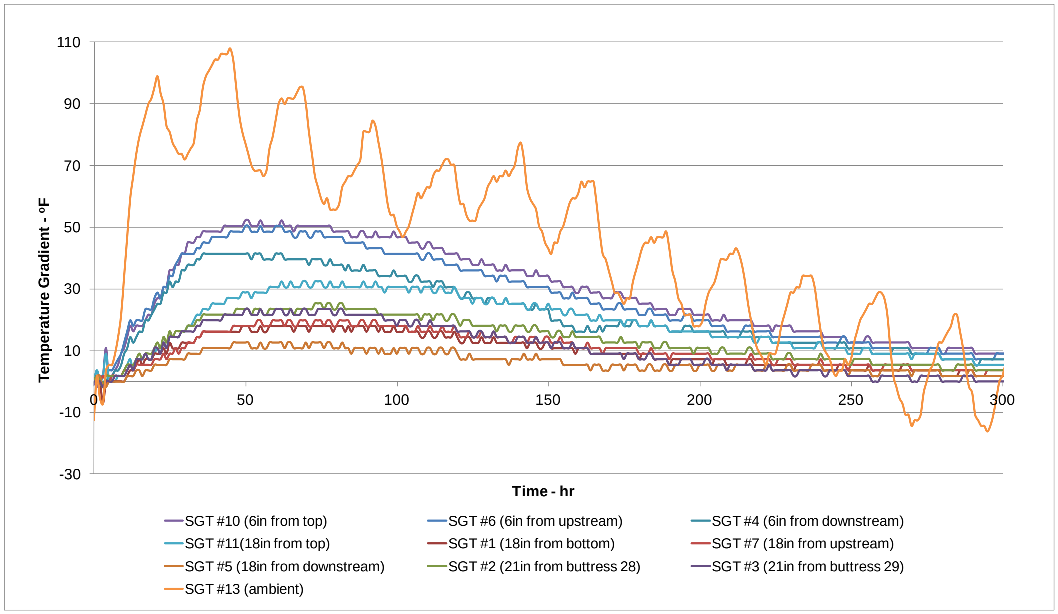

Figure 7 –Gradients from the center for the first 300 hours after 4/24/08 concrete placement. Stony Gorge Dam Modification.

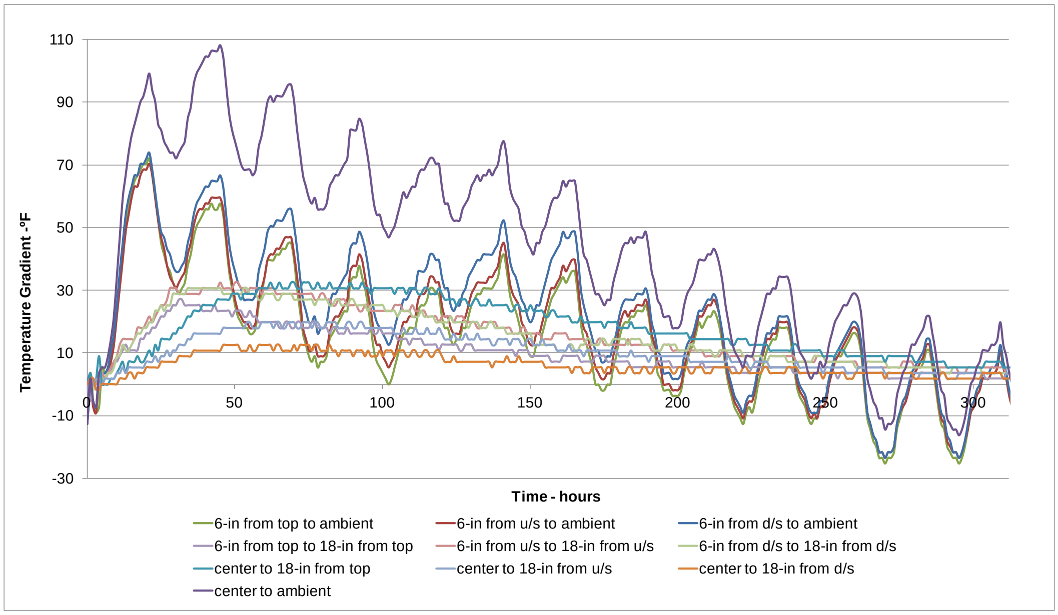

Figure 8 –Comparison of various temperature differences between sensors for the first 300 hours after 4/24/08 concrete placement. Stony Gorge Dam Modification.

Note that gradients for the sensors when compared to ambient temperature are the most severe at night and each evening a peak is depicted on the graph.

Deer Creek Dam Spillway Temperature Monitoring

Deer Creek Dam is a zoned, earthfill embankment dam and part of the Provo River Project northeast of Provo, Utah. The dam has a concrete chute spillway located at the right abutment controlled by two radial gates. In 2009, a modification to strengthen the concrete spillway walls for seismic loadings was initiated. A high-strength, RSMC seismic buttress was constructed, encasing the original counter-fort walls (Figure 8).

Figure 9 - RSMC seismic buttress encases the original left spillway wall counterforts. Deer Creek Dam Spillway Modification.

Design strength for the Deer Creek Dam Spillway buttresses was 7,000 lb/in^2 at 28 days, resulting in a required average strength of 8,400 lb/in^2 for 90 percent of all tests to exceed the design strength. Due to the heavily congested reinforcing steel and need to pump the concrete, the nominal maximum size aggregate (NMSA) was 1-inch and the total cementitious content was 964 lb/yd^3 with 20 percent substitution of Class F fly ash and 5 percent silica fume. The cement content was limited to a maximum of 750 lb/yd^3. Concrete was provided by Westroc, Inc. and pumped by Dudley Concrete Pumping. Concrete mixture proportions and average quality assurance test data are shown in Appendix A.

Three successive and overtopping concrete placements were planned by the contractor spanning the length of the spillway on each side (Figure 9). The placements varied in size from 170 to 150 cubic yards. The first lift began on April 25, 2008 with about 9 or 10 days between each subsequent lift.

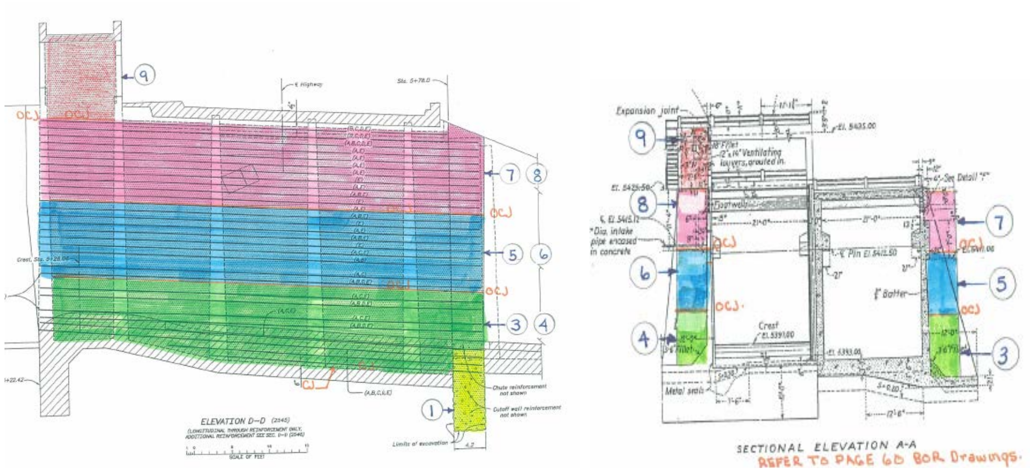

Figure 10 - RSMC seismic buttress contractors concrete placement plan, elevation (left), section (right), placement numbers circled. Deer Creek Dam Spillway Modification.

Each buttress section was approximately 6.5 ft thick, 7.5 to 12.5 ft high, with 11 ft between counterfort walls. Each lift either fully (bottom) or partially (top) incorporated the existing counterforts. The concrete is heavily reinforced with vertical and horizontal reinforcing steel including four bundled No. 11 bars installed vertically on about 9-inch centers. Due to the reinforcement congestion post-cooling was not an option.

Deer Creek Dam Spillway Sensor Installation Plan

Temperature sensors[2] were installed to investigate the temperature rise of RSMC followed by laboratory tests to estimate strength under the field temperature cycle.

Twenty-two concrete temperature monitoring devices were installed in three placements on the left spillway wall buttress. These placements were identified as Nos. 4, 6, and 8 in the contractor's concrete placement plan. Sections of the placements that were monitored were located between buttresses at Sta. 5+28.6 and Sta. 5+46.6. These locations were chosen because access to the monitoring wires could be obtained via the highway bridge above. The plan, profile, and elevation sections of temperature monitoring locations for placements Nos. 4 and 6 are shown in Appendix B. Instrumentation of the third placement (No. 8) was added later and followed the same configuration as the previous placements.

The instruments were installed after the contractor finished installing reinforcing steel and as the forms were being erected. Although the block size between buttresses was almost the same as at Stony Gorge the amount of reinforcing steel was significantly greater. Figures 11 and 12 show the typical reinforcing steel layout. In order to ensure the sensors remained within the designated measurement locations during concrete placement, the individual sensors were fixed to extra reinforcing steel bars with both zip ties and duct tape as shown in Figure 13. These extra bars were installed due to the quantity and close spacing

[2] Because of availability, a few of the sensors supplied by MERL were maturity sensors, however, only the temperature recording function was utilized.

of reinforcing steel that prevented access into the forms. Instrumentation was placed at the desired locations and secured to the reinforcing steel with zip ties. Sensors and sensor wires were tied to the underside of reinforcing steel bars to avoid damage during concrete placement. All wires were run through a PVC conduit to the top of the wall and then to the old highway bridge for access. Figures 14 through 17 show the installation of temperature sensors in the Deer Creek Dam left spillway buttress. The sensors designated to measure the ambient sun and shade temperatures were secured to upstream fencing located on the old highway bridge as shown in Figure 18.

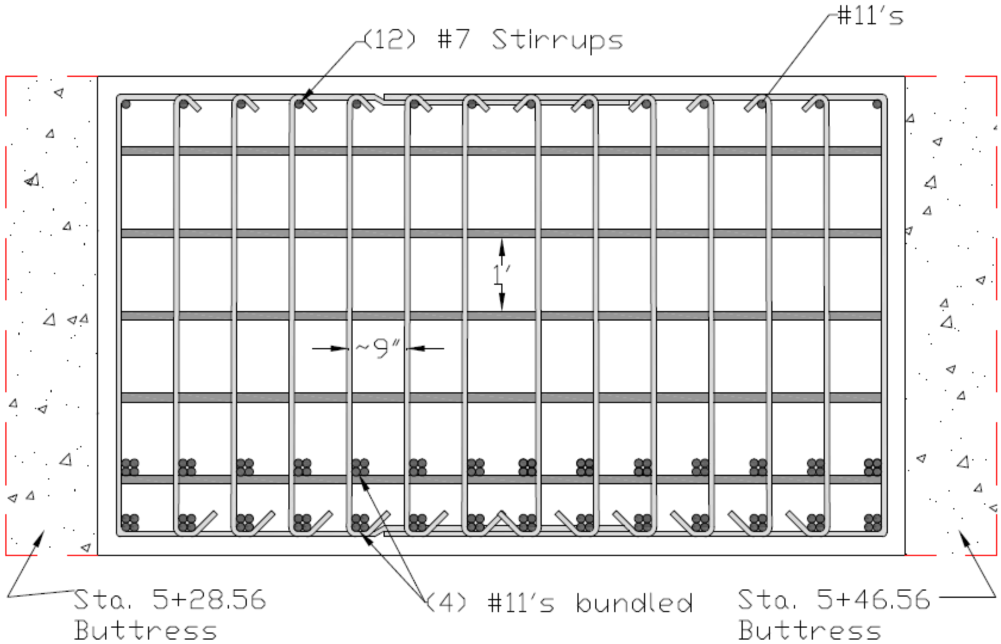

Figure 11 - Typical reinforcing steel layout. Deer Creek Dam Spillway Modification.

Figure 12 - Reinforcing steel for left seismic wall buttress. Deer Creek Dam Spillway Modification.

Figure 13 - Temperature sensors attached to reinforcing steel ready for placement in the left seismic buttress wall. Deer Creek Dam Spillway Modification.

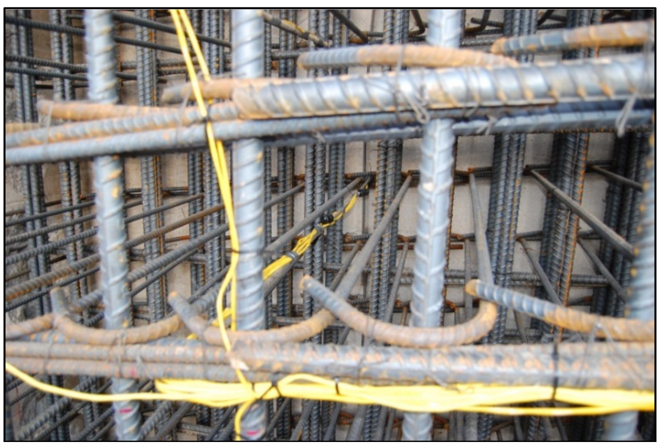

Figure 14 - Temperature sensors embedded in RSMC, left seismic wall buttress. Deer Creek Dam Spillway Modification.

Figure 15 - Installation of temperature sensors and wire leads. Deer Creek Dam Spillway Modification.

Figure 16 - Bundled lead wires being routed into 2 in. diameter pipe. Deer Creek Dam Spillway Modification.

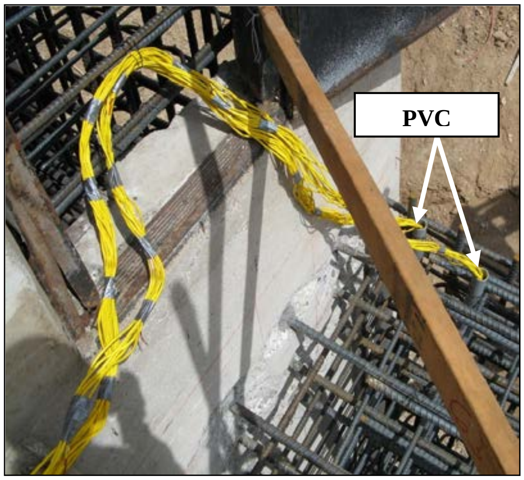

Figure 17 - Bundled lead wires routed to top of left seismic wall buttress. Deer Creek Dam Spillway Modification.

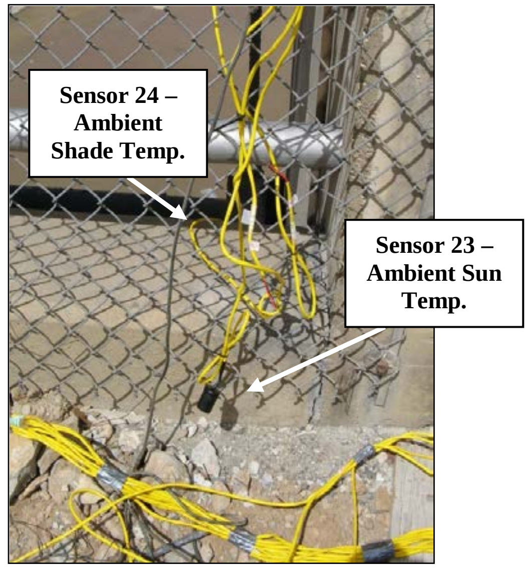

Figure 18 - Ambient temperature monitors for left seismic wall buttress. Deer Creek Dam Spillway Modification.

Concrete Placing for Deer Creek Dam Spillway Modification

Figures 19 and 20 show RSMC for Placement No. 4 (bottom lift) placed on April 25, 2008. The recorded placing temperature of the concrete for all buttresses ranged from 51 ^{o}F to 69 ^{o}F. Fresh and hardened properties of concrete were tested by Reclamation Provo Area Office laboratory personnel. Initial concrete deliveries had slumps ranging from 5 to 7 inches. The concrete was placed in approximately 18 inch lifts and consolidated using internal vibrators.

Figure 19 - Six-inch slump high strength RSMC flowing through bundled No. 11 reinforcing steel bars. Spacers were placed between the bars to allow concrete to fill in the void. Deer Creek Dam Spillway Modification.

Figure 20 - Six-inch slump high strength RSMC nearing temperature instrumentation wires. Deer Creek Dam Spillway Modification.

The average initial placing temperature recorded in the field for each of the monitored placements is presented in Table 3. Mass concrete placement temperatures were originally limited to a maximum of 70 °F in the project specifications [8]. Based on the in-situ temperature readings at the lower two lifts (placement No. 4 and 6), the initial concrete placing temperature limits for the remaining RSMC was reduced to about 50 to 55 °F. Means to control thermal cracking performed by the contractor included lowering the initial concrete placing temperature with ice and providing thermal insulation blankets.

Table 3 – Summary of placement information. Deer Creek Dam Spillway Modification.

Placement Number	Date	Initial Placement Temperature (Avg. Ambient 50 ºF)
No. 4	4/25/08	57 ºF
No. 6	5/6/08	66 ºF
No. 8	5/14/08	53 ºF

Deer Creek Dam Spillway Sensor Results

Tables 4 through 6 summarize the temperature data collected within the first week after placement for each of the monitored placements. Values in orange indicate the time period with the highest recorded temperature for each sensor.

Table 4 - Temperatures recorded within 7 days after concrete placement No. 4. Deer Creek Dam Spillway Modification.

Sensor ID	Location	12 hrs	1 day (24 hrs)	2 days (48 hrs)	3 days (72 hrs)	7 days (168 hrs)	Max. Temp ºF
DCT15	center	118.4	161.6	163.4	158.0	114.8	165.2
DCT21	6" from 5+28.56	95.0	129.2	132.8	129.2	105.8	134.6
DCT22	6" from 5+46.56	89.6	125.6	131.0	127.4	102.2	131.0
DCT10	6" from top face	111.2	132.8	118.4	107.6	73.4	134.6
DCT18	6" from bottom face	66.2	98.6	105.8	100.4	82.4	105.8
DCT17	6" from interior face	71.6	107.6	116.6	114.8	93.2	116.6
DCT12	6" from exterior face	105.8	141.8	145.4	138.2	89.6	147.2
DCT23	ambient sun	41.0	86.0	75.2	78.8	71.6	98.6
DCT24	ambient shade	44.6	48.2	46.4	60.8	46.4	73.4

Table 5 - Temperatures recorded within 7 days after concrete placement No. 6. Deer Creek Dam Spillway Modification.

Sensor ID	Location	12 hrs	1 day (24 hrs)	2 days (48 hrs)	3 days (72 hrs)	7 days (168 hrs)	Max. Temp ºF
DCT6	center	143.6	174.2	170.6	154.4	100.4	177.8
DCT19	6" from 5+28.56	102.2	131.0	140.0	129.2	93.2	140.0
DCT20	6" from 5+46.56	114.8	138.2	138.2	129.2	95.0	140.0
DCT1	6" from top face	141.8	152.6	127.4	111.2	73.4	152.6
DCT9	6" from bottom face	125.6	150.8	143.6	132.8	98.6	150.8
DCT8	6" from interior face	111.2	131.0	125.6	114.8	82.4	132.8
DCT3	6" from exterior face	140.0	156.2	127.4	120.2	82.4	158.0
DCT23	ambient sun	50.0	55.4	69.8	51.8	80.6	104.0
DCT24	ambient shade	59.0	53.6	50.0	48.2	42.8	71.6

Table 6 - Temperatures recorded within 7 days after concrete placement No. 8. Deer Creek Dam Spillway Modification.

Sensor ID	Location	12 hrs	1 day (24 hrs)	2 days (48 hrs)	3 days (72 hrs)	7 days (168 hrs)	Max. Temp ºF
DCT30	center	102.2	159.8	163.4	158.0	127.4	165.2
DCT29	6" from top face	100.4	127.4	145.4	138.2	98.6	145.4
DCT31	6" from bottom face	105.8	138.2	136.4	129.2	107.6	140.0
DCT32	6" from exterior face	60.8	77.0	113.0	107.6	51.8	120.2
DCT23	ambient sun	51.8	66.2	78.8	80.6	42.8	114.8
DCT24	ambient shade	55.4	57.2	57.2	60.8	48.2	82.4

Figure 20 shows the temperature data as a function of time collected from all the sensors in all three placements up to 800 hours from the start of placement No. 4. A maximum temperature of 178 ºF (and maximum temperature rise of 122 °F as shown in Figure 23) was recorded at the center of placement No 6. between 28 to 34 hours after placement. The maximum temperature would likely have been higher had the contractor not taken measures to reduce the initial placing temperature or if the placements occurred during the summer when the aggregate stockpiles would be warmer.

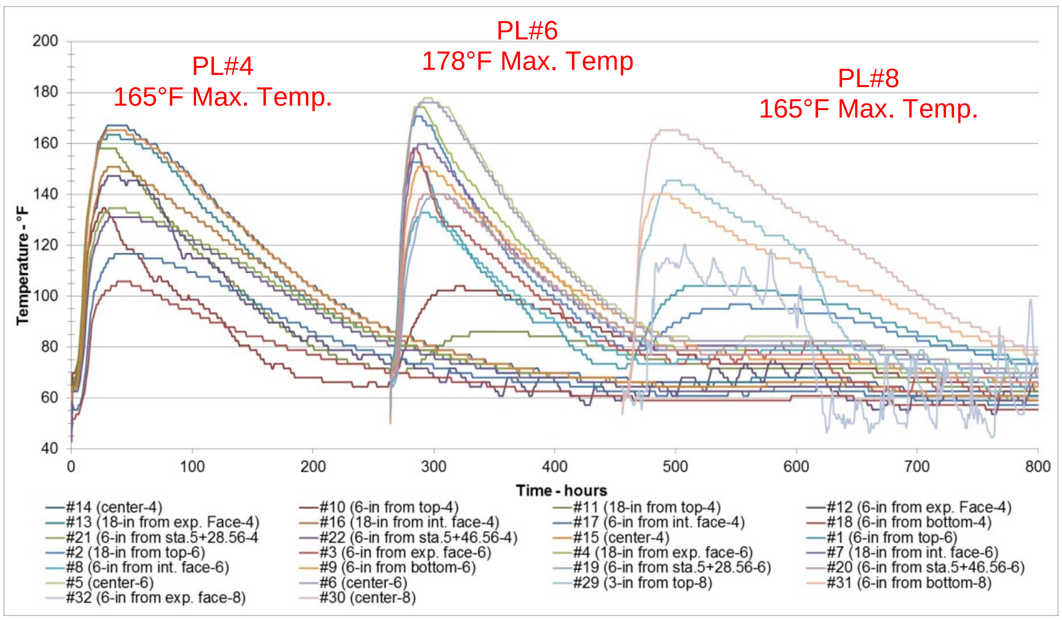

Figure 21 - Temperature data collected from all sensors in placement No.'s 4, 6, and 8 up to 800 hours. Deer Creek Dam Spillway Modification.

The influence a successive concrete placement has on the temperature curve of the previous concrete lift is shown in Figure 22. Note the sensor 6 inches below the top of placement No. 4, which had already cooled to about 65 ºF, increased to a new peak of 104 ºF (a 39 ºF increase), due to the temperature rise of placement No. 6 above. The sensor 18 inches below the top of placement No. 4 also increased but by only about 16 ºF. Placement No. 6 was affected similarly.

Placement No. 4 provided both insulation for and heat addition to placement No. 6 resulting in an increase of the center peak temperature of placement No. 6 by about 13 ºF.

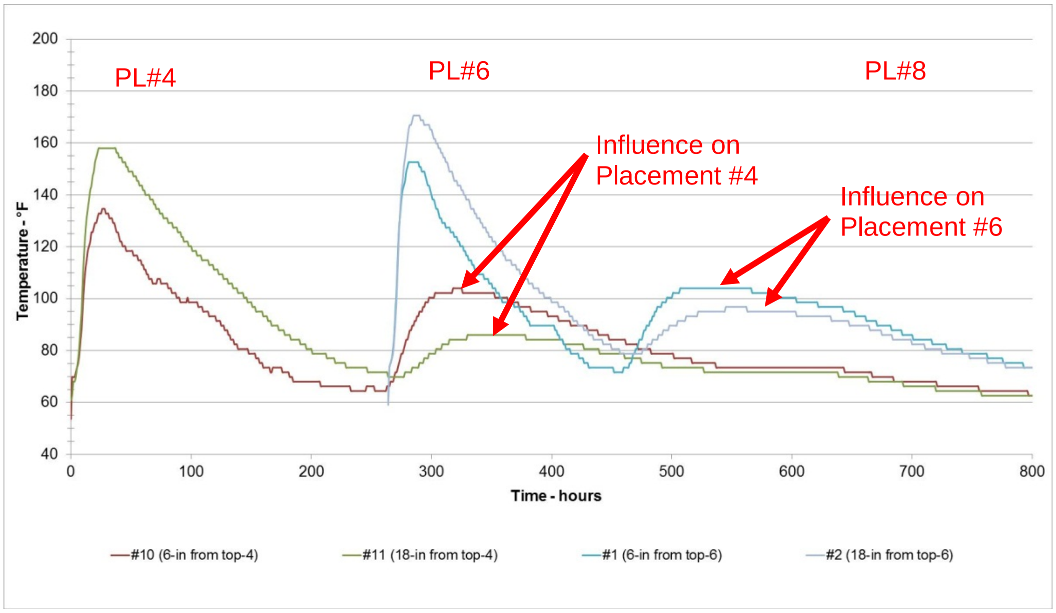

Figure 22 - Temperature data during placement No.'s 4, 6, and 8. Note influence on previous placements by the subsequent placement temperatures. Deer Creek Dam Spillway Modification.

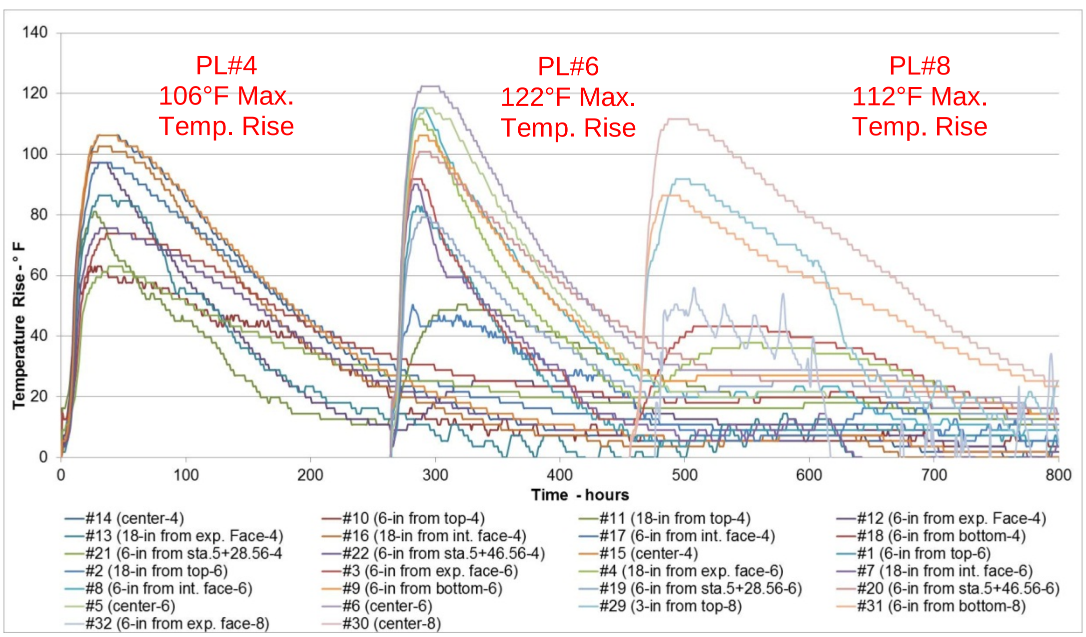

Figure 23 - Temperature rise of all sensors in placement No.'s 4, 6, and 8 up to 800 hours. Deer Creek Dam Spillway Modification.

The specifications required that the temperature differential between the center of the placement and the outside face of the concrete be less than 36 ºF. Insulation blankets were used to minimize heat loss from the surface and reduce the temperature differential from the center to the outside face. Keeping the blankets in place during the windy winter weather proved challenging. Every sensor in all the Deer Creek Dam spillway placement exceeded a temperature differential of 36 ºF when compared to ambient temperature.

Blankets and formwork provided insulation so the temperature differential from the center to the outside face was somewhat less than it would have been if the placements were left exposed. However, the sensors 6 inches from the outside face still recorded a temperature differential of more than 50 ºF to the center sensor as shown in Figure 24.

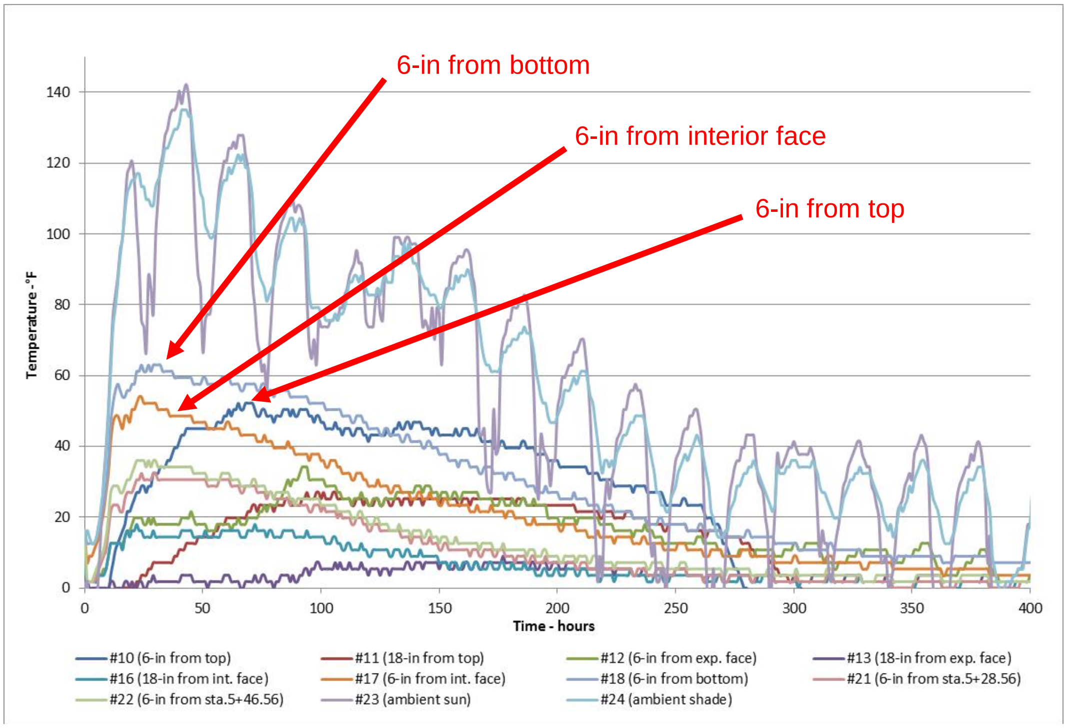

Figure 24 - Temperature gradient from center for all sensors in placement No. 4 up to 400 hours. Maximum temperature of highlighted sensors, from top to bottom, are 63 ºF, 52 ºF, 52 ºF. Deer Creek Dam Spillway Modification.

All three placements reached internal temperatures above 155 ºF, the theoretical maximum temperature recommended to prevent DEF. The chemistry of the low-alkali Type II-V cement plus pozzolan combination was checked using Heinz and Ludwig's proposed equations for limiting the $\overline{S}/A$ and $\overline{S}^2/A$ ratios to 0.7% and 2.0%, respectively [3]. Both ratios were below the recommended limit, thus DEF was not a concern.

Summary of In-situ Monitoring

The peak interior temperature of the Stony Gorge RSMC was about 20 ºF less than the Deer Creek Dam Spillway RSMC, even though the average ambient temperature at Stony Gorge Dam was 10 ºF higher than at Deer Creek Dam. At

Stony Gorge, the lower cementitious content was effective in reducing the peak internal temperature.

Both the Stony Gorge and Deer Creek Dam placements exceeded the 35 ^{o}F maximum specified temperature differential when comparing the sensors at the center of the placement to the sensors 6 inches from the surface. The differentials peaked at about 50-60 ^{o}F. At Deer Creek Dam insulation blankets were used to minimize heat loss from the surface and reduce the temperature differential from the center to the outside face, also reducing the potential for thermal shock at night.

Although a thorough surface evaluation of the concrete was not conducted by the author, no major cracking has been formally reported to the MERL. At Deer Creek Dam some of the placements are covered with backfill and at Stony Gorge Dam access to the face of the dam is limited. It is recommended that the concrete surfaces from these dam modifications be inspected at some point in the future to record the location of any existing cracks. The high concrete strengths at earlier ages may be enough to overcome the internal stresses from the temperature differentials. A complete thermal analysis to determine the measureable effects of these temperature gradients is recommended when using RSMC for critical structures. This thermal analysis should be used to set reasonable limits for temperature differentials, placement sizes, and concrete placement sequencing.

Historical Temperature Monitoring of RSMC

The in-situ temperature rise for other Reclamation structures has been recorded in the past. For this study the temperature data for concrete placements which resembled RSMC rather than traditional mass concrete were evaluated and compared to Stony Gorge and Deer Creek Dam temperature monitoring results.

The intake structures at New Waddell Dam, in Central Arizona Project, in Arizona were instrumented in 1990 after cracking in larger placements was noticed. The concrete was designed with a high cementitious materials content to compensate for relatively weak aggregate [9]. In-situ temperature development at New Waddell Dam was recorded for three NMSA mixtures, as follows:

- ¾-inch NMSA placed in the intake structure bridge pier
- 1½-inch NMSA in the 5 ft thick intake structure walls
- 3-inch NMSA in the 5 to 7 ft thick intake structure footing

Mass structural concrete placement temperatures were also recorded at the Durango Pumping Plant, Animas-La Plata Project, in Colorado in 2005 due to concerns of potential cracking in the primarily underground structure which is exposed to a high water table [10]. The in-situ temperature rise of 2-inch NMSA mass structural concrete floor slabs was recorded for mixtures containing both Type II cement and Type V cement. It is not possible to directly compare the temperature rise for the two cements as the Type II cement placements were in the winter and the Type V cement was used during the summer placements.

Table 7 summarizes the average recorded in-situ temperatures of RSMC for the three New Waddell Dam mixtures and the two Durango Pumping Plant mixtures. Values in orange highlight the period during which the maximum temperature for each placement was recorded. In-situ temperature data at New Waddell Dam is shown in Figure 25. The ambient air temperatures in Arizona were high increasing the potential for extremely high internal temperatures. In-situ temperature data from Durango Pumping Plant is shown in Figure 26.

Table 7 - Temperatures recorded within 7 days after concrete placement.

Structure	Location	1 day (24 hrs)	2 days[1] (48 hrs)	3 days[1] (72 hrs)	7 days (168 hrs)	Max. Temp °F
New Waddell Dam	Bridge Pier	160	176	172	125	176
New Waddell Dam	Intake Tower Wall	140	166	170	139	170
New Waddell Dam	Intake Tower Footing	130	148	152	134	152
Durango Pumping Plant	Pumping Plant Floor Slab (Summer Type V Cement)	135	144	133	104	144
Durango Pumping Plant	Pumping Plant Drain Slot (Winter Type II Cement)	107	102	93	66	109

[1]Approximate age for some sensors where data was not recorded every hour.

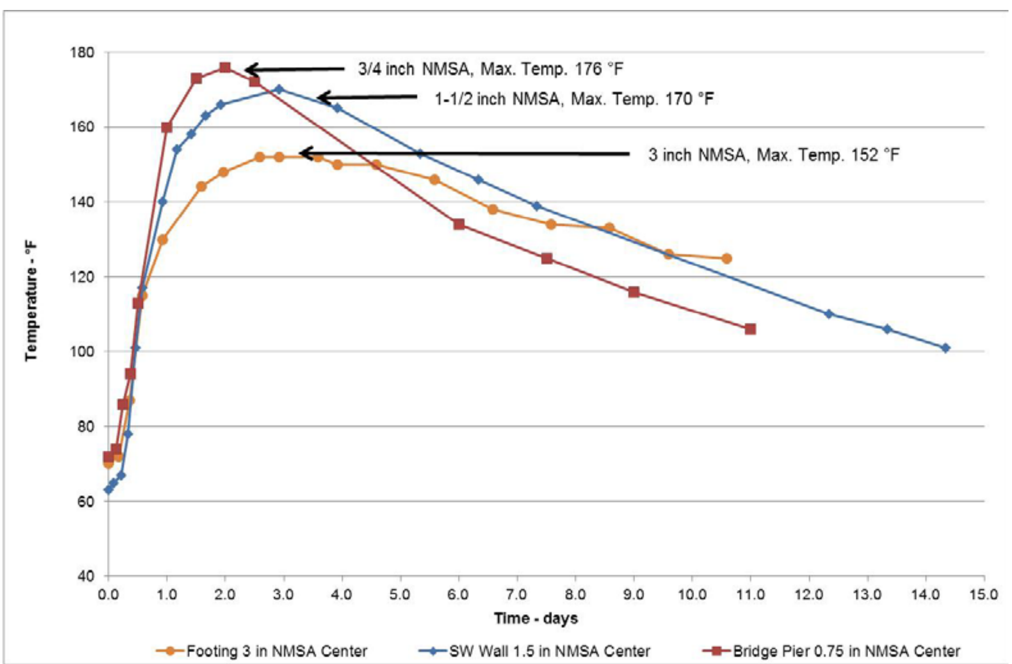

Figure 25 – Average temperature data from New Waddell Dam placements with various NMSA. New Waddell Dam, Central Arizona Project, AZ.

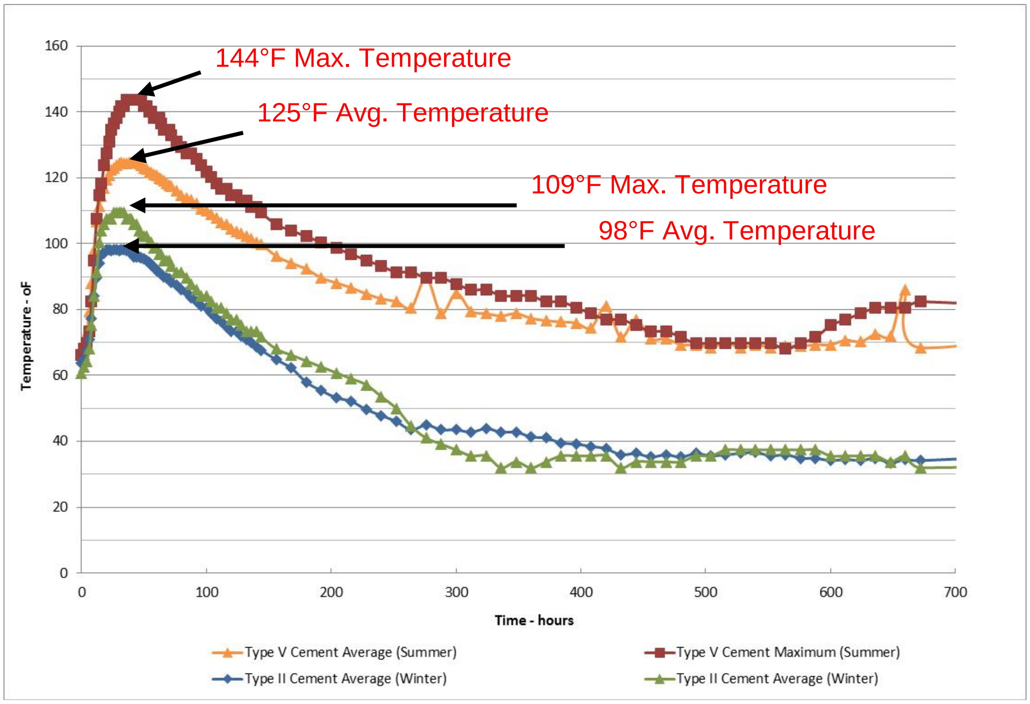

Figure 26 - Average temperature data from Durango Pumping Plant placements with various cement types is different seasons. Durango Pumping Plant, Animas-La Plata Project, CO.

Summary of In-situ Temperatures

A summary of the test data discussed thus far is presented in Table 8. Peak temperatures for all of these placements occurred within the first three days.

Table 8 – Summary of recorded temperatures and maximum temperature rise of various concrete placements.

Structure	1 day (24 hrs)	2 days (48 hrs)	3 days (72 hrs)	7 days (168 hrs)	Max. Temp ºF	Temperature Rise
Stony Gorge	143.6	156.2	147.2	102.2	156.2	104
Deer Creek Placement #4	161.6	163.4	158.0	114.8	165.2	113
Deer Creek Placement #6	174.2	170.6	154.4	100.4	177.8	120
Deer Creek Placement #8	159.8	163.4	158.0	127.4	165.2	109
New Waddell Dam	160	176	172	125	176	100
	140	166	170	139	170	91
	130	148	152	134	152	96
Durango Pumping Plant	135	144	133	104	144	77[1]
	107	102	93	66	109	49

[1] *Type V cement plus pozzolan, all other placements were Type II or Type II/V cement plus pozzolan(s).*

Methods to Determine Anticipated Temperature Rise

ACI Committee 207, *Mass Concrete* has been working on, but has not released, a document to review RSMC. Some general practices have been used during construction of various projects, but not analyzed and documented thoroughly. No guidelines are available to the designer to deal adequately with RSMC.

Currently many designers and contractors use Figure 4.1 of ACI 207.2R *Thermal and Volume Change Effects on Cracking of Mass Concrete* when preparing their temperature control plan for mass concrete [11]. With the exception of Durango Pumping Plant all of the recorded temperature rises shown in Table 8 exceed those presented in ACI 207.2R Figure 4.1. The 1960's graph depicts the adiabatic temperature rise of concrete mixtures with a total cement content of 376 lb/yd^{3}for Type I, II, III and IV cements. This graph was based on older cements with a lower Blaine fineness compared to cements available today and concretes made with much lower total cement contents than used in most current RSMC applications. Also, this graph does not include mixtures containing supplementary cementitious materials (pozzolans).

For a Type II cement, ACI 207.2R Figure 4.1 indicates about a 60 ºF temperature rise at 28 days. For the SRMC mixtures evaluated in this report thus far, the temperature rise ranged from 49 to 120 ºF and the average cementitious content was about 790 lb/yd^3 (compared to 376 lb/yd^3 for ACI 207.2R Figure 4.1). Extreme caution should be used when applying this graph to RSMC mixtures. At a minimum, the design of critical Reclamation features that contain RSMC should include mixture proportioning investigations and adiabatic temperature rise testing using materials from the project area. Results of the adiabatic temperature rise test should be used in a thermal analysis to assure all stresses developed stay within the linear range for the concrete.

The data from ACI 207.2R Figure 4.1 is also used in a simplified calculation method presented in John Gadja's PCA document *Mass Concrete for Buildings and Bridges* which estimates the potential temperature rise of a mixture [12]. In this method, the heat contributed by all cementitious materials is calculated on a cement equivalent basis. Since various pozzolans contribute different amounts of heat to the mixture they are counted as a percentage of a unit of cement.

The cement equivalent is then multiplied by a factor that is equivalent to the 28-day temperature rise for a given cement type (from ACI 207.2R Figure 4.1) divided by the 376 lb/yd^3 of cement. Total cement contents and the effects of various amounts of pozzolans (including fly ash, silica fume and ground-granulated blast furnace slag) have a large effect on the accuracy of this method, especially when large volumes of supplementary cementitious materials are used.

Concrete proportions and temperature data from nine field placements and eight laboratory adiabatic temperature rise tests were compared to this simplified method. Overall, the method would get a designer close to the anticipated temperature rise for a given mix, however, it does seem to under predict temperatures at lower total cementitious contents and over predict them at higher total cementitious contents as shown in Figure 28. A summary of the 17 data sets used in Figure 28 are presented in Appendix C.

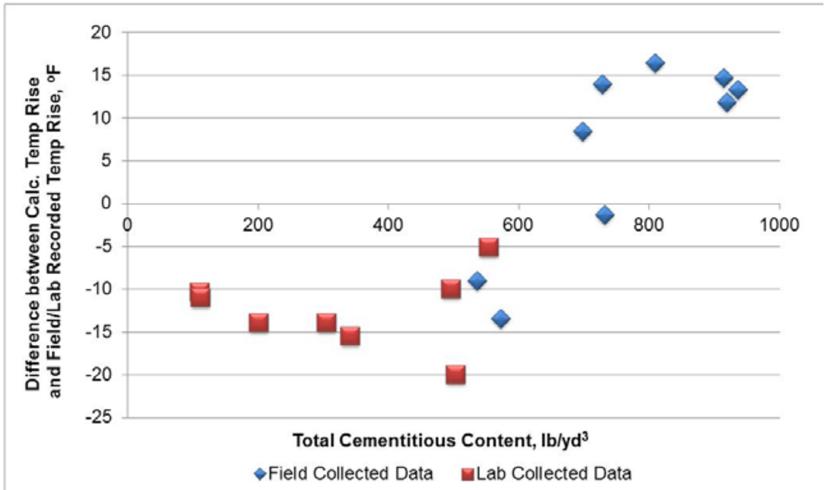

Figure 27 – Comparison of simplified calculation for temperature rise to field and laboratory temperature rise data.

There are additional methods to predict temperature rise, through analytical means as shown in ACI 207, or by analysis with semi-adiabatic testing. Evaluation of these methods was not conducted for this study. Future research is recommended to evaluate other methods to predict adiabatic temperature rise. It is also recommended that Reclamation consider assisting ACI with updating Figure 4.1of ACI 207.2R. Graphs that include data for mixtures with higher total cementitious contents and containing pozzolans would better reflect the current state of practice for RMSC. Historically, Reclamation has been a leader in conducting this type of research and testing. Reclamation has a unique capability to test the full adiabatic heat rise of concrete. Currently there are few, if any, commercial testing laboratories that have the equipment required to perform this type of testing. The benefits to Reclamation extend beyond the design of critical structures to a potential use for future Safety of Dams modifications.

Laboratory Study

Concrete making materials from the batch plant that supplied concrete for Deer Creek Dam Spillway Modification were submitted by the contractor for additional laboratory testing as requested by Reclamation. The RSMC mixture used for the project was replicated and laboratory tests were performed to compare the mass cured strength performance to the standard laboratory fog cured strength performance.

Adiabatic Temperature Rise

The adiabatic temperature rise of the Deer Creek Dam Spillway Modification concrete mix was tested in the laboratory using USBR 4911, *Temperature Rise of Concrete* [13] and compared to maximum in place temperature recorded at the project. The adiabatic temperature rise was found to be 110 ^{o}F and with a peak temperature of 172 ^{o}F as shown in Figure 32. The peak recorded temperatures in the field ranged from 165 to 178 ^{o}F. This illustrates that large concrete placements should be expected to reach, and potentially even exceed, the full adiabatic heat rise potential without post cooling.

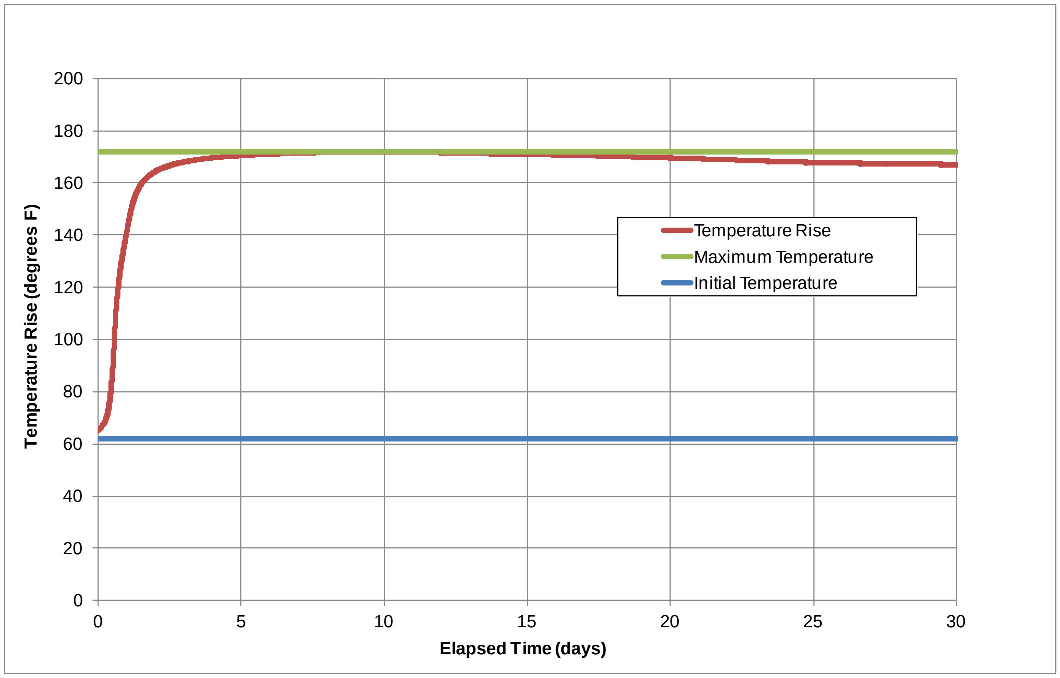

Figure 28 – Adiabatic temperature rise of concrete for laboratory mixture. Deer Creek Dam Spillway Modification.

Note that at this high of an overall temperature (62 ^{o}F starting temperature plus 110 ^{o}F temperature rise) the environmental chamber struggled maintain the adiabatic temperature which is why after about 14 days the graph shows a slight decrease in temperature.

Strengths at Different Curing Temperatures

Performance of concrete designed according to ACI 318, Building Code for Structural Concrete, is normally based on standard test specimens moist cured at 73 ^{o}F [14]. RSMC may undergo substantially different temperature cycles due to the heat generated within the concrete mass that cannot be dissipated which can have an effect on both the rate of strength development and the ultimate strength. Under most conditions, concrete cured at higher temperatures usually has higher early compressive strength, but lower ultimate strength. Thus, the strength of fog-cured test specimens may differ from the actual in-place concrete. The building code does take into consideration different field cure environments for the purposes of evaluating construction loads and requirements for forming concrete, but not for general acceptance.

As a part of this test program, the compressive strength and elastic properties of RSMC were determined for three curing conditions; simulated in-situ cure, adiabatic cure, and fog cure. The simulated in-situ cure test specimens were cured using temperature controlled environmental chambers programmed incrementally to achieve the same internal concrete temperatures recorded in the field. Adiabatic cure test specimens were cured in a temperature controlled environmental chamber simultaneously performing USBR 4911 for the mix. Traditional fog-cured test specimens were cured between 70 and 77 ^{o}F at 100% relative humidity as per ASTM C39 [15]. Photos of laboratory testing are presented in Appendix D.

Four-inch diameter by eight-inch long (4- by 8-inch) cylinders were used for this research program to minimize the volume of concrete needed. Standard 6- by 12-inch test cylinders were also tested for comparison with the 4- by 8-inch fog cured specimens. Specimens were tested for compressive strength and elastic properties at ages ranging from 12 hours up to 90 days' age to evaluate the rate of strength and elastic properties gain under the different curing conditions. Table 9 and Figure 29 present the results of the compressive strength tests while Table 10 and Figure 30 present results of the elastic properties tests. Complete test data can be found in Appendix E.

Both the 4- by 8-inch and 6- by 12-inch, fog-cured test specimens displayed comparable strength and comparable strength gain through 90 days' age. The adiabatic and in-situ cured test specimens, however, showed a significant early rise in compressive strength that plateaued after only 3 days' age. Little compressive strength gain was noted beyond 7 days' age. The variability between specimens at the same age was also high, as shown in Appendix E.

Fortunately, the average 3-day compressive strength for the in-situ cured concrete is comparable to the average fog-cured 28-day compressive strength. The quick strength gain of the in-situ cured specimens also did not appear to have any deleterious effects on the concrete as described in the cursory petrographic examination of the concrete presented in Appendix F.

Table 9 – Average compressive strengths for the Deer Creek Dam Modification mixture under varying curing conditions, lb/in^2.

Age	Simulated In-situ Temp Cure 4-by 8-in Specimen	Adiabatic Cure 4-by 8-in Specimen	Fog Cure 4-by 8-in Specimen	Fog Cure 6-by 12-in Specimen	Field QA Specimens 6-by 12-in Specimen	Specified F'c
12 hrs	2433	-	-	-	-	7000
24 hrs	5170	4013	2920	2850	-	
3 day	6177	6837	4590	4150	-	
7 day	6330	6477	5170	4850	5400	
14 day	6473	6947	6220	-	-	
28 day	6343	6543	6550	6810	7010	
90 day	6313	7100	7450	7590	7730	

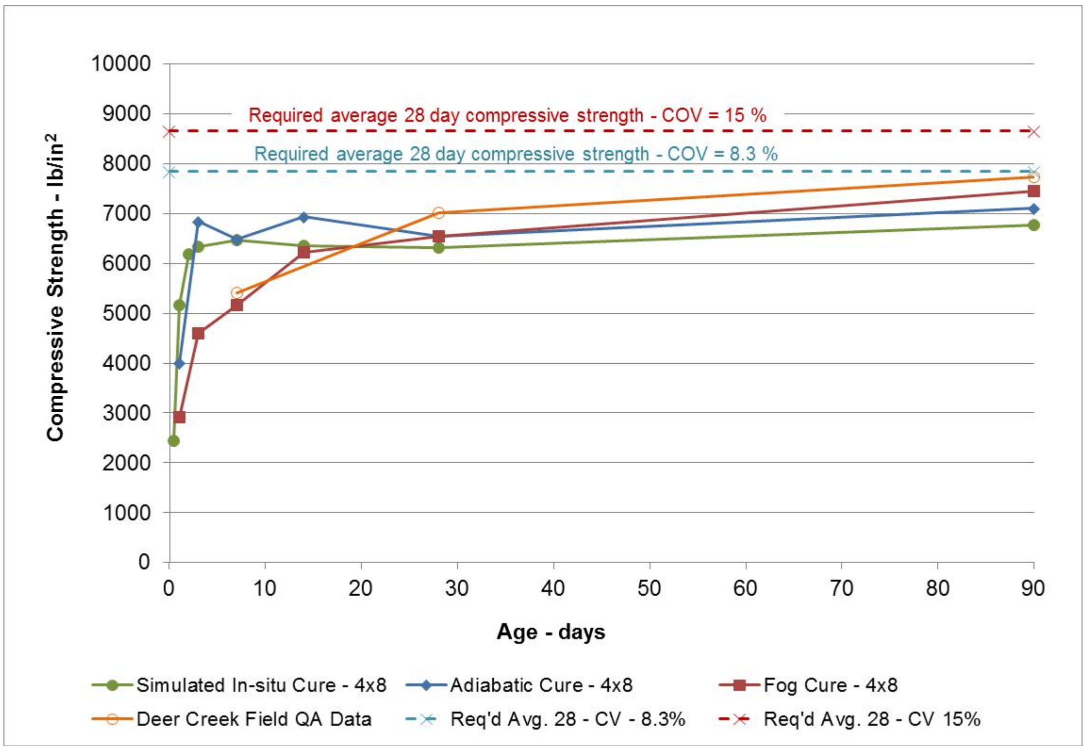

Figure 29 – Graph of average compressive strengths for the Deer Creek Dam Modification mixture under varying curing conditions.

Table 10 – Average modulus of elasticity for the Deer Creek Dam Modification mixture under varying curing conditions, 10^6 lb/in^2.

Age	Simulated In-situ Cure 4-by 8-in Specimen	Adiabatic Cure 4-by 8-in Specimen	Fog Cure 4-by 8-in Specimen	Fog Cure 6-by 12-in Specimen
24 hrs	3.15	3.15	2.94	2.61
3 day	4.36	4.36	3.39	3.43
7 day	4.27	4.27	3.29	2.99
14 day	4.46	4.46	4.00	-
28 day	4.56	4.56	4.32	4.23
90 day	4.74	4.74	4.37	4.64

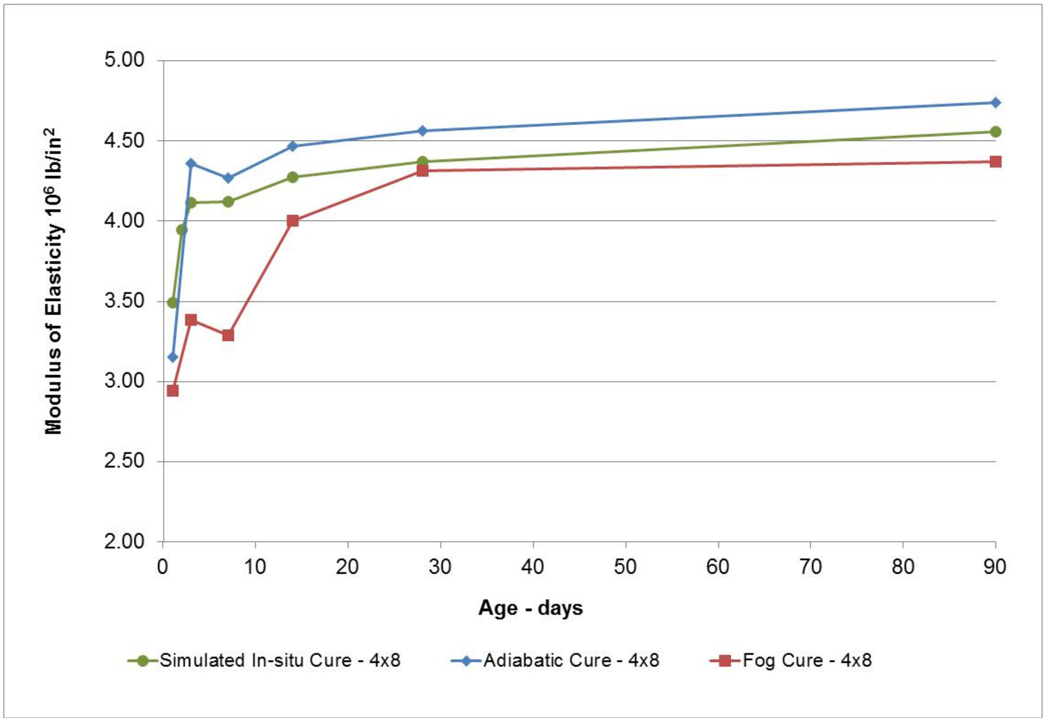

Figure 30 – Graph of modulus of elasticity for the Deer Creek Dam Modification mixture under varying curing conditions.

In practice, if the 28-day fog-cured compressive strength of RSMC does not meet the required design strength, it is common to assume strength gain will continue with age. This may hold true for the fog-cured specimens where the ultimate strength potential of a given concrete mix can be achieved. However, uncertainty of the strength curve of the actual placement arises based on the adiabatic and in-situ cured test specimens. The adiabatic and in-situ cured test specimens have the potential to dry out or to expend available water for complete hydration. The degree at which the same phenomenon occurs in the actual placement is still unclear.

It is routine to specify a 56-day (or longer) fog-cured compressive strength for mass concrete. Caution in using this practice should be used for RSMC as the high temperatures appear to limit the strength gain potential of a given mixture. The perceived benefit of waiting longer to achieve the design strength could be inaccurate.

Figure 31 illustrates that the Deer Creek simulated in-situ cured specimens have an almost flat strength gain curve on a logarithmic scale, where the fog-cured specimens from the same concrete show a continued strength increase past the peak of the simulated in-situ cured specimens. For reference, fog-cured strength specimens from Hoover Dam are also shown to demonstrate 1) the lower initial strength and 2) the continued increase in strength over time. At 60 years age, strengths for Hoover Dam concrete exceed the 90 day strengths for simulated in-situ cured Deer Creek specimens which had almost 600 lb/yd^3 more total cementitious materials.

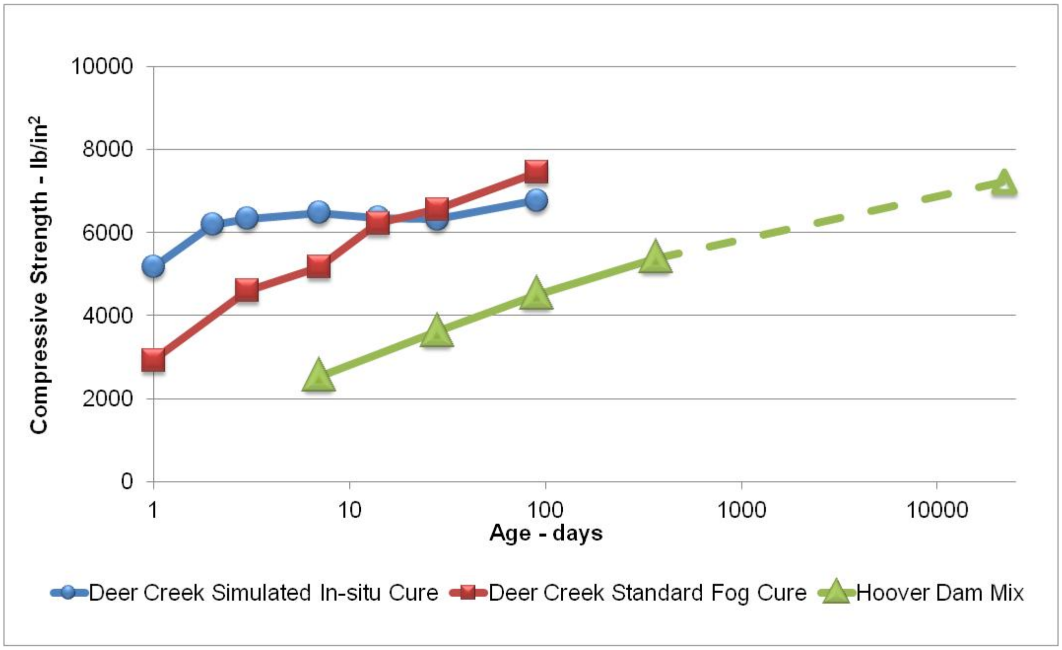

Figure 31 - Strength gain over time for the Deer Creek lab mix compared to the Hoover Dam, logarithmic scale.

Caution is urged for adjusting RSMC mixtures if the compressive strength is lower than expected. Traditional concrete mixture adjustments for low strength are to reduce the water to cementitious materials (w/cm) ratio. In this case, the lower w/cm ratio would result in an increase in cementitious content which would generate more heat and be somewhat counterproductive for increasing the strength. It would be just as, or more, beneficial to reduce the peak temperature of the concrete than to increase the cementitious content for long-term strength gain. Also, if a lower initial design strength could be utilized a long-term benefit may be attained.

Specimen Size Requirement

Because of the large aggregate used in traditional mass concrete, large diameter cylinders (up to 12-inches in diameter) are often needed, with additional quality assurance 6- by 12-inch cylinders that are wet-sieved to remove the plus 1½-inch material. This practice is usually not needed for RSMC because of the smaller aggregate typically used for these mixtures. ASTM C31 requires the specimen diameter to be at least three times the NMSA. In this study, the strength of 4- by 8-inch test specimens was comparable to 6- by 12-inch specimens. The practice of going to smaller diameter 4- by 8-inch cylinders would be acceptable for these higher strength mixtures with small NMSA. Smaller diameter cylinders are easier to make, transport and test. Such correlations should be developed before instituting this practice for a large project.

Additional Construction Considerations

Another problem with RSMC is reinforcing steel congestion. Structures placed in multiple lifts must have the surface of each lift scarified and cleaned to assure bond to the next layer. Lift surface cleaning is difficult due to both accessibility problems and the high strength of the concrete itself. Traditional lift surface cleaning is normally performed a few days after placement when the concrete is typically about 3000 lb/in^2 but high strength RSMC may be twice as strong by this time and equipment capable of cutting the surface must be used. High pressure water blasting equipment must use higher nozzle pressures. Otherwise lift surface cleanup may need to moved up in the schedule to immediately after forms are stripped. Figures 33 and 34 show the construction joint cleanup for both the new RSMC and the existing counterfort wall.

Figure 32 - Construction joint lift surface cleanup for RSMC. Deer Creek Dam Spillway Modification.

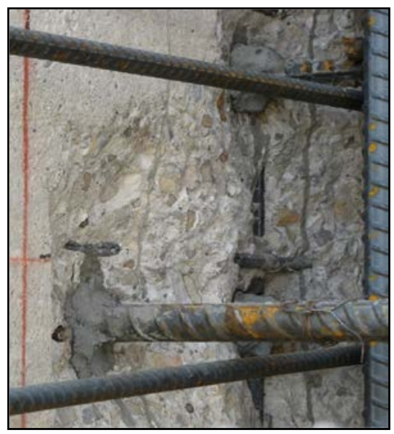

Figure 33 - Cleanup of left counterfort wall surface prior to placement of the RSMC. Deer Creek Dam Spillway Modification.

One additional comment raised during the Fall 2011 ACI conference was that the temperature sensors measuring the near surface concrete temperature should be closer than the 6-inches used in this research. There is a much greater risk of cracking at the surface and industry practice is to place the temperature monitoring devices around 3 inches or less from the surface. This practice has been incorporated into more recent projects and the recommended specifications for RSMC.

Measures to Control Temperature

Measures used for these, and other, Reclamation projects to control concrete temperatures in order to minimize thermal cracking are tabulated below. A combination of measures appropriate for the project may be needed and will not necessarily be sufficient to control thermal cracking.

Table 11 – Potential measures to control temperature and reduce thermal cracking.

Measure	Rationale
Reduce design strength	Lowering the total cementitious content will reduce the overall heat potential of a placement.
Reduce placement temperature	Lowering the initial placement temperature will lower the final peak internal temperature.
Increase pozzolan content	For a given total cementitious content, pozzolans such as Class F fly ash and GGBFS can contribute less heat than a mixture containing 100% cement.
Use largest practical NMSA	Larger aggregates have less surface area per unit volume and require less cement to coat the particles and achieve the same strength and workability.
Select a coarse aggregate with low thermal expansion	Aggregates such as granite and limestone have a lower coefficient of thermal expansion which can result in lower internal stresses that can cause cracking.
Use thermal insulation blankets	Insulating blankets can be used to minimize the temperature differential between the center and exterior of the placement resulting in less potential for thermal shock or cracking.
Use Type V or Low Heat cement	Lower heat cements generally can reduce the peak internal temperature of a mix for a given cementitious content
Reduce variability (COV = X %)	Overdesign for field strength requirements in accordance with USBR 4211 is a function of the coefficient of variability (COV). Reducing field variability can reduce the COV of the mix which results in a lower target strength. Lower target strengths require less cementitious materials.
Time of year	Whenever possible schedule construction activities to minimize environmental effects. Concrete placements during summer months may lessen the risk of exceeding thermal gradients whereas winter placements may reduce the peak temperatures.
Post-cooling	When other methods to control heat and reduce internal stress are insufficient, post-cooling tubes that circulate cold water can remove excess heat from the concrete.
Smaller placements	Reducing the placement size will increase the surface area to volume ratio and may aid in cooling.
Waiting longer between placements	Adjacent placements can contribute to the peak temperature of subsequent lifts so allowing the placement to cool sufficiently before beginning the next placement can reduce this effect.

Specification Recommendations

Reclamation designers have initiated changes to the way RSMC is specified based on information learned in this research project and during the construction of these and other recent Dam Safety modifications. Proposed additions to specification language for RSMC are presented in Appendix G. This language has recently been adopted into Reclamation project specifications and Guide Specifications. Specific limits for maximum temperature differential could vary based on the recommendations of a thermal analysis, if one is performed.

The additional specification items included are:

- Defining RSMC
- Requiring a RMSC Temperature Control Plan submittal
- Allowing higher pozzolan contents
- Allowing the use of Ground-Granulated Blast Furnace Slag (GGBFS), blended cements, and ternary blends[3]
- Listing of the types of acceptable temperature monitoring devices
- Lower concrete placing temperature
- Placing and curing limitations

Conclusions

- Delayed ettringite formation was not a concern for the RSMC in the two field studies studied. Each project should be evaluated on an individual basis for its risk of DEF.

- The placements studied exceeded the maximum specified 35 ^{o}F temperature differential when comparing the sensors at the center of the placement to the sensors 6 inches from the surface.

- The RSMC surfaces at Stony Gorge Dam and Deer Creek Dam spillway should be inspected at some point in the future to determine whether any cracks have developed.

- Design considerations should include strength, steel reinforcement congestion and heat issues. Specifications that explicitly address RSMC should be incorporated into the project documents.

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[3] Ternary blends are a combination of three or more cementitious materials, sometimes necessary to obtain special properties.

- Monitoring at the near surface should be about 3 inches or less from the surface for future studies.

- When using RSMC for critical structures, mixture proportioning investigations and an adiabatic temperature rise test should be conducted during design using materials from the project area. Results from the adiabatic temperature rise should be used in a thermal analysis to assure stresses developed stay within the linear range of the concrete.

- When using RSMC for critical structures, a complete thermal analysis is recommended to determine the measureable effects of the temperature gradients. This thermal analysis should be used to set reasonable limits for temperature differentials, placement sizes, and concrete placement sequencing.

- The maximum concrete temperature rises recorded for large SRMC placements were equivalent to the adiabatic temperature rise. Without post cooling, large concrete placements should be expected to reach the full adiabatic heat rise potential.

- In practice, if the 28-day fog-cured compressive strength of RSMC does not meet the required design, continued strength gain with age of the actual concrete placement should not always be anticipated.

- A 56-day (or later age) design strength should not be used for RSMC as high temperatures appear to limit the in-situ strength gain potential of a given mixture, and the perceived benefit of waiting longer to attain the design strength could be inaccurate.

- The practice of using 4- by 8-inch cylinders would be acceptable for RSMC with smaller aggregate. Smaller diameter cylinders are easier to make, transport and test. Strength correlations should be developed before instituting this practice for a large project.

- Future research is recommended to study other methods to predict adiabatic temperature rise and to assist with updating Figure 4.1of ACI 207.2R. Testing should include mixtures with higher total cementitious contents and pozzolans to better reflect the current state of practice for RMSC.

- Future research should further evaluate maximum temperature differentials incrementally for very large (or long) placements.

References

[1] ACI Manual of Concrete Practice, ACI 207.1 Guide to Mass Concrete, American Concrete Institute, Farmington Hills, Michigan, 2009.
[2] Mehta,P. and Monteiro, P., Concrete – Microstructure, Properties, and Materials, The McGraw Hill Companies, Inc., 2006.
[3] Day, Robert L., Research and Development Bulletin RD108T, The Effect of Secondary Ettringite Formation on the Durability of Concrete: A Literature Analysis, Portland Cement Association, Skokie, Illinois, 1992.
[4] Divet, Loïc, Delayed ettringite formation in massive concrete structures: Summary of studies conducted on deteriorated bridges, Bulletin Des Laboratories Des Ponts et Chaussées, May-June-July-August 2003, Ref. 4473, pg. 91-111.
[5] Concrete Manual, Part 1 eighth edition, US Department of Interior, Bureau of Reclamation, reprint 1988.
[6] Intellirock II™, Engius, LLC. 712 Eastgate, Stillwater, OK 74074, www.engius.com.
[7] US Department of Interior, Bureau of Reclamation, Stony Gorge Dam Modification. Orland Project, California, Solicitation 06SP202026, Specification 20-C0642.
[8] US Department of Interior, Bureau of Reclamation, Deer Creek Dam Modification - Phase III, Utah, Solicitation 07-CC-40-8203.
[9] US Department of Interior, Bureau of Reclamation, New Wadell Dam, Stage II, Regulatory Storage Division, Central Arizona Project, Arizona, Solicitation 0-CC-32-00930, Specification DC-7797.
[10] US Department of Interior, Bureau of Reclamation, Durango Pumping Plant Stage 2, Animas-La Plata Project, Colorado, Solicitation 04-NA-40-8013, Specification 40-8013.
[11] ACI Manual of Concrete Practice, ACI 207.2R Thermal and Volume Change Effects on Cracking of Mass Concrete, American Concrete Institute, Farmington Hills, Michigan, 2009.
[12] Gajda, J., Mass Concrete for Buildings and Bridges, Portland Cement Association, Skokie, Illinois, 2007.
[13] Concrete Manual, Part 2, Ninth Edition. A Water Resources Technical Publication. U.S. Department of Interior, Bureau of Reclamation, Denver, CO, 1992.
[14] ACI 318 Building Code for Structural Concrete, American Concrete Institute, Farmington Hills, Michigan, 2012.
[15] ASTM International, West Conshohocken, PA, 2010.

APPENDIX A
Mixture Proportions of Concrete Placements Studied

Reinforced Structural Mass Concrete

Project	Orland Project, CA	Provo River Project, UT	Central Arizona Project, AZ			Animas-La Plata Project, CO	
Feature	Stony Gorge Dam [i]	Deer Creek Dam Spillway [ii]	New Waddell Dam [iii]			Durango Pumping Plant [iv]	
Placement Description	Diaphragm Wall	Buttress Walls (Placement No. 6)	Bridge Pier	Intake Tower Wall	Intake Tower Footing	Summer Mix	Winter Mix
Date Range	April 2008	April 2008	October 1990	December 1990	October 1990	June 2005	November 2004
NMSA	1½-inch	¾-inch	¾-inch	1½-inch	3-inch	2-inch	2-inch
Cement Type	Type II	Type II	Type II	Type II	Type II	Type V	Type II
Cement - lb/yd^3	529	735	647	583	459	523	509
Pozzolan (Class F Fly Ash) - lb/yd^3	176	183	162	146	114	175	170
Silica fume - lb/yd^3	-	46	-	-	-	-	-
Sand - lb/yd3	1405	710	1260	1182	1000	1150	1105
Coarse aggregate - lb/yd^3	1535	1830	1520	1740	2240	1830	1820
Water - lb/yd^3	280	310	265	255	205	256	264
Admixtures	AEA / WRA	AEA / WRA / HRWRA	AEA / WRA	AEA / WRA	AEA / WRA	AEA / WRA	AEA / WRA
Percent air – gravimetric	5	3.5	5.2	4.2	3.7	3.3	3.6
Percent air - pressure meter	4.4	5.3	5.4	4.4	4.2	3.9	4.1
Slump - inches	5	6	3	4-1/4	3-3/4	2-1/2	2-1/2
Placement Temperature - ^oF	52	58	76	61	74	67	60
Average ambient temperature - ^oF	63	50	77	64	67	70	33
Maximum recorded temperature - ^oF	156	178	176	152	170	144	109
Maximum temperature rise - ^oF	104	120	100	91	96	77	49
Design Strength - lb/in^2	4000 @ 28 days	7000 @ 28 days	4000 @ 28 days	4000 @ 28 days	4000 @ 28 days	4000 @ 28 days	4000 @ 28 days
Required average strength for COV = 15% (90 % exceeding design strength) - lb/in^2	4950	8660	4950	4950	4950	4950	4950
Testing Results (USBR Quality Assurance)							
Average 7 day Compressive Strength - lb/in^2	3370	6180	4170	3680	3630	3790	3310
Average 28 day Compressive Strength - lb/in^2	4620	7920	4670	4900	4380	5450	4520
Average 90 day Compressive Strength - lb/in^2	5740	7734	5390	N/A	5160	N/A	5430
USBR Required Average Strength based on COV of QA Test Specimens - lb/in^2	4590 (COV = 10.1 %)	7840 (COV = 8.3 %)	4950 (COV = 15%)	4340 (COV = 5.7%)	4560 (COV = 8.9 %)	4530 (COV = 9.1 %)	4530 (COV 9.2)

[i] U.S. Department of the Interior, Bureau of Reclamation, L-29 Report of Concrete Construction, Solicitation 06SP202026, Stony Gorge Dam Modification, Orlando Project, California, 2008.

[ii] U.S. Department of the Interior, Bureau of Reclamation, L-29 Report of Concrete Construction, Solicitation 07-CC-40-8203, Deer Creek Dam Spillway Modification - Phase III, Provo River Project, Utah, February 8 to May 19, 2008.

[iii] U.S. Department of the Interior, Bureau of Reclamation, L-29 Report of Concrete Construction, Solicitation 0-CC-32-00930 , New Wadell Dam Stage II, Central Arizona Project, Arizona September to December 1990.

[iv] U.S. Department of the Interior, Bureau of Reclamation, L-29 Reports of Concrete Construction, Solicitation 04-NA-40-8013, Durango Pumping Plant Stage #2, Animas-La Plata Project, Colorado, Janunary 2004 - July 2005.

APPENDIX B
Temperate Sensor Locations

STONY GORGE

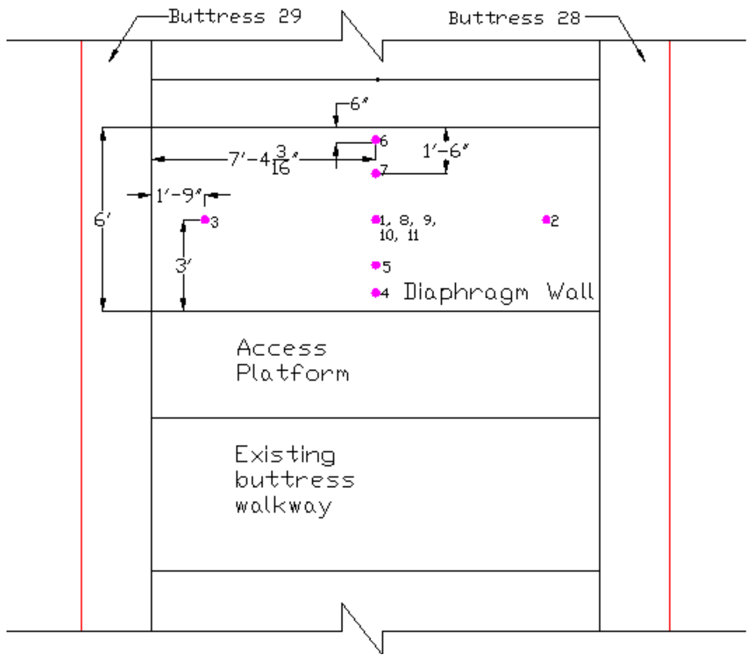


Figure B-1. Plan view of temperature monitors installed at Stony Gorge Dam Modification Project, Orland Project, California.

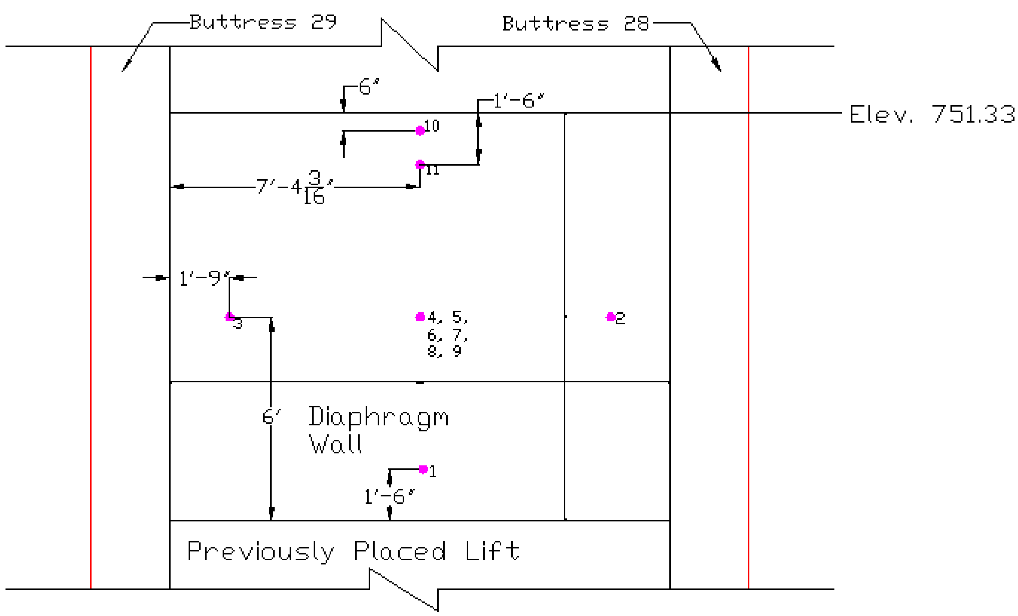


Figure B-2. Elevation view of temperature monitors installed at Stony Gorge Dam Modification Project, Orland Project, California.

DEER CREEK

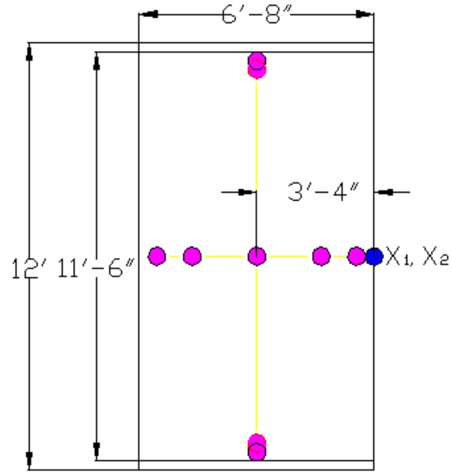


Figure B-3. Plan view drawing depicting placement location for temperature monitors; Deer Creek Dam Spillway Modification, Utah.

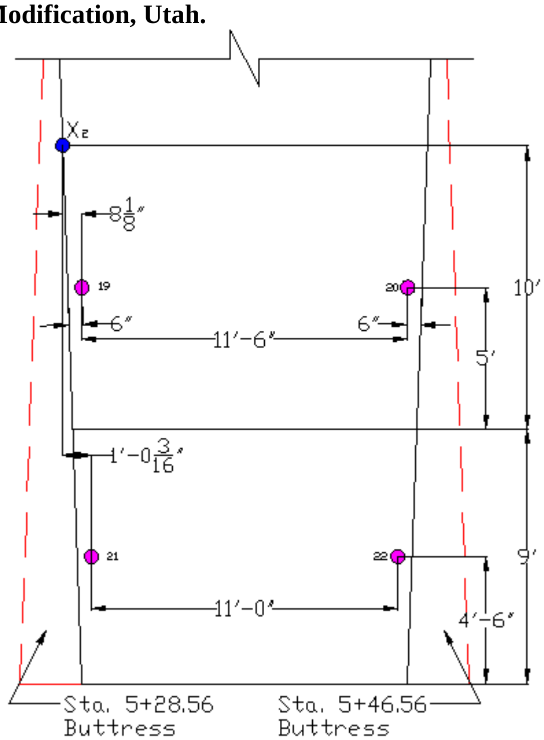


Figure B-4. Elevation view drawing depicting placement location for temperature monitors; Deer Creek Dam Spillway Modification, Utah.

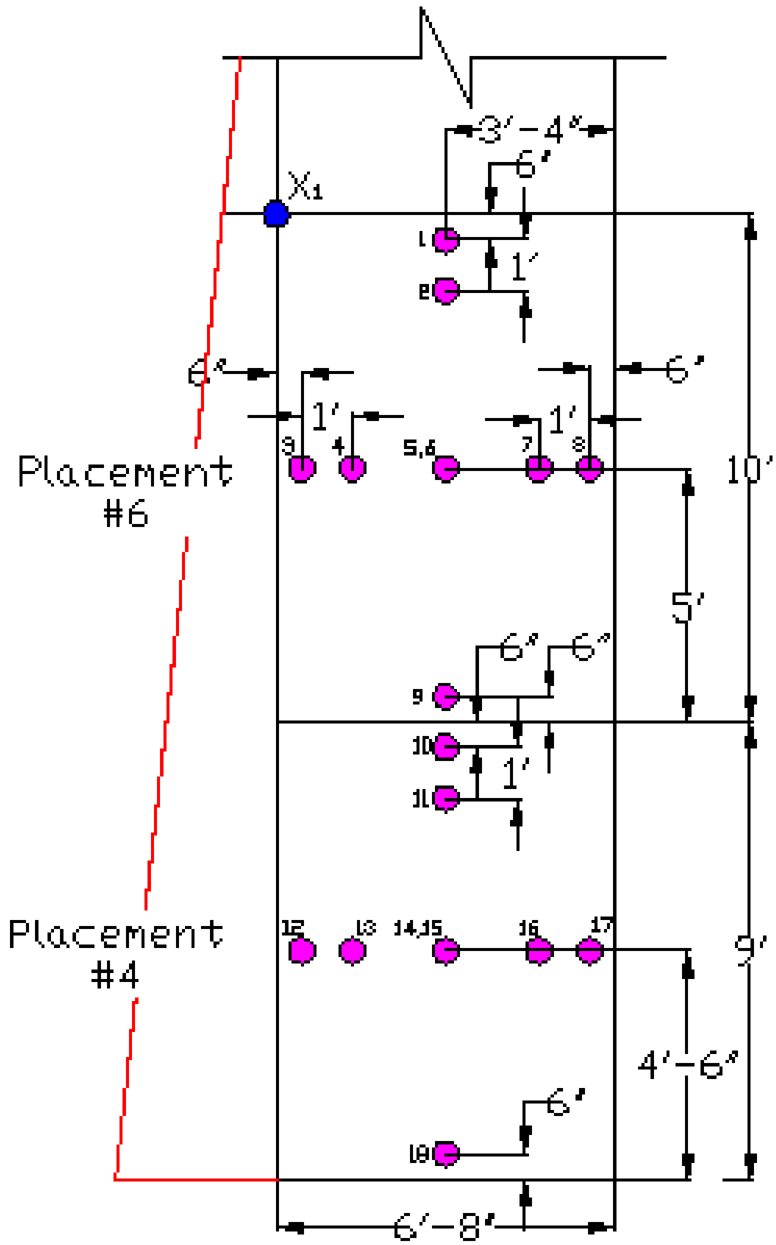
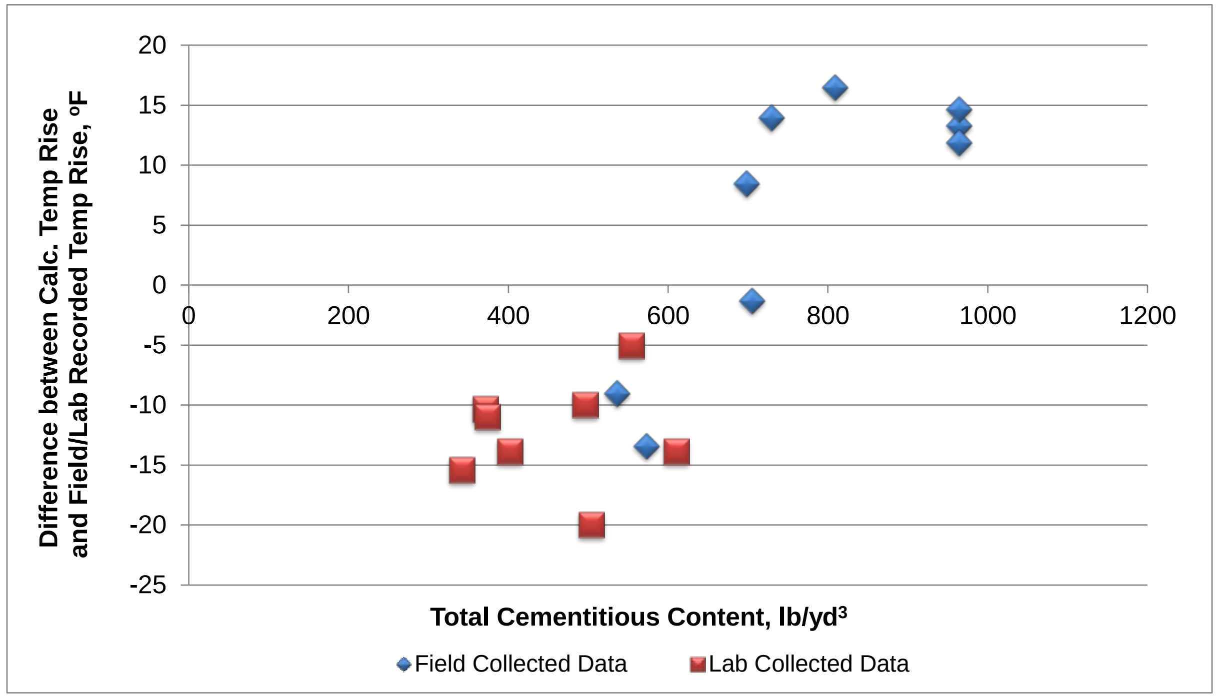


Figure B-5. Sectional view drawing depicting placement location for temperature monitors; Deer Creek Dam Spillway Modification, Utah.

APPENDIX C
Comparison of Calculated, Field, and Laboratory Temperature Rise

		Reinforced Structural Mass Concrete									Unreinforced Tradational Mass Concrete							
MIXTURE ID		Stony Gorge	Deer Creek	Deer Creek	Deer Creek	New Waddell Dam	New Waddell Dam	New Waddell Dam	Durango Pumping Plant	Minidoka Dam	Canton Dam Lab Study	Canton Dam Lab Study	Folsom JFP Lab Study	Folsom JFP Lab Study	Folsom JFP Lab Study	Folsom JFP Lab Study	Folsom JFP Lab Study	Folsom JFP Lab Study
Description		diaphragm wall	No. 4 from L29	No. 6 from L29	No. 8 from L29	Bridge Pier	Intake Tower Wall	Intake Tower Footing	Type V mix	20% HR	CDF-12	CDF-H	FMC-A-5	FMP-1-6	FMC-B-16	FMC-C-17	FMP-3-17	JFP-3-35
Materials, lb/yd^3	Equivalent Cement Factor	Type II + Class F Ash	Type II/V + Class F Ash + SF	Type II/V + Class F Ash + SF	Type II/V + Class F Ash + SF	Type II + Class F Ash	Type II + Class F Ash	Type II + Class F Ash	Type V + Class F Ash	Type I/II + Class F Ash	Type I/II + Class F Ash	Type I/II + GGBFS	Low Heat + Class F Ash	Low Heat + Class F Ash	II/V + Class F Ash	Type II/V + GGBFS	Type II/V + GGBFS	Type II/V + GGBFS
Cement	1	529	735	735	735	647	583	459	523	376	240	201	347	388	353	111	112	305
Fly Ash (Class F)	0.5	176	183	183	183	162	146	114	175	160	102		149	166	151			
Silica Fume	1.25		46	46	46													
GGBFS 50% replacement	0.9											201						305
GGBFS 75% replacement	0.8															260	262	
Fine Agg	-	1427	813	852	723	1260	1182	1000	1150	1158	1033	1021	1073	988	1046	1082	1022	863
Coarse Agg	-	1566	1731	1681	1820	1520	1740	2240	1830	1795	2586	2502	2287	2380	2228	2415	2583	2350
Water	-	264	334	324	305	265	255	205	256	217	145	181	248	240	223	223	178	204
Total Cementitous Used	-	705	964	964	964	809	729	573	698	536	342	402	496	554	504	371	374	610
% Pozzolan Used	-	25%	24%	24%	24%	20%	20%	20%	25%	30%	30%	50%	30%	30%	30%	70%	70%	50%
Cement Multiplication Factor from ACI 207.2R Fig 4.1 (°F rise for a cement type/376 lb/yd^3)		0.16	0.14	0.14	0.14	0.16	0.16	0.16	0.14	0.16	0.16	0.16	0.14	0.14	0.14	0.14	0.14	0.14
Calculated Equivalent Cement Content		617	884	884	884	728	656	516	611	456	291	382	422	471	429	319	322	580
Calculated Adiabatic Temperature Rise[1]	(°F)	**103**	**126**	**124**	**124**	**116**	**105**	**83**	**85**	**73**	**47**	**61**	**59**	**66**	**60**	**45**	**45**	**81**
Temperature Rise Recorded in Field	(°F)	**104**	**113**	**112**	**109**	**100**	**91**	**96**	**77**	**82**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**
USBR 4911 Adiabatic Temperature Rise	(°F)	**N/A**	**N/A**	**110**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**62**	**75**	**69**	**71**	**80**	**55**	**56**	**95**
Difference from Calculated Adiabatic Temperature Rise	(°F)	**-1**	**13**	**12**	**15**	**16**	**14**	**-13**	**8**	**-9**	**-15**	**-14**	**-10**	**-5**	**-20**	**-10**	**-11**	**-14**

[1] *Reference:*
Gajda, John; Mass Concrete for Building and Bridges, Portland Cement Association, Skokie, Illinois, 2007.
ACI Manual of Concrete Practice, ACI 207.2R-07 Report on Thermal andVolume Change Effects on Cracking of Mass Concrete, Figure 4.1, American Concrete Institute, Farmington Hills, Michigan, 2012.



Comparison of simplified calculation for temperature rise to field and laboratory temperature rise data.

APPENDIX D
Photos of Laboratory Study

Appendix D Figure 1 - Concrete in the laboratory mixer made with materials from the Deer Creek Dam Spillway Modification project.

Appendix D Figure 2- Concrete in the laboratory after mixing.

Appendix D Figure 3 - Standard USBR 4911 temperature rise container (surrounded with insulation after concrete is added).

Appendix D Figure 4 - Making 4-by 8-inch concrete specimens for compressive strength and elastic properties testing.

Appendix D Figure 5 - USBR 4911 Adiabatic Temperature Rise test in progress with companion adiabatic strength specimens (inside environmental chamber). The environmental chamber temperature is matched to the internal temperature inside the insulated temperature rise container.

Appendix D Figure 6 - In-situ cure strength specimens cured inside a small environmental chamber programmed with the actual Deer Creek Dam Spillway temperature curve.

APPENDIX E
Test data from Laboratory Study

THERM

Deer Creek Mixture

Mix ID	**DC-T**	AC	Adiabatic Cure	Specimens placed in Adiabatic Chamber with Temp Rise after casti
Cast Date	**9/1/2009**			

FR Fog Cure — Standard

FC Field Cure — Follows temp rise of center of placement as recording in field (BLUE

AC Adiabatic Cure — Specimens placed in Adiabatic Chamber with Temp Rise after casti

Specimen ID	Test Age	Break Date	Diameter inches	Length inches	Cure Type	Ultimate Load, lb	Strength lb/in^2	Modulus 10^6 lb/in^2
DC-T-AC-1	24 hrs	9/2/2009	4	8	Adiabatic Cure	50730	4040	3.13
DC-T-AC-2	24 hrs	9/2/2009	4	8	Adiabatic Cure	50031	3980	3.00
DC-T-AC-3	24 hrs	9/2/2009	4	8	Adiabatic Cure	50532	4020	3.32
Average						50431	4010	3.15
DC-T-AC-4	3	9/4/2009	4	8	Adiabatic Cure	83927	6680	4.15
DC-T-AC-5	3	9/4/2009	4	8	Adiabatic Cure	87197	6940	4.06
DC-T-AC-6	3	9/4/2009	4	8	Adiabatic Cure	86526	6890	4.86
Average						85883	6840	4.36
DC-T-AC-7	7	9/8/2009	4	8	Adiabatic Cure	85067	6770	4.10
DC-T-AC-8	7	9/8/2009	4	8	Adiabatic Cure	79929	6360	4.40
DC-T-AC-9	7	9/8/2009	4	8	Adiabatic Cure	79213	6300	4.31
Average						81403	6480	4.27
DC-T-AC-10	14	9/15/2009	4	8	Adiabatic Cure	83781	6670	4.37
DC-T-AC-11	14	9/15/2009	4	8	Adiabatic Cure	90958	7240	4.61
DC-T-AC-12	14	9/15/2009	4	8	Adiabatic Cure	87029	6930	4.41
Average						87256	6950	4.46
DC-T-AC-13	28	9/29/2009	4	8	Adiabatic Cure	80470	6400	4.68
DC-T-AC-14	28	9/29/2009	4	8	Adiabatic Cure	82583	6570	4.53
DC-T-AC-15	28	9/29/2009	4	8	Adiabatic Cure	83733	6660	4.49
Average						82262	6540	4.56
DC-T-AC-16	90	11/30/2009	4	8	Adiabatic Cure	88070	7010	4.71
DC-T-AC-17	90	11/30/2009	4	8	Adiabatic Cure	87159	6940	4.83
DC-T-AC-18	90	11/30/2009	4	8	Adiabatic Cure	92338	7350	4.67
Average						89189	7100	4.74

THERM

Deer Creek Mixture

Mix ID **DC-T**

Cast Date **9/1/2009**

FR Fog Cure Standard

FC Field Cure Follows temp rise of center of placement as recording in field (BLUE

AC Adiabatic Cure Specimens placed in Adiabatic Chamber with Temp Rise after casti

Specimen ID	Test Age	Break Date	Diameter inches	Length inches	Cure Type	Ultimate Load, lb	Strength lb/in^2	Modulus 10^6 lb/in^2
DC-T-FC-19	12 hrs	9/1/2009	4	8	Field Cure	30713	2440	-
DC-T-FC-20	12 hrs	9/1/2009	4	8	Field Cure	30459	2420	-
DC-T-FC-21	12 hrs	9/1/2009	4	8	Field Cure	30696	2440	-
Average						30623	2430	-
DC-T-FC-22	24 hrs	9/2/2009	4	8	Field Cure	65370	5200	3.64
DC-T-FC-23	24 hrs	9/2/2009	4	8	Field Cure	63761	5070	3.48
DC-T-FC-24	24 hrs	9/2/2009	4	8	Field Cure	65903	5240	3.35
Average						65011	5170	3.49
DC-T-FC-25	2	9/3/2009	4	8	Field Cure	78267	6230	3.89
DC-T-FC-26	2	9/3/2009	4	8	Field Cure	78307	6230	3.96
DC-T-FC-27	2	9/3/2009	4	8	Field Cure	76603	6100	3.98
Average						77726	6190	3.95
DC-T-FC-28	3	9/4/2009	4	8	Field Cure	77584	6170	4.12
DC-T-FC-29	3	9/4/2009	4	8	Field Cure	80231	6380	4.05
DC-T-FC-30	3	9/4/2009	4	8	Field Cure	80758	6430	4.17
Average						79524	6330	4.11
DC-T-FC-31	7	9/8/2009	4	8	Field Cure	82218	6540	4.08
DC-T-FC-32	7	9/8/2009	4	8	Field Cure	79029	6290	4.15
DC-T-FC-33	7	9/8/2009	4	8	Field Cure	82796	6590	4.13
Average						81348	6470	4.12
DC-T-FC-34	14	9/15/2009	4	8	Field Cure	81014	6450	4.25
DC-T-FC-35	14	9/15/2009	4	8	Field Cure	80253	6390	4.30
DC-T-FC-36	14	9/15/2009	4	8	Field Cure	77839	6190	2.77
Average						79702	6340	3.77
DC-T-FC-37	28	9/29/2009	4	8	Field Cure	80798	6430	4.29
DC-T-FC-38	28	9/29/2009	4	8	Field Cure	81779	6510	4.42
DC-T-FC-39	28	9/29/2009	4	8	Field Cure	75435	6000	4.40
Average						79337	6310	4.37
DC-T-FC-40	90	11/30/2009	4	8	Field Cure	87185	6940	4.54
DC-T-FC-41	90	11/30/2009	4	8	Field Cure	82950	6600	4.54
DC-T-FC-42	90	11/30/2009	4	8	Field Cure	84961	6760	4.6
Average						85032	6770	4.56

THERM

Deer Creek Mixture

Mix ID **DC-T**

Cast Date **9/1/2009**

FR Fog Cure — Standard

FC Field Cure — Follows temp rise of center of placement as recording in field (BLUE

AC Adiabatic Cure — Specimens placed in Adiabatic Chamber with Temp Rise after casti

Specimen ID	Test Age	Break Date	Diameter inches	Length inches	Cure Type	Ultimate Load, lb	Strength lb/in^2	Modulus 10^6 lb/in^2
DC-T-FR-43	24 hrs	9/2/2009	6	12	Fog Cure	78624	2780	2.57
DC-T-FR-44	24 hrs	9/2/2009	6	12	Fog Cure	82335	2910	2.66
Average						80480	2850	2.61
DC-T-FR-45	3	9/4/2009	6	12	Fog Cure	116602	4120	2.92
DC-T-FR-46	3	9/4/2009	6	12	Fog Cure	118016	4170	3.95
Average						117309	4150	3.43
DC-T-FR-47	7	9/8/2009	6	12	Fog Cure	136430	4830	3.11
DC-T-FR-48	7	9/8/2009	6	12	Fog Cure	137310	4860	3.00
DC-T-FR-49	7	9/8/2009	6	12	Fog Cure	137374	4860	2.85
Average						137038	4850	2.99
DC-T-FR-50	28	9/29/2009	6	12	Fog Cure	198894	7030	4.24
DC-T-FR-51	28	9/29/2009	6	12	Fog Cure	194005	6860	4.19
DC-T-FR-52	28	9/29/2009	6	12	Fog Cure	185099	6550	4.27
Average						192666	6810	4.23
DC-T-FR-53	90	11/30/2009	6	12	Fog Cure	210283	7440	4.86
DC-T-FR-54	90	11/30/2009	6	12	Fog Cure	215326	7620	4.51
DC-T-FR-55	90	11/30/2009	6	12	Fog Cure	217952	7710	4.56
Average						214520	7590	4.64
DC-T-FR-56	24 hrs	9/2/2009	4	8	Fog Cure	36400	2900	2.66
DC-T-FR-57	24 hrs	9/2/2009	4	8	Fog Cure	36812	2930	3.33
DC-T-FR-58	24 hrs	9/2/2009	4	8	Fog Cure	36822	2930	2.83
Average						36678	2920	2.94
DC-T-FR-59	3	9/4/2009	4	8	Fog Cure	61642	4910	3.22
DC-T-FR-60	3	9/4/2009	4	8	Fog Cure	57992	4610	3.55
DC-T-FR-61	3	9/4/2009	4	8	Fog Cure	53139	4230	-
Average						57591	4580	3.39
DC-T-FR-62	7	9/8/2009	4	8	Fog Cure	69769	5550	3.35
DC-T-FR-63	7	9/8/2009	4	8	Fog Cure	64202	5110	3.08
DC-T-FR-64	7	9/8/2009	4	8	Fog Cure	60940	4850	3.44
Average						64970	5170	3.29
DC-T-FR-65	14	9/15/2009	4	8	Fog Cure	77544	6170	3.84
DC-T-FR-66	14	9/15/2009	4	8	Fog Cure	78731	6270	4.16
Average						78138	6220	4.00
DC-T-FR-68	28	9/29/2009	4	8	Fog Cure	78879	6280	4.10
DC-T-FR-69	28	9/29/2009	4	8	Fog Cure	83480	6640	4.16
DC-T-FR-70	28	9/29/2009	4	8	Fog Cure	84715	6740	4.69
Average						82358	6550	4.32
DC-T-FR-71	90	11/30/2009	4	8	Fog Cure	101106	8050	4.37
DC-T-FR-72	90	11/30/2009	4	8	Fog Cure	86075	6850	4.37
Average						93591	7450	4.37

APPENDIX F
Cursory Petrographic Examination of Tested Concrete

CURSORY PETROGRAPHIC EXAMINATION OF CONCRETE

Subject: Therm Cylinders **Fog, Blue Box, and HR Chamber samples DC-T-R68, DC-T-FC67, and DC-T-AD13**

As-received examination

Mechanically-fractured cylinders failed in compression

Aggregate

Gravel: Natural heterogeneous: gneiss, limestone, sandstone, and chert; about 30 percent of total mass; slightly rough surface textures; angular to subrounded in shape; size ranges from about 4 mm (1/4 in) to 30 mm (1 in); particles exhibit various textures from fine to medium grained; structureless; particles exhibit a chiefly moderate bond ranging from poor to good; slightly absorptive; breaks around and through aggregates
Sand: Natural heterogenous

Gravel and sand: Apparently petrographically of satisfactory physical quality and potentially deleteriously reactive with the Portland cement used

Paste

Medium dark gray; subtranslucent glimmering luster; slightly absorptive; moderately hard; moderately well consolidated; not carbonated; close contact with aggregates; no fractures observed; no evidence of SCM, contamination, or bleeding

Air voids

Evenly graded; numerous; spherical in shape; greater than 3 percent; empty; interior luster and color same as paste

Conclusions

The examined concrete appears petrographically of satisfactory quality. The aggregate appears petrographically of satisfactory quality and shows no evidence of deleterious reactivity with the cement used. The paste and aggregates are generally moderately well distributed, graded, and packed and the paste aggregate bond is poor to good.
No evidence of deterioration could be detected in the examined concrete. The concrete is well hydrated and not carbonated

APPENDIX G
Recommended Specification Language for RSMC

Report DSO-12-02*: Thermal Properties of Reinforced Structural Mass Concrete, Dam Safety Technology Development Program*

SECTION 03 30 00

CAST-IN-PLACE CONCRETE

This is additional language to be added to a concrete specification if RSMC is used. This language is intended to be in addition to the standard concrete language and not a replacement. Advisors from the Materials Engineering and Research Laboratory are available to assist with the incorporation of this language.

PART 1 GENERAL

1.01 REFERENCE STANDARDS

1.	ASTM C 150-11	Portland Cement
2.	ASTM C 595-11	Standard Specification for Blended Hydraulic Cements
3.	ASTM C 618-08a	Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
4.	ASTM C 989-11	Standard Specification for Slag Cement for Use in Concrete and Mortars

1.02 DEFINITIONS

A. [1]Reinforced Structural Mass Concrete (RSMC): Reinforced Structural Mass Concrete is any placement [thicker than [2.5]-feet]. RSMC is expected to generate significant heat during curing and will require the use of a reinforced structural mass concrete mix design and temperature control measures.

1.03 SUBMITTALS

A. RSN 03 30 00-X, Concrete Placement Schedule:

1. Written plan describing location, sequence, and date of concrete placements scheduled. [In addition, methods used to control temperature of concrete placements.]
2. Complete, detailed concrete placement schedule showing the Contractor's plan for placement of individual features, units, and other elements of concrete work.

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[1] Reinforced Structural Mass Concrete: Should be called out on drawings for concrete placement where the minimum section is greater than 2-3 feet. Without preventive measures, heat generated during curing will result in unacceptable stresses due to thermal gradients in this concrete before adequate strength is developed. Preventive measures include: increased pozzolan content, lower placement temperature, insulating during curing (as needed). ConcreteWorks software may be useful to determine limits, http://texasconcreteworks.com. Non-reinforced gravity dam mass concrete should be included in a separate specification section. Contact MERL for assistance.

B. RSN 03 30 00-X,Reinforced Structural Mass Concrete Temperature Control Plan[2]

1. Mix designs for concrete in Reinforced Structural Mass Concrete .
2. Methods used to control temperature.
   a. Anticipated maximum temperature of concrete in each placement and methods used to calculate maximum temperature. Complete, detailed concrete placement plan for reinforced structural mass concrete, showing the name and manufacturer of the temperature monitoring device, placement locations and intended temperature monitoring plan.
   b. For each reinforced structural mass concrete placement
3. Detail as necessary to show location, sequence, and date of concrete placements scheduled for each item. Identify how sequencing of adjacent reinforced structural mass concrete placements will be determined.

PART 1 PRODUCTS

2.01 CEMENTITIOUS MATERIALS

A. Cementitious materials options:

1. Specified portland cement plus percent by weight specified pozzolan specified in Table 03 030 00A - Mix Table.
2. [Specified portland cement plus specified percent slag cement by weight specified in Table 03 030 00A - Mix Table.]
3. [Blended Hydraulic Cement meeting the percent replacement of pozzolan or slag above.]
4. Ternary blends.[3]

B. Portland cement:

1. ASTM C 150, Type [4][____].
   a. Meet equivalent alkalies requirements of ASTM C 150 - Table 2.

C. ASTM C 595, Blended Hydraulic Cement, IP or IS.

   a. Meet equivalent alkalies requirements of ASTM C 595 Table 2 Option G and Table 3.

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[2] For use when Reinforced Structural Mass Concrete is used.

[3] Ternary blends are a combination of three or more cementitious materials, sometimes necessary to obtain special properties.

[4]Specify Type (I, II, III, or V). Default is Type II (moderate sulfate resistance). Consult with MERL for selection.

D. Pozzolan:

1. ASTM C 618, Class F, except,

   a. Sulfur trioxide, maximum: 4.0 percent.

   b. Calcium oxide, maximum: 8.0 percent.

   c. Loss on ignition, maximum: 2.5 percent.

   d. Test for effectiveness in controlling alkali-silica reaction under supplementary optional physical requirements in Table 3 of ASTM C 618. Use low-alkali cement for test.

   e. Does not decrease sulfate resistance of concrete by use of pozzolan.

      1) Demonstrate pozzolan will have an "R" factor less than 2.5.

      2) R = (C-5)/F

      3) C: Calcium oxide content of pozzolan in percent determined in accordance with ASTM C 114.

      4) F: Ferric oxide content of pozzolan in percent determined in accordance with ASTM C 114.

E. Slag Cement:

1. ASTM C 989, Grade [100 and/or] 120.

2.05 ACCESSORIES

A. Temperature Monitoring Devices for monitoring Reinforced Structural Mass Concrete placements :

1. A device to monitor and record the concrete temperature as a function of time may consist of a Thermocouple, Thermistor, or Resistant Temperature Device (RTD)

2. Operating range: 14 degrees Fahrenheit to 185 degrees Fahrenheit.

3. Accurate to 1.8 degrees Fahrenheit, or 1 degree Celsius minimum.

4. Records temperature at least once per hour, and the max/min over a 24-hour period can be obtained. Records data in a digital format.

2.06 MIX PROPORTIONS

A. Design mix in accordance with Table 03300A – Mix Table: [5]

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[5] Table needs to be revised for project requirements. Delete unnecessary rows. Fill in all field for mixes listed.

Table 03 30 00A – Mix Table

Mix No	Feature	f'c (lb/in^2)	Max. w/cm *[6]	NMSA **	% Pozzolan*** A: Class F Ash B: Slag Cement C: Silica Fume	Slump	Air Content [7]	Notes
3	RSMC	4500		[No. 467]	A: 35 ± 10 or B: 50-80	1 – 3	TBD	[1,2]

*Maximum water/cementitious ratio. Cementitious to mean cement plus pozzolan.
**Nominal Maximum Size Aggregate
*** Percent of specified cementitious by weight.

NOTES:

1 – Ternary blended cementitious materials which meet the specifications may be submitted for approval by the COR.
2 –RSMC will require the use of a reinforced structural mass concrete mix design and temperature control measures.

2.07 CONCRETE TEMPERATURE

A. RSMC temperature at placing: 50 to 70 degrees F (10 to 21 degrees C).

PART 3 EXECUTION

3.01 PREPARATION

3.02 TEMPERATURE CONTROL OF REINFORCED STRUCTURAL MASS CONCRETE

A. Design Reinforced Structural Mass Concrete Temperature Control Plan for a maximum temperature in concrete of 155 degrees F during curing and protection.

B. Install at least 2 monitoring devices at each of the following locations:

1. At the center of the thickest sections.
2. Along the coolest anticipated concrete face(s) at a depth of 1.5 inches from the surface.
3. Every 150 cubic yards of concrete or in each individual placement.
4. Monitor ambient temperature for the duration of the temperature control.

C. Install temperature monitoring devices prior to placement according to approved temperature control plan.

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[6] Select based on durability requirements (freeze/thaw, sulfate resistance) and desired strength.

[7] Select air based on maximum aggregate size and freeze-thaw environment, see Concrete Manual or ACI 318.

D. Maintain temperature differentials between the temperature monitoring devices at the thickest section and the outside face of not more that 35 degrees F.

1. Insulate with concrete blankets or other approved insulation as necessary.

2. Do not remove blankets until the sensors at the outside face are with 35 degrees F of the lowest ambient temperature in a 24 hour period.

E. Evaluate temperature records daily and make adjustments to temperature control methods as necessary.

F. Temperature information shall be available to COR in digital format daily.

G. If the internal concrete temperature of any placement exceeds 155 degrees F, or if the difference in temperature between the center of the thickest section and the outside face exceeds 35 degrees F then a revised temperature control plan must be resubmitted and approved prior to any further placements.

3.01 PLACING

H. [Seven days minimum between adjacent placements, or as approved by COR.]

1. [8]COR may require more than [7] days between adjacent placements based on temperature control plan or project temperature data of reinforced structural mass concrete.

3.03 CURING

A. Water curing:

1. Obtain approval of COR for water curing methods used for Reinforced Structural Mass Concrete. Additional requirement may be necessary to prevent rapid cooling of the concrete surfaces.

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[8] RSMC only