Chapter 7
Caltrans Advancements / High Performance Concrete

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7 Caltrans Advancements / High Performance Concrete

Introduction

Conventional (or classical) structural concrete used throughout the 1930s and still much in use today is a concrete mix design having 500-700 pounds per cubic yard (pcy) of cementitious material that gains its design strengths of 2,500 to 4,500 pounds per square inch (psi) in 28 days. High consistency for classical concrete was a 6 to 7-inch slump. The earliest special performance requirements were:

- Entrained air for freeze-thaw resistance
- Fine grind cement or manufactured cement with high calcium alumina content for rapid strength gain
- Alkali content limitations to mitigate ASR
- Tri-calcium aluminate (C_3A) limitations for durability improvements

The Federal Highway Administration (FHWA) has promoted a broad category of concrete mix designs that meets specific fresh and/or hardened criteria. These concrete mix designs are collectively termed High Performance Concrete (HPC), indicating enhanced particular properties when compared to more conventional concrete.

High Performance Concrete

Currently, with technological advances, concrete can be designed to meet an even larger variety of performance requirements than practical or even possible only 20 years ago. During the 1990s the term high performance concrete evolved into a catch-all name for existing special concretes that met special performance requirements that could not be uniformly attained using conventional materials and handling practices. HPC incorporates specific materials in the mix design and/or specific placement procedures to produce concrete with enhanced specific performance criteria. The Federal Highway Administration (FHWA) through the Research and Technology Report Center has established eight strength and durability performance criteria for evaluating HPC designs:

- Compressive strength
- Modulus of elasticity
- Shrinkage

- Creep
- Freeze-Thaw
- Scaling
- Abrasion
- Chloride permeability

Other special characteristics such as reduced hydration heat for temperature control, low and high unit weights, consistency, workability and curing enhancements can also be considered for HPC specifications. Supplementary cementitious materials and admixtures are almost always needed to achieve the desired results. Aggregate gradations may be more highly optimized for workability or specific hardened concrete characteristics. Extensive suitability testing of a HPC mix design is often required prior to use on a project as the margin for error is more limited with enhanced performance requirements.

FHWA maintains a web site that provides downloads of the High Performance Concrete Structural Designers' Guide, which details the material selection process: http://www.fhwa.dot.ca.gov/publications/research/infrastructure/structures/hpc/12042/index.cfm (visited 6/10/10).

FHWA also maintains a web site which details the criteria for High Performance Concrete for Highway Structures:
http://www.fhwa.dot.gov/bridge/HPCdef.htm (visited 6/10/10).

FHWA also publishes HPC Bridge Views, an electronic newsletter in which Caltrans has provided significant contributions:
http://www.hpcbridgeviews.com/directory.asp (visited 6/10/10).

HPC strength and durability characteristics are further described in the following sections.

High Early Strength

One of the earliest enhanced specific performance criteria that engineers desired was a high early strength gain for a quicker return to service, accelerated construction sequences, and to compensate for low ambient temperatures. Early strength is not an isolated design goal, as durability and shrinkage must also be considered. There are now very high early strength concretes that are durable and have shrinkage properties that are comparable and sometimes superior to classical concretes.

As early as 1918, bauxite and limestone were combined to form High Alumina Cements (HAC), which achieved accelerated strength gain. Rapid strength gain was due to the high reactivity of mono-calcium aluminates, comprising 50-60% of the cement. Unfortunately it became apparent that the hardened cement was unstable and that significant strength loss due to crystalline changes could occur years later under particular temperature and moisture conditions. For years, Type III Portland cement with its extra-fine grinding, was used to reduce the time needed to achieve design strength. Using Type III cement mixes with high-volume cement contents made typical 28-day strengths attainable in less than a week. One drawback with Type III cement use is that the resulting concrete has a much higher dry shrinkage when compared to a classical concrete; this was especially true when used in combination with calcium chloride, which was used as an admixture to accelerate strength gain. Another major drawback to calcium chloride is the corrosion potential of the reinforcing steel is greatly increased. Non-chloride accelerating admixtures have been developed to overcome the corrosive potential but dry shrinkage is still an issue.

The need for early strength concretes increased after most of our infrastructure was built because maintenance and repair became more frequent and required greater attention as the aged highway system approached and exceeded its initial design life. The pre-cast concrete industry also had incentive for early strength concrete. The longer the time required to gain strength, the greater the amount of space required to store the elements. Reducing the time needed before form removal translated to a faster casting cycle and more efficient use of forms.

Including water reducing admixtures with high cement content concrete mixes resulted in lower water-cement ratios. Although the desired early strength was achieved, the ultimate strength was much higher than needed. Precast manufacturers took advantage of high cement contents, fine grind, and water reducers to more quickly meet required strengths for form release. With or without fine grind cement a very high early strength concrete resulted, often with much higher ultimate strength than the structural needs of the pre-cast element. The mixes tended to be less workable with a consistency that is often described as “sticky,” i.e., the concrete tends to stick to working tools and not trowel well.

The need to make repairs on existing structures with as little impact on traffic flow as possible inspired interest in the development and control of rapid strength gaining patching materials. Laboratory tests and field trials were performed on some promising high early strength materials and techniques. Evaluation of the field trials was a coordinated effort between material, design and construction engineers. In the 1980s Caltrans Materials and Research Engineer, Leo Ferroni, oversaw a 6-year study which included rapid set patching materials. This work is documented in Caltrans research report “New Materials and Techniques for the Rehabilitation of Portland Cement Concrete” (FHWA/CA/TL-85/16, Paul Krauss, October 1985). California Test 551 as well as current specifications for Caltrans use of bagged rapid setting concretes for patching material resulted from this research project. Material suppliers still modify their bagged rapid setting patching materials to meet standards resulting from the study. These rapid setting patching materials were available for use in 50-100 pound bags, requiring only small equipment and water for mixing at the construction site. The rapid set materials were ideal for spall repair, bridge joint header construction, or for bonding dowels into existing structures proved to be ideal.

The experience with bagged material led to using rapid setting materials for larger elements, such as approach slabs. The basic cementitious materials were similar, but removing and replacing larger highway elements within an eight-hour shift required mix designs that facilitated the mixing and placing of cubic yards rather than cubic feet. Smaller concrete elements such as asphalt expansion dams (headers), deck patches, and paving notches on retrofitted approach and departure slabs had volumes low enough to be conducive to using pallets of 50-pound bagged material and small mixers with capacities of only several cubic feet. However replacing whole approach slabs or panels might require several transit mixers of material making small 50-100 pound concrete bags inappropriate.

Rapid setting concrete was used in approach slabs for the La Cienega structure on the I-10 Santa Monica Freeway after the Northridge Earthquake in 1994. Some may find it interesting that the contractor proposed casting the entire structure with rapid strength concrete. The proposal was denied because of uncertainties concerning placement logistics, heat of hydration and long-term creep. The rapid setting concrete was wet batched into transit trucks, just as with typical conventional concrete, and exceeded compressive strengths over 3,000 psi in just a few hours. Set time was controlled with admixtures to achieve the needed transit and working time.

Performance specifications were later written for the rapid setting concrete material and the material meeting the requirements was termed Fast Setting Hydraulic Cement Concrete (FSHCC). The first use of this specification in California was in 1996 on a contract to replace PCC paving panels for the I-5 "Boat Section[1]" in Sacramento. A continuous volumetric mixer batched the FSHCC on site, with the mixing performed by a chute augur at the point of delivery. Production rates were about 50 cubic yards per 8 to 10-hour shift. At about the same time, smaller approach slab and paving panel replacements were done from transit mixers batched from a central batching plant.

Competitive suppliers of calcium sulfoaluminate cement, the cement used in several bagged patching materials meeting Caltrans and ASTM Type K cement specifications, pursued contracts having slab replacement projects that originally specified finer grind cements with calcium chloride admixtures. With Caltrans encouragement, these rapid setting cements

[1] The "Boat Section" is a three-quarter-mile section of I-5 in Sacramento from R Street to the Capitol Mall overcrossing. Caltrans engineers call this area the "Boat Section" because it is below the water level. It was named back the late '60s when it had to be drained as one of the last portions of Interstate 5 to be completed. Without pumps, the area would fill up with water during storms.

replaced the Portland Cement by contract change orders (CCO). The CCOs were typically at credit to the State as the savings from the increased number of paving panels that could be replaced in a given window more than offset the cost of the higher priced cement. One of the first projects was about 50 cubic yards of pavement panel replacements done on the Los Angeles I-110 Freeway in 1995. Another example was 180 cubic yards of pavement slab replacements on the Los Angeles I-10 Freeway near the I-57 in 1996 over several night shifts.

In April 1997, the Long Life Pavement Team created a Task Force with the objective of reaching a new plateau in roadway reconstruction; replacing a section of freeway with new 40-year concrete during a weekend freeway closure. The publicized slogan was "give us a weekend and we will give you forty years." The mission for the Task Force was "Get in, Get out, and Stay out." A construction strategy using very high early strength concrete on the production scale of a conventional large-scale concrete placement was required that entailed solving engineering problems as well as overcoming reluctance to use cements having setting characteristics very different than Portland Cements on such a large scale. There was some resistance from a large portion of the concrete industry, portions of academia and even from some within Caltrans. The reluctance was overcome and use of FSHCC became acceptable and more widespread as a result of the successful work of the Task Force.

The Task Force outlined a well thought out strategy. To develop the expertise and gain experience without risk to major traffic impediment, the first large scale uses were done behind permanent (i.e., duration of the contract) lane closures. The first was done by a CCO written to replace the Portland Cement with calcium sulfoaluminate cement in an existing contract adding a High-Occupancy Vehicle (HOV) Lane on the I-605 Freeway in Los Angeles. In July of 1997, 600 cubic yards were placed in one shift, about the same rate of production as conventional concrete on this project. The concrete was batched from a central batch plant into transit trucks and placed using a 12-ft wide slip form paver, the first time such a mix design was placed with such equipment. Set time was controlled by citric acid to facilitate transportation time and placement needs.

In August of 1998, again where return of traffic was not an issue, an even larger scale paving operation was done by substituting a FSHCC-mix design for the conventional mix design in a construction project that used a 24-ft wide slip form paver for the 60/71 Interchange in Los Angeles. The concrete was central batched and transported by end dump trucks to increase the production rate. Around 4,000 cubic yards were placed over several days. Two different cement suppliers were used under CCO. Daytime air temperatures reached 100°F, making control of set time even more difficult. The concrete in the 60/71 Interchange construction project consistently achieved strengths ranging from 3,000 to over 4,000 psi in 4 hours; a real success under adverse hot weather.

Though two FSHCC projects during this same time period did not fare as well, a contract to replace slabs on the I-5 near Burbank between January and May 1999, was an overwhelming success. The contract required 400 psi in flexure (3,500-4,500 psi in compression) in 3 hours. The contractor was consistently achieving this in 2 hours after placement. During the job, nighttime temperatures sometimes dropped below 45°F. Transit trucks were batched from a central batching plant that was 15 miles from the site. For this job all but the cement itself and a portion of the mix water was batched from the plant into the truck. The cement was added via 1 to 2-ton super bags that were filled and weighed with the amount of cement needed for the predetermined 6 to 7 cubic yard batches used in the transit trucks. The paving panel replacement production rates were around 100 cubic yards per shift. Though not on the same production rates as the I-605 or the 60/71 projects, a significant amount of FSHCC was centrally batched and transported a substantial distance prior to placement. The panel replacement project proceeded without failure, shift after shift.

Smaller jobs were done throughout the State during this time period. Strengths typically expected over weeks were achieved consistently and controllably in a few hours. The plastic and set characteristics were engineered to meet specific needs of placement and design loads under a variety of conditions. Set times were controlled to meet travel time, placement work including anticipation of varying queue times and still meet design strengths before shift end.

In March of 1999 the largest FSHCC project to that date was awarded. Over 20,000 cubic yards of FSHCC were used to replace panels and even whole lane miles on the I-10 Freeway in Pomona. For one section, two lane miles of the existing truck lane were removed and replaced with FSHCC during one continuous operation. On the weekend of October 22, 1999 work began on this portion of the freeway at 10 p.m. Friday night and continued until 3 a.m. Monday morning. About 5,000 cubic yards were placed, all concrete met specification requirements for strength. The placement was continuous save for a few hours when the central plant electrical system broke down and when the roller strike-offs wore out and required replacement. The remaining 15,000 cubic yards were placed during nighttime shifts, lanes being open to traffic daily by 5 a.m. For the duration of this job commuter traffic was interrupted only one time due to a batching error. This project was an overwhelming success winning multiple awards:

- 2001 ACPA Excellence in Concrete Pavement Award for Restoration
- 2001 ACPA Excellence in Concrete Pavement Award for Transportation Management
- 2001 California "Tranny" for Transportation Management
- 2001 Marlin J. Knutson Award for Technical Achievement
- 2001 Caltrans Partnering Award (Bronze)
- 2002 California Excellence in Transportation Award for Innovation

Following the success of the I-10 Pomona project, resistance to using this technology withered. Portland Cements mixed with large dosages of powerful accelerators and High-Range Water Reducers were developed that achieved similar results thus increasing competition among suppliers. A few bridge structural elements have been placed using fast setting rapid hardening concretes. Within a short time this technology became a much more standard tool for Caltrans when faced with repairing or rehabilitating existing facilities.

Today, high early strength concrete tests attain 4,000 psi in 3 hours using mobile mixers. In special situations, as much as 5,000 psi in as little as 2 hours has been achieved using mobile continuous mixers. Current FSHCC specifications include shrinkage restrictions and thermal stability requirements to limit possible HAC crystal changes to improve long-term durability.

High Strength Concrete

High strength concrete (HSC) is defined by ACI as being above 6,000 psi; that being said, HSC typically tests from 8,000 psi up to and exceeding 20,000 psi. Depending on the design application, strength development times can exceed 90 days. Precast manufacturing yards used extensive vibration for low slump mixes placed into well built forms that accommodate rugged treatment as voids are filled during consolidation. But HSC can now also be designed to have high slump or be self-consolidating making HSC more readily available for cast-in-place concrete.

In 1994 “High Strength” concrete was specified for the construction of the Main Street Overcrossing (OC) in District 12 (Orange County Rt. 5, PM 33), where a cast-in-place post-tensioned box girder structure design required that f'_c = 6,500 psi in one of the superstructure frames. Several months of trial batches failed to meet the required 7,100 psi strength (f'_c + 600 psi for the trial batch). The contractor then proposed a mix having 900 pounds of cement per cubic yard. As 800 pounds was the maximum allowed by the Standard Specifications of that contract, a change order was required prior to use. The 900-pound mix ultimately reached 7,800 psi. The contractor made a trial placement of this rich mix in a footing, but still had reservations about being able to consistently attain the 6,500 psi requirement and provide a mix that was workable enough to make a deck that will meet the specifications. Following construction of the superstructure the Structure Representative suggested in a memo to the Chief of Structure Construction that such concrete requiring the large dosages of water reducers to make strength was “only marginally suitable for bridge superstructures”. The memo also reported only minor deck cracking which did not require remedial repairs. Over 13 years later, in 2007, a bridge maintenance inspection reported only minor deck cracking; verifying that the “high strength” cement content concrete did not result in long-term or short-term cracking issues.

In 2000, six years after the Main Street project, a concrete strength requirement of f'_c = 5,900 psi for the Gene Autry Bridge, again in District 12, also saw a request by the contractor to increase the cement content above the limit. In this case the recently added 25% fly ash requirement was given as the need for the request. However, the request was denied and the concrete requirements were achieved within the specification limits. Though in this case, also documented in a memo from the Structure Representative to the Chief of Structure Construction, 35 days were given to achieve a slightly lower required strength; one can still conclude some advancement in achieving high strength concrete took place for cast-in-place concrete. However, it was precast concrete elements that brought more rapid advancements in high strength concrete, transmitting this advancement to cast-in-place concrete.

Pre-cast concrete structure elements benefitted greatly from techniques used to achieve HSC. Pre-casters gained experience rapidly through their efforts to achieve high early strength for form removal, storage, and transport. One of the first, if not the first, bridge design to take advantage of pre-casters abilities to achieve high strength was the Sacramento River Bridge in District 2, on I-5 near Anderson, just south of Redding. Construction began in 2001 on the first bridge in California to use spliced high strength concrete pre-cast bulb-tee girders and a cast-in-place concrete deck. Due to environmental constraints the design span length was 154 feet and the structural depth was 6.5 feet. Another restriction was that work could only be done between May and October, and all falsework was to be removed by October 15. Specified concrete strength (f'_c) for diaphragms and the girders including closure pours was 8,700 psi, significantly higher than the Main Street Overcrossing in District 12. The pre-cast yard located in Oregon achieved the required girder strength, and with the aid of the Structure Representative, the local concrete supplier followed suit producing the high strength cast-in-place concrete. Due to the time constraints the cast-in-place concrete needed to reach strength in 10 days; the average 10-day compressive strengths (f_c) were 10,000 psi. In comparison with the District 12 experience, one can see how fast acceptance of HSC technology developed within Caltrans.

The large bridge construction projects of the District 4 Toll Bridge Program accelerated the use of HSC. The new Zampa Bridge that replaced the 1927 Carquinez Bridge was the first new bridge to be built under this program. Work on this new suspension bridge, the first major suspension bridge in the United States since the Verrazano Narrows Bridge in New York, began January 2000. The two cast-in-place concrete towers consisting of two legs tied together with struts rise 426 ft above the water. Though HSC was not specified for any of the structural elements, the lower tower legs utilized HSC to aid the construction schedule. The towers were constructed using jump forms that were supported by attachments to the previous lifts. Approximately 30 lifts using the jump forms were required for each leg, the height of each lift about 13 feet. Each lift required a strength of 1,200 psi before the form could be “jumped” to the level of the next lift. To achieve this strength within a 12-hour period, HSC was implemented by change order since it exceeded maximum cementitious content specifications and used metakaolin for the first time in a major Caltrans structural element. The mix consisted of 940 pounds per cubic yard (or 10-sack) cementitious material and 1/2 in. maximum coarse aggregate. The cementitious materials were 705 lbs (75%) Type II Portland Cement, 188 lbs (20%) class F fly ash, and 47 lbs (5%) metakaolin. The result was an 8 to 10-in. slump (flowing) mix that reached 1,500 psi by 12 hours and exceeded 10,000 psi at 28 days. All aggregate was from local sources.

Both the 1.2-mile Skyway pre-cast segmental box bridge, a portion of the new easterly spans of the San Francisco-Oakland Bay Bridge (SFOBB), and the 1.4-mile Benicia-Martinez light weight concrete cast-in-place segmental box bridge marked a substantially increased HSC volume. For the Benicia-Martinez Bridge HSC concrete was used because of the modulus of elasticity (MOE) requirements and the desire to have early strengths high enough to prestress and move the traveler forms to the subsequent cast-in-place segment. The Skyway was designed with a HSC strength requirement of 8,000 psi for the segment box and pier tables. MOE requirements in the segments increased the need for somewhat higher compressive strengths. Both bridges utilized cementitious contents above 900 pounds per cubic yard. The Benicia-Martinez Bridge segments were as high as 980 pounds per cubic yard while a typical mix for the HSC in the Skyway had 940 pounds per cubic yard. The lightweight aggregate for Benicia-Martinez came from South Carolina while all the aggregate for the Skyway superstructure came from Canada. Figure 7-1 shows the compressive strength results for cast-in-place concrete for the Skyway. Figure 7-2 shows the compressive breaks from the pre-cast yard in Stockton where the segments were fabricated.

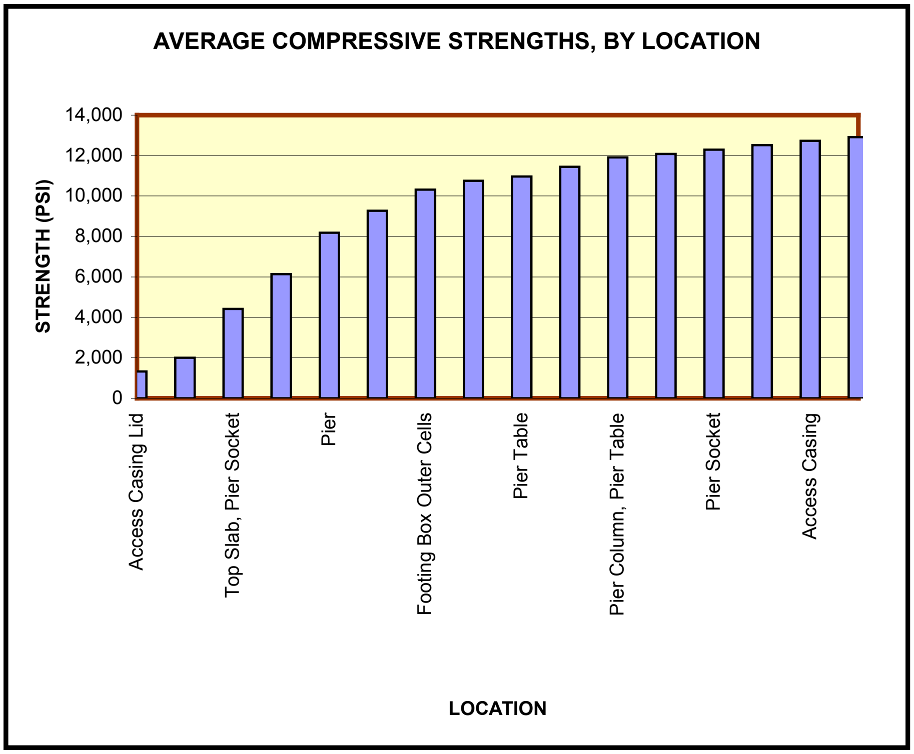

Figure 7-1. Skyway Cast-in-Place Compressive Strengths.

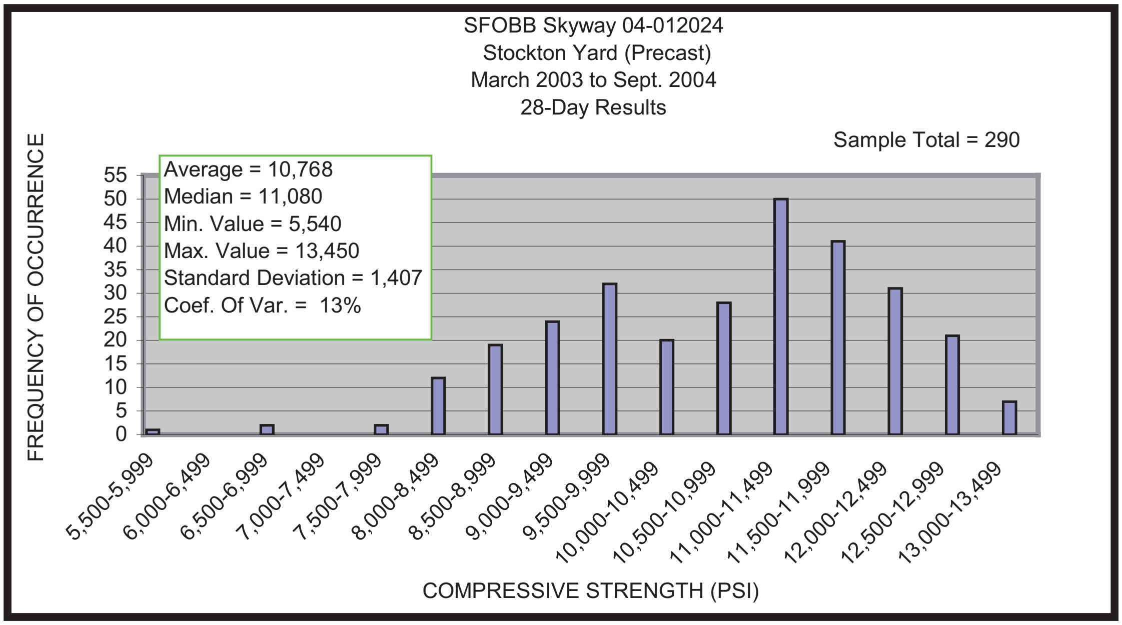

Figure 7-2. Stockton Precast Yard Compressive Strengths for Skyway Elements.

HSC has come into its own since the early Caltrans uses such as the 6,500 psi for the Main Street Bridge. Though by 2008 making high strength cast-in-place concrete was no longer a major concern for Caltrans designers, the HSC was not a given in all areas within the State.

The Angeles Crest Highway Bridge is a 208-ft long single span, precast, prestressed concrete, spliced bulb-tee-girder bridge, spanning an area that was washed out during the spring thaws of 2006 and 2007 along Scenic Route 2, northwest of the city of Los Angeles within the Angeles National Forest. Placement logistics of the precast girders played a significant role in the selection of HSC. The bridge consists of six 96-inch deep girders with a 7.7-inch thick cast-in-place concrete deck. The girders were shipped to the site in three segments with lengths of 56, 92, and 56 ft (See Figure 7-3). With 2 closure pours of 2 ft, the total girder length was 208 feet. The girders were spliced together on the ground in a staging area near the bridge location and then moved onto the abutments. Intermediate and end diaphragms were then cast followed by placement of the concrete deck. The specified concrete compressive strength for the precast girders was 8,500 psi at 56 days. With the bridge located in a freeze-thaw environment at an elevation of 6,500 ft, there was an additional requirement for air entrainment. Air entrainment reducing the strength of concrete meant the concrete mix without air entrainment needed to achieve strength of about 8,500 psi.

The specified air-entrained HSC for pre-cast girders went without incident. But when the time came for the closure pours of cast-in-place concrete the contractor found it difficult to obtain the 8,500 psi concrete from local suppliers, which could have resulted in a major issue if the opening of the bridge was significantly delayed. With technical guidance of the Caltrans Structure Representative, the problem was solved and closure pour concrete meeting all the specifications was successfully placed. Following completion, the Angeles Crest Bridge won the Precast/Prestressed Concrete Institute (PCI) Design Award for 2009.

Figure 7-3. Angeles Crest Highway Bridge Girder Placement.

The 10,000 cubic yards of HSC used to construct cap beams and bent caps at the eastern and western ends of the Self-Anchored Suspension (SAS) bridge portion (the signature structure) of the San Francisco-Oakland Bay Bridge (SFOBB) used both flowing (8 to 9-in. slump) and self-consolidating concrete (SCC). Figure 7-4 (Bay Bridge May 6, 2008) shows strength vs. time for the first pour of SCC on the W2 Cap Beam. This first placement had 56-day breaks approaching 14,000 psi. One quantitative change in the bent cap concrete was that the high strengths were achieved with only 800 pounds per cubic yard cementitious materials and high performance SCMs such as silica fume or metakaolin were not used. About half the pours were flowing and half were SCC. Pours ranged from 300 to 2,000 cubic yards, the typical pour being 1,500 cubic yards.

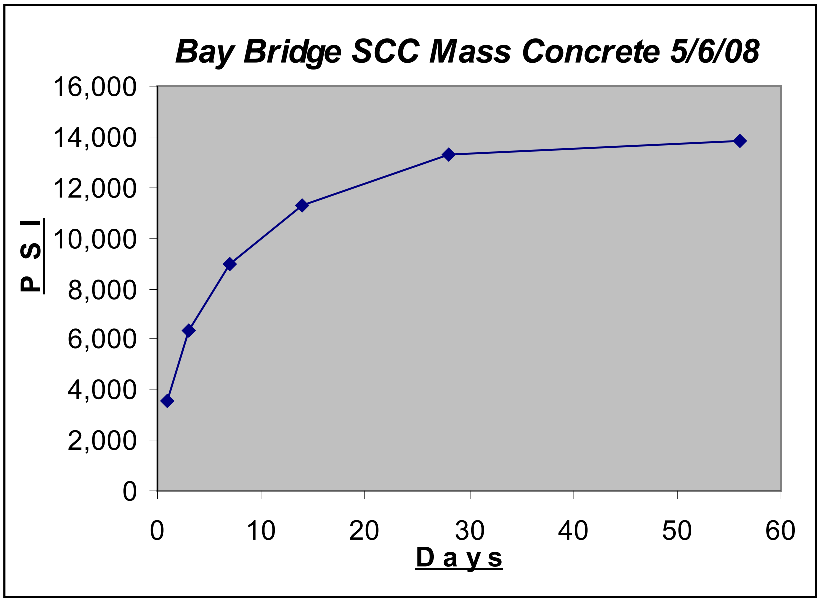

Figure 7-4. Bay Bridge Time vs. Strength Curve.

Workable high strength mixes require inclusion of workability enhancing admixtures. Table 7-1 shows the high strength mix designs for the SAS cap beams. Note the only differences are the admixtures and the coarse-fine aggregate ratios.

Table 7-1. Bay Bridge Concrete Mixes.

Material	High Slump	SCC
Cement, Type II-V	600 lb/yd^3	600 lb/yd^3
Fly Ash, Class F	200 lb/yd^3	200 lb/yd^3
Sand	1,180 lb/yd^3	1,497 lb/yd^3
Coarse Aggregate 1/2″max	1,753 lb/yd^3	1,424 lb/yd^3
Sand portion of aggregate	41%	52%
Type F (HRWR) admixture	35 oz/yd^3	80 oz/yd^3
Viscosity Modifying admixture	0	12 oz/yd^3
Stabilizer / Water reducer	15 oz/yd^3	30 oz/yd^3
Water	264	264
w/cm ratio	0.33	0.33

High strength concrete requires a blend of selected materials and attention to all production aspects. Cementitious materials up to 980 pounds per cubic yard and possibly more have been used on Caltrans projects. As shown above in Table 7-1, high strength concretes, even SCCs, do not necessarily need cementitious contents above 800 pounds per cubic yard. SCMs are useful for conversion of calcium hydroxide to calcium silica hydrates. Because of their small size, SCMs also improve the particle size distribution. Trial batching is needed to determine the optimum cementitious blend and compatibility of cementitious materials.

Aggregate cleanliness, shape, texture, and gradation require attention. Any contaminants like clays and rock dust increase the cementitious paste requirements. Maximum size may be less than one-half inch. Coarse crushed materials provide more surface area to bond with cementitious materials. Trial batching will be needed to determine the optimum gradation. Although it reduces ultimate strength, air entrainment is mandatory for structural elements in freeze-thaw environments.

Trial batching is needed to provide a workable consistent mix. When high strength components are mixed, attention must be given in order of placement in the mixer (as sticky mixes or balling material may happen), temperature, mixing time and the order that admixtures are introduced to the mix. All have impacts on the fresh concrete properties.

Workmanship as in any structural concrete pour is critical when placing high strength concrete. Delivery time must be considered. An axiom in Structure Construction is that workability should not be changed with additional water for any mix; this is even more important for high strength mixes since there is a smaller margin for error as all the mix components have been selected for their high qualities. Although high strength concretes may have large slumps, consolidation is still essential for strength development. Curing is all important; controlling temperature reduces cracking and increases durability. Temperature history is often captured as part of the quality process while the structural element is curing.

Quality control, important for any concrete, is even more important for high strength concrete as there is less tolerance for a mixing or batching error. Product uniformity, achievement of designed strength requires strict enforcement of material standards and placement processes by sampling and inspection throughout the mixing and placing process.

Self-Consolidating Concrete

Self-consolidating concrete is a concrete that flows without segregation to a level state without the use of vibration. Over the 30 years, SSC has rapidly grown in acceptance to the point where the concrete is now part of the 2010 Standard Specifications, as section 90-5. While being developed, SCC was referred to as self-levelling concrete, self-compacting concrete and highly-workable concrete. SCC is allowed for all precast construction, and gaining acceptance in bridge, building and tunnel construction, SCC flow characteristics are achieved by:

- Reducing internal friction through the use of additional fine materials;
- Reducing surface tension by maintaining low water-cementitious ratio with a high-range water-reducing admixture like polycarboxylate ethers;
- Mitigating segregation by reducing the water-cementitious ratio that also can reduce bleeding, and
- Increasing apparent viscosity with a viscosity modifying admixture (VMA) and use of additional fine materials.

Figure 7-5 shows a typical SCC placement tremie.

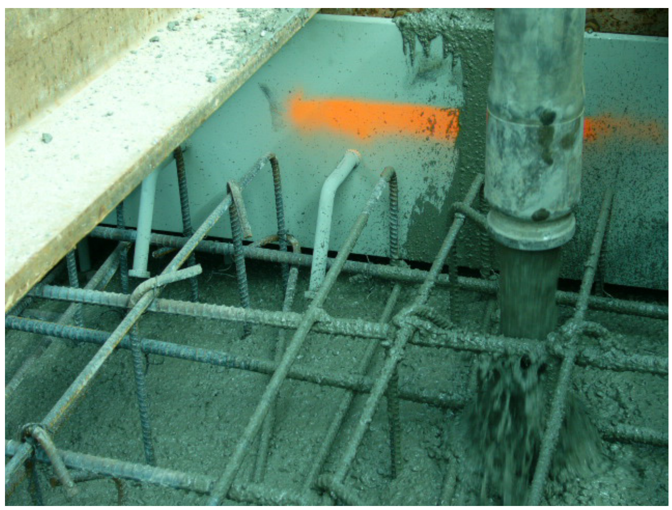

Figure 7-5. SCC Placement Tremie.

SCC resulted from the combinations of two advances in admixture technology. The first was the continued development and refinement of high-range water reducers. The second was the development of anti-wash admixtures. Anti-wash or anti-washout admixtures (AWA) were developed for concrete placed under water to prevent water exposed concrete surfaces from being diluted or even deeper portions of the fresh concrete from being washed away. The AWA makes the concrete more cohesive though still flu d by inhibiting the displacement of free water in the mix by the heavier constituents of aggregate, cementitious material, etc. When mixed with a neat cement paste of medium consistency the paste can be placed in a container of water and the water remains clear. Conversely a neat cement paste without AWA will make the water cloudy. Caltrans used AWA to mitigate cement fines getting suspended and settling in rivers or streams due to placement of concrete. The Caltrans Transportation Laboratory (Translab) tested the admixture by placing a lined paper behind a glass container, placing a mortar mix incorporating the AWA in the mix, and checking to see if the lines on the paper could still be seen through the water. A mortar with AWA placed in the container of water left the water clear keeping the lines on the paper visible. A mortar without AWA immediately clouded the water making it impossible to see through.

AWAs were the predecessor of viscosity-modifying admixtures (VMA). Very fluid mixes whether fluid by excess water or by use of water reducers will cause a concrete mix to segregate. VMAs prevent or mitigate this, and reduce bleeding which is a form of segregation. A very fluid mix that is even self-leveling will still be cohesive with a VMA. Also increasing the fines in the mix mitigates segregation. The aggregate fine-coarse ratios are modified with more fine material for SCC mixes. Additionally, more cementitious materials may be used or other materials such as limestone dust may be added.

Caltrans first used SCC on a large scale during the seismic retrofit of the Richmond-San Rafael Bridge Seismic Retrofit Project when the material was added by CCO. Large underwater steel shells about 14 ft in diameter were filled with concrete without removing water. Due to the size of the shells and the distance a conventional tremie concrete travels, placement of concrete would have required several simultaneous tremie placements to prevent intermixing of water with the concrete mix. SCC allowed for one tremie used down the middle of the shell as the SCC flowed from the tremie to the far perimeters of the placement. SCC was used on the Skyway project, again the material was added by CCO. The footing boxes were filled with both normal weight and lightweight SCC. Figure 7-6 contains two photos; a footing box and the pour front 45 ft from discharge. Note the cohesiveness of the mix. Table 7-2 shows SCC mixes used for the Skyway.

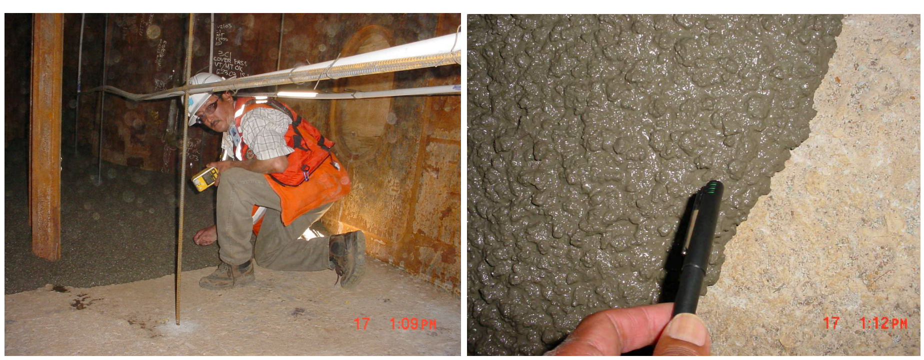

Figure 7-6. Footing Box and Pour Front 45 ft from Discharge.

Table 7-2. Skyway SCC Mixes.

Ingredients	Mix 1	Mix 2
Water, lb	291	267
Portland Cement Type II, lb	291	334
Fly Ash, lb	440	0
Ground Granulated Blast Furnace Slag, lb	0	334
Lightweight “Realite” one-half inch coarse aggregate	875	0
Lightweight “Port Costa” fine aggregate, lb	620	0
Normal weight “Sechelt” fine aggregate, lb	580	1,523
Normal weight “Sechelt” coarse aggregate, lb	0	1,523
HRWR, fluid ounces	240	32
VMA, fluid ounces	100	52
Recover Stabilizer (Type D Retarder)	24	20
Slump Flow Test - Diameter of Spread, inches	28	25

SCC was also successfully used to fill a large void under a steel support plate as part of a sliding plate joint assembly. The SCC was pumped through 2-inch diameter pipes that penetrated the steel plate. Figure 7-7 shows photos of the placement pipe and subsequent grouting of the hole left when the pipe was removed. The SCC performed as a high strength grout, but it was a concrete. SCC became the specified material for this use on all the sliding plate joints to be placed on the east spans of the SFOBB.

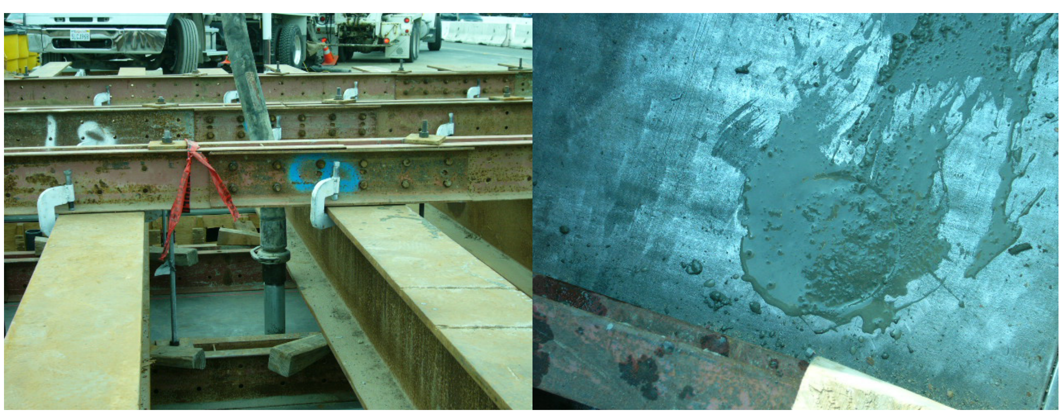

Figure 7-7. Concrete Pump Hose Connected to 2″Pipe and Grouted Pipe Delivery Void.

Figure 7-8 shows formwork for SCC Pours, W2 bent cap of the west end of Self-Anchored Suspension Bridge. Blue Hoses are intake hoses providing water to the internal cooling system for mass concrete and Red Hoses are for the warmer return water.

Figure 7-8. Forms for SCC Placement with Water-cooling Tubes.

The 10,000 psi concrete used for constructing the bent cap beams at the ends of the SAS structure required HSC. Some of the pours were congested enough with rebar to warrant the use of SCC. The cementitious content for the SCC was limited to 800 pounds per cubic yard for both conventional and SCC as discussed in the above section on HSC. The only differences between the conventional and SCC mixes were the fine to coarse aggregate ratio, the amount of High-Range Water Reducer used in each mix, and the addition of VMA in the SCC mix. Cementitious content and water, at least by design, were exactly the same. Upon analysis of the compressive strength cylinders taken from the project, it is notable the SCC had consistently higher strengths. This can be seen in Figure 7-9. The points are the average tests of two separate conventional concrete pours and average tests from four separate SCC pours. This graph does not include the data from the pour shown in Figure 7-4.

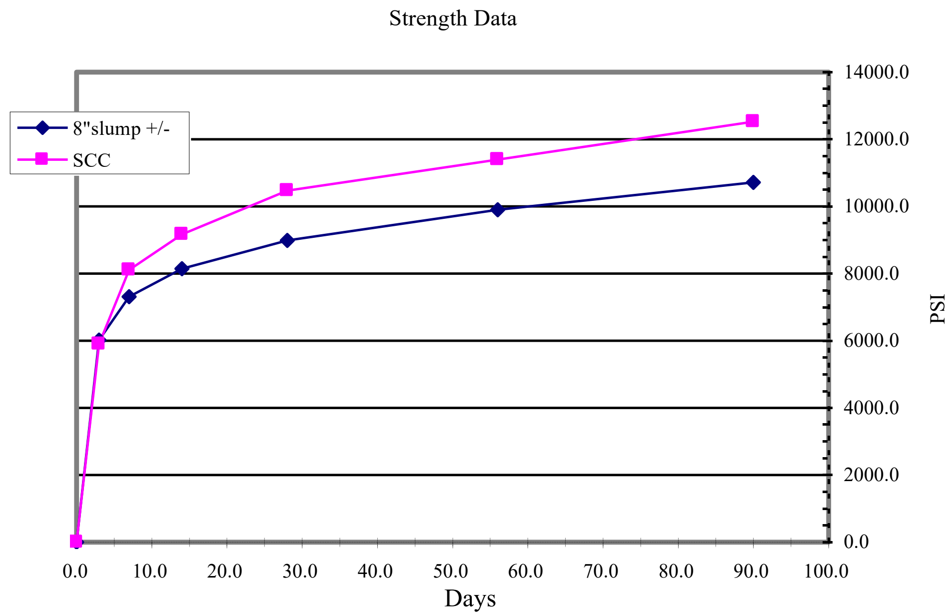

Figure 7-9. Nifty Graph Comparing Strength of SCC and High Slump Concrete.

A SCC early strength mix was used in the closure pours that connected the west-end bent cap beam (W2) to the orthotropic steel boxes to the east making up the superstructure of the Self-Anchored Suspension Bridge. Early strengths were desired to avoid the structure going through a thermal movement cycle at the gap to be filled with reinforced concrete between the concrete bent and steel boxes before adequate strength was obtained in the concrete. Type III cement was used with a Type C chemical accelerator admixture. The slump flow rates ranged from 25 to 29. The first closure pour tying the east-bound orthotropic box structures to the W2 bent was completed about 10 p.m. in one evening. By 5 a.m. the following morning concrete strengths exceeded 3,000 psi. Previous monitoring showed no thermal movement between about 9 p.m. in the evening and 5 a.m. in the morning. The second closure pour done 2 weeks later tying the west bound orthotropic boxes to the W2 bent had similar results though achieving a lower strength of about 1,800 psi by 5 a.m. because the accelerator was cut back to facilitate placement; 1,200 psi was the 5 a.m. target.

Prequalification testing was required to verify flow and segregation characteristics meet specifi ations. A mockup flow test is also included in the current contract special provisions for SCC use. Trial batch test reports document slump flo , flow rate, visual stability index, J-Ring passing ability, column segregation, bleeding, compressive strength and minimum compressive strength.

- Slump flow – Slump flow measures the consistency of fresh SCC. The test measures the spread of a SCC sample after release from a standard slump mold. Spread is the average of flow in two orthogonal directions as shown in Figure 7-10.

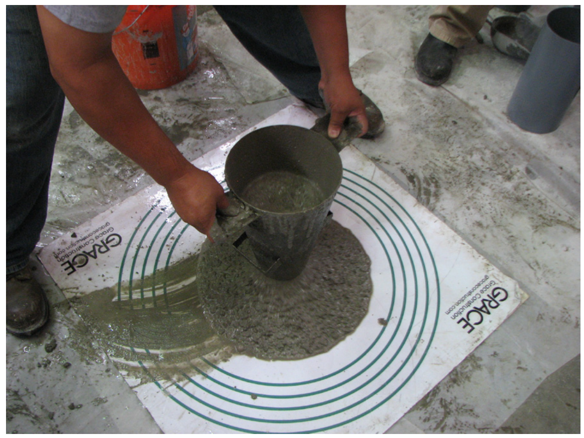

Figure 7-10. Slump Flow Measurements.[2]

- Flow rate – Referred to as T_{50}, flow rate measures the viscosity of the mix. Higher viscosities are indicated by longer times to complete the flow test. High viscosities are associated with “sticky” or “cohesive” mixes. The T_{50} time is the time required for the SCC sample to spread to 20 inches. The use of 50 in the T_{50} symbol represents 500 mm, approximately 20 inches.
- The Visual Stability Index (VSI) is a relative viscosity rating. After the flow test has been performed a visual check is made for segregation and bleeding. A halo is a cement paste ring around the flow that has segregated from aggregate. Aggregate may also segregate as a pile at the center of the flow. The visual stability ratings run from zero to three and are described in Table 7-3.

[2] http://www.hpcbridgeviews.com/i50/Article4.asp (visited 7/20/10)

Table 7-3. VSI Ratings.

Rating	Description
0	Highly Stable – No segregation, no bleeding
1	Stable – No segregation, slight bleeding (surface sheen)
2	Unstable – Mortar halo at exterior ring (< 0.5 inch). There may be an aggregate accumulation at the center of the flow.
3	Highly Unstable – Large mortar halo (>0.5 inch). There may be a large aggregate accumulation at the center of the flow.

- J-Ring Passing Ability – The J-Ring Apparatus, shown in Figure 7-11, is a solid ring supported by 16 dowels. Each dowel diameter is 5/8 inch and the dowels are spaced approximately 2.36 inches apart along the circumference of a 12-inch diameter circle. The test assesses the passing ability of a SCC mix as it flows through the dowel array. Similar to the slump flow test, material is released from a slump test mold at the center of the J-ring apparatus. The flow measurement is the average of two orthogonal measurements of the slump flow. Acceptable results are within one inch of the slump flow tests.

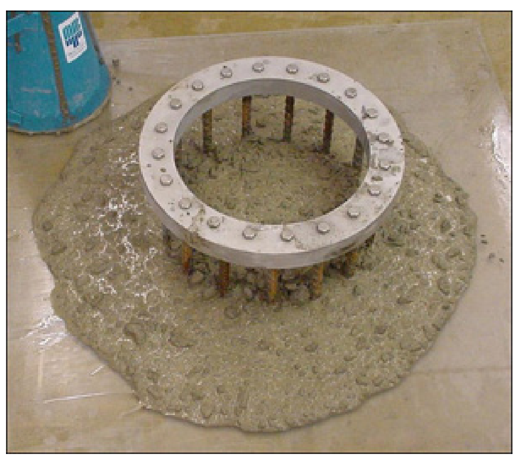

Figure 7-11. J-Ring Apparatus.[3]

[3] http://www.hpcbridgeviews.com/i50/Article4.asp (visited 7/20/10)

- Column Segregation – The column segregation test, shown in Figure 7-12, verifies that coarse aggregate does not segregate in a column placement. After filling a cylindrical mold without tamping or vibration, the mold is separated into upper and lower thirds. The material from each third is washed on a No. 4 sieve and the masses are compared to determine the percentage of segregation.

Figure 7-12. Column Segregation Apparatus.[4]

- Bleeding – ASTM C232 testing determines the percent of bleed water by comparing the mass of water in a specimen to the mass of bleed water drawn off.
- Strength – The strength test is a standard California 521 Test for five cylinders.
- Minimum Strength – The minimum strength test, also California 521, verifies all cylinders exceed minimum specified strength.

After passing all prequalification testing including the mock up QA/QC testing can be limited to slump flow and the typical conventional concrete tests.

[4] http://www.hpcbridgeviews.com/i50/Article4.asp (visited 7/20/10)

Mass Concrete

When measures beyond those typical for concrete construction need to be taken precisely to keep a concrete element from exceeding 160°F, that element is determined by Caltrans to be Mass Concrete. The measures can be applied to the concrete mix design, or to the in-place hardening concrete, or both. The measures taken to prevent the hardening concrete from exceeding 160°F are necessary for the durability of the structural element. It is most efficient from a contractual point of view to detect structural elements having a potential to exceed 160°F before a contract goes out for competitive bid. However, sometimes this potential may be missed during the design phase. If this happens, it is imperative to the durability of the structure to address the issue by CCO; as part of the design and evaluation process for temperature controls, the decision should include cost analysis data for the potential solutions.

Both the practices used to determine in advance potential mass concrete and the measures taken have changed through the years. Besides a basic understanding of the physical characteristics that result in mass concrete, a review of the changes regarding mass concrete over time is useful for grasping the substance of the theory and practice of mass concrete. The better the assimilation of the substance of the theories and practices of the measures taken over the years, the better the structure engineer is able assess solutions to the practical problems that sometimes arise during construction. This is true even when mass concrete is identified and addressed in the contract document.

The Basics of Mass Concrete

As the cementitious materials in concrete hydrate, heat is given off because hydration is an exothermic reaction. The amount of heat produced is proportional to the quantity of concrete placed. As concrete element size increases, the amount of heat produced will also increase. The temperature of the element depends on the rate of heat dissipation to the environment compared to the rate of heat generation. Dissipation depends on the size and geometry of the element while the generation depends on the cementitious material.

High temperatures can affect concrete durability by inducing stresses or chemically altering the cement hydration process. Temperature induced stresses are a result of volume changes that can result in cracking. Delayed Ettringite Formation (DEF) is the temperature induced alteration of the hydration process. It is generally recognized that up to a curing temperature of 160°F, DEF will not occur; while the further above 160°F, the greater the probability of occurrence.

Ettringite is a naturally occurring mineral formed in hydrated Portland cement along with calcium silica hydrates. When high heat occurs, ettringite may not form. However, after the heat caused by hydration subsides, ettringite formation may resume, resulting in an expansive gel within the hardened concrete. This can result in a material related stress similar to Alkali Silica Reaction deterioration.

High peak temperature can also lower compressive strength and related properties by restricting the hydration process due to self-desiccation, a condition where hydration cannot continue because of a lack of available water. Therefore attention to temperature during hardening is all-important.

ACI Committee 207 was established in 1930 for the development of theory and practice of mass concrete for the construction of large dams.   Since then, mass concrete theory has become applicable to much smaller concrete elements because of high performance concrete (HPC) incorporating higher cementitious contents to achieve properties like higher strength, self-compaction, and lower permeability. In 1987 ACI Committee 207.1 changed its title from “Mass Concrete for Dams and Other Massive Structures” to simply “Mass Concrete.”ACI 116R-00 defines mass concrete 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.”

Though Caltrans is interested in preventing cracking due to mass concrete, it is the potential long-term material related distress that triggers mass concrete measures. Therefore the peak temperature potential of 160°F is used by Caltrans to define concrete as Mass Concrete and the need for thermal control measures. Besides, if peak temperatures are controlled the detrimental effect of differential temperature is also mitigated.

Thermal Control Measures

The measures used to control concrete temperature can be either active or passive.

Passive measures control temperature through actions taken before placement. Passive controls may include limiting the amount of cementitious material, using supplementary cementitious materials that generate heat at a lower rate, placing elements in several lifts or segments thus limiting the size of each pour by addition of cold joints, lowering the concrete temperature at placement by adding ice as part of the mix water, chilling stockpiled ingredients prior to batching, or cooling the batched concrete with liquid nitrogen injections. Figure 7-13 consists of two photos of the nitrogen injection process. Figure 7-14 is a photo of ice being added in the batching process.

Figure 7-13. Liquid Nitrogen Coolant Injection.

Figure 7-14. Batching Concrete with Ice.

Active methods control concrete temperatures after placement and during or after hardening. Active controls include the installation of surface insulation, the use of cooling pipes that contain a cooling liquid and act as a heat exchanger within the concrete element or using water curing for evaporative cooling on the surfaces. Figure 7-15 is a typical view of cooling pipe installation.

Figure 7-15. Cooling Pipes Tied to Reinforcing Steel.

Identifying Mass Concrete

The ACI definition does not characterize mass concrete in terms of a specific dimension. Caltrans, as well as other agencies, uses minimum dimensions to identify mass concrete for the purpose of determining when thermal controls are needed. Having a universal standard minimum dimension of an element to identify what would be mass concrete is adequate for large concrete placements with the traditional specified concrete compressive strengths that are no more than 5,000 psi. However with use of higher-strength HPC concretes, minimum dimensions identifying mass concrete have become less predictable.

The dimensions identifying mass concrete in Caltrans bridges changed through the years while the measures used to control temperatures remained similar until the recent increase in the use of HPC. Around 1978-1980, specifications for Potato Slough Bridge and Dumbarton Bridge both identified mass concrete as any structural concrete element with a minimum dimension of 4 feet. By 1992 the minimum dimension identifying mass concrete in structural elements was increased to 7 feet. In 1995 as a soft conversion to metric the minimum dimension was changed 2 meters, or 6.6 feet. With the return to English units, the minimum dimension is 7 feet. This minimum 7-ft dimension has been shown to be adequate for identifying mass concrete in advance when concrete is not high strength concrete.

In October 1999, concrete placed in a 6-ft diameter steel shell shaft rising to the surface of the Carquinez Straits was monitored for temperature; this new pile was part of a seismic retrofit for the 1962 Benicia-Martinez Bridge. The concrete in the pile had no thermal control and was instrumented with thermocouples. The peak temperature occurred along its central axis at 22 feet down. Six feet down from the surface, the temperature reached no more than 122°F. Temperature differentials between concrete near the steel shell that was submerged in the cold bay water and concrete at the center axis of the pile were less than 14°F.

During construction of the Spanish Creek Arch Bridge in District 2, peak temperature measurements on a column with a cross dimension of 4 x 8 ft demonstrated the difficulty in relying totally on the dimensional method to identify mass concrete. The peak temperature was 161°F at 24 hours. This concrete was high strength concrete at a specified 42-day 6,000 psi with also an air requirement since the bridge is in a freeze thaw zone. The mix had the maximum 800 lb/yd^3 of cementitious, which was Type IP cement. This particular Type IP cement utilized a pozzolanic material that was 84% reactive material with a Blaine fineness of 680 m^2/kg; this made the heat rate similar to, if not higher, than Portland cement. In addition, the Blaine fineness of the cement was a relatively high 480 m^2/kg when compared to a more typical being around 400 m^2/kg.

The combination of hot cementitious material, long delivery distance for the concrete, summer temperatures and borderline geometry resulted in high peak temperatures in the curing element. The issue of mass concrete was contemplated during design and based upon the assumptions made by the design team. It was decided that mass concrete would only be an issue at the footing blocks of the arches. Structure Construction staff were diligent in verifying design assumptions and when the temperature was measured on the relatively small column the project staff were able to get a thermal control plan in place for the 8 x 9 ft arch rib.

Figure 7-16 shows the temperature rise for two 5-ft concrete cubes with the only exception to identical mix designs being the Blaine fineness of the cement. As shown in the figure, the temperature of the cube with the finer cement, (380 m^2/kg), rose 10°F higher than the cube with the coarser cement (300 m^2/kg).

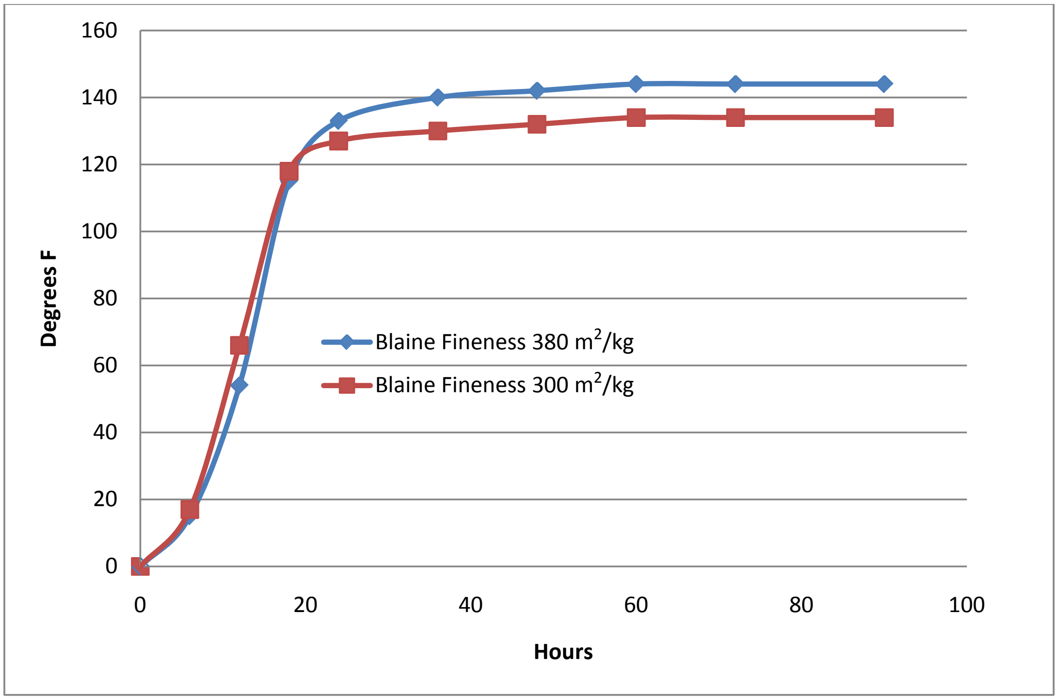

Figure 7-16. Temperature Variation Due to Blaine Fineness.

The column was placed in summer. Therefore with a mix having heat generation characteristics nearly as high as possible within standard specifications and placed during summer, the column did not exceed the 160°F limit used to define mass concrete. Though this is closer to the limit than desirable, the mix design is as hot a mix as can be expected and is high strength. (Note: precautions are called for in this situation but the measures are similar to those taken for concrete placed under hot weather construction discussed in Chapter 5.) Therefore for identifying potential mass concrete in advance, 7 ft serves quite well unless HPC is to be used.

Passive Control by Prescriptive Specifications, Caltrans Practice Prior to the 21st Century

Before 1999, all mass concrete temperatures in California bridges were easily controlled with passive methods specified in prescriptive specifications because mass concrete was generally limited to large footings and columns that had traditional bridge concrete compressive strength requirements seldom more than 4,000 psi. Prescriptive specifications for passive thermal controls were easily employed by special provisions. Initial placement temperature was limited to 65°F when the cementitious material was no more than 564 pounds per cubic yard with the pozzolan being 15% of the cementitious material by weight. For each additional 10 pounds of cement or 20 pounds of pozzolan above the 564 pounds cementitious content, the initial maximum temperature was reduced 1°F. Also, the Type II cement's combined tricalcium silicate and tricalcium aluminate content was limited. Cementitious content was changed to 590 lb/yd3 when Caltrans went to SI units in 1995. The 590 lb/yd3 was due to a soft conversion of class concrete which in this class became 350 kg/m3. Fly ash after 1997 was 25% as required in most structural concrete offsetting the slight increase in cement. The prescriptive specifications were practical for both Caltrans and the contractor.

Field experience showed that these prescriptive specifications of passive measures and having no contractual temperature performance requirements for the in-place concrete were adequate. During reconstruction of the Cypress Bridge structures on Route 880 in Oakland, California, after the Loma Prieta Earthquake, the contract used the prescriptive specifications detailed above with 7 ft as the minimum dimension identifying mass concrete. Six large footings and one column built between 1994 and 1996 were monitored for temperature to check the assumptions that the prescriptive specifications resulted in acceptable control of concrete temperature. The footings were as large as 30 x 45 x 10 ft and the cross section of a column was 9 x 9 ft. The mixture proportions required 564 m^3 of cementitious materials with 15% fly ash for the column and the first footings. Peak temperatures were no higher than 144°F, an acceptable temperature limit preventing DEF. In 1996, fly ash percentages were increased to 30% and then 40% in an attempt to reduce temperature differential within the element to 36°F. The last footing was 10 feet thick and the concrete was 40% fly ash, reducing the peak temperature to 126°F. The additional fly ash reduced the differential to the 36°F target, which Caltrans has since found to be conservative for large elements.

Following the Cyprus project from July to October 1999, five large footings for the 580/680 Interchange bridges in Dublin, southeast of Oakland, California, were monitored for temperatures. The footings were 26 to 36 feet wide, 35 to 40 feet long, and 10 to 12 feet thick. Only the specified passive temperature controls were used. All mixes had 590 lb/yd^3 of cementitious material with either 25 or 35% fly ash. The peak temperatures ranged between 135°F and 140°F, and temperature differentials for the 35% fly ash mixes were near the 36°F limit though somewhat above it. The actual field data again demonstrated that the prescriptive specified passive controls were appropriate.

Performance Based Requirements

Mass Concrete Specifications for the replacement Carquinez Bridge (Alfred Zampa Bridge on Route 80 northeast of Oakland, awarded in 2000 and completed in 2003) were performance based. The contractor was to determine the measures taken to keep concrete temperatures within specified limits. Any element with a least dimension that exceeded 2 meters was limited to a peak temperature to 158°F and temperature differential of 36°F. The tower footing concrete and the massive anchor blocks for the cables were mass concrete and were monitored by measuring temperatures in the concrete via thermocouples connected to data loggers.

The contractor used the Schmidt Analysis Model for thermal predictions and temperature controls. The analysis was reasonably accurate resulting in adequate temperature controls. Cooling pipes were considered by the contractor, but not chosen. However, the contractor chose to use the active measure of applying, maintaining and monitoring thermal insulation blankets to control differential temperature between the center of mass and concrete near the exterior surfaces, and the passive measure of limiting the cement content and increasing fly ash to 35%. Mass concrete segments for the anchor blocks at Carquinez measured approximately 50×20×14 ft. Approximately 40 such segments were cast to produce the 4 anchor blocks, 2 on either side of the bridge. The footing for each of the 4 tower legs that made up the two towers was 59×72×16 ft, with each tower leg joined by a 49×23×16 ft tie beam so that each tower had 1 continuous footing. The insulated concrete, with the required f'_c no more than 4,000 psi, did not exceed the maximum specified peak and differential temperature. Figure 7-17 is an anchor block. The grout tubes extend 58 ft into the blocks. Figure 7-18 shows the footing, or pile cap, form work for the mass concrete. Figure 7-19 is a photo of the Zampa south anchor blocks and towers rising above the pile caps.

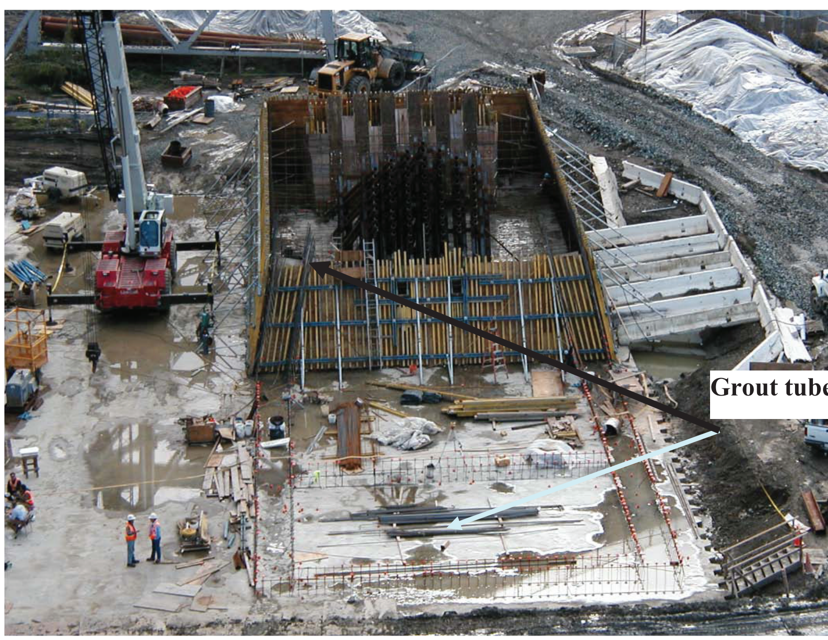

Figure 7-17. Anchor Block, Zampa Bridge.

Figure 7-18. Footing or Pile Cap Prior to Placement of Mass Concrete.

Figure 7-19. Zampa Bridge Anchor Blocks and Towers During Construction.

The Use of Active Thermal Control by Caltrans

With the use of High Strength Concrete, the combination of passive methods with thermal blankets was inadequate for large *massive elements*. During the design of the new Noyo Bridge, the new Benicia/Martinez Bridge, and the new east spans of the SFOBB, internal cooling pipes were discussed for the larger higher strengths concrete elements. The pier tables for the Skyway portion of the new SFOBB east spans, were not only larger than the 580/680 footings, but were to be cast with high strength concrete in the range of 8,000 psi. The large foundation blocks and anchors for the west end of the Self-Anchored Suspension (SAS) portion of the new SFOBB were also of special concern. The planned blocks at Bent W2 where the SAS meets the Yerba Buena Island transition structures were 64 × 64 × 33 ft with corrosion resistant concrete that reached compressive strength of 9,000 psi. The new Benicia and Noyo River bridges also anticipated large elements with higher strength concrete than what was typical though not as high as the 8,000 psi design.

There were concerns that existing prescriptive passive specifications would not work for the anticipated mass concrete envisioned for the three bridges and that active controls, particularly cooling pipes, would be necessary. The unlimited combinations of pipe size and arrangement, water temperature and flow, and possible combination with other passive controls led to the use of a performance specification. Except as noted below, these first performance specifications used 149°F as the limit in order to have some built-in safety factor. However, it was also concluded that practical experience was needed and a cooling pipe thermal control system should be done on some existing project.

To demonstrate to contractors that cooling pipes were a viable option, Caltrans designed a heat exchanger made of steel pipes for one of the large footings at the 580/680 Interchange project. In November 2000, concrete for a 580/680 Interchange footing was changed to a high-strength concrete. This concrete generated significantly more heat than the mixtures used on the other footings at this site that were of a similar size and had previously been monitored for temperature. Active temperature control in the 38 × 38 × 12 ft footing consisted of 1 inch diameter steel cooling pipes that were installed at the same time as the reinforcing steel and configured into 3 square coils with pipes spaced approximately 3 ft apart vertically and horizontally. A 50-ton chiller was used to cool water circulating through the pipes.

With the use of cooling pipes peak temperature of the footing with high strength concrete was comparable to the other normal strength concrete footings on the project previously monitored for temperature. The normal strength concrete had 590 lb/yd^3 of cementitious material, 35% of it fly ash while the high strength had 800 lb/yd^3 cementitious, 20% fly ash 5% metakaolin. This comparable peak temperature is shown in Figure 7-20. Figure 7-21, a time vs. temperature graph comparing the high strength to the normal strength 6 × 12 inch cylinder both cured under identical conditions, shows the difference in temperature generation of the 2 concretes. These two graphs demonstrate the effectiveness of the cooling pipes. A report was written and included with the bid documents on other projects where active controls were expected.

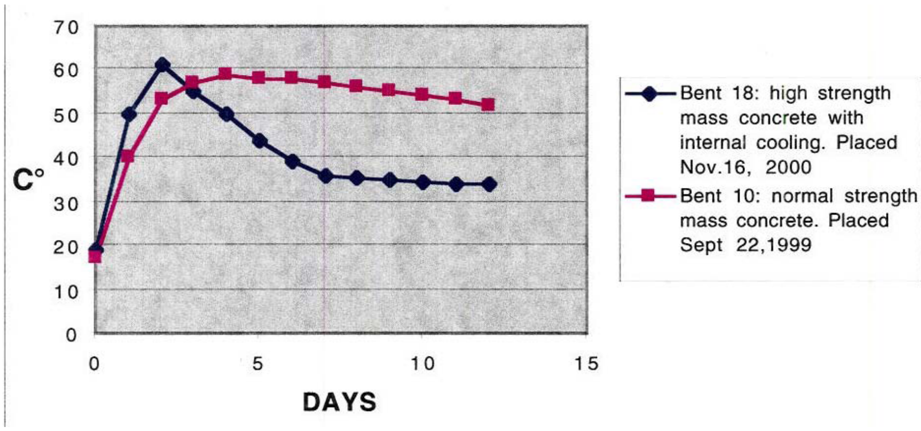

Figure 7-20. 580/680 Footing Concrete Temperature vs. Time.

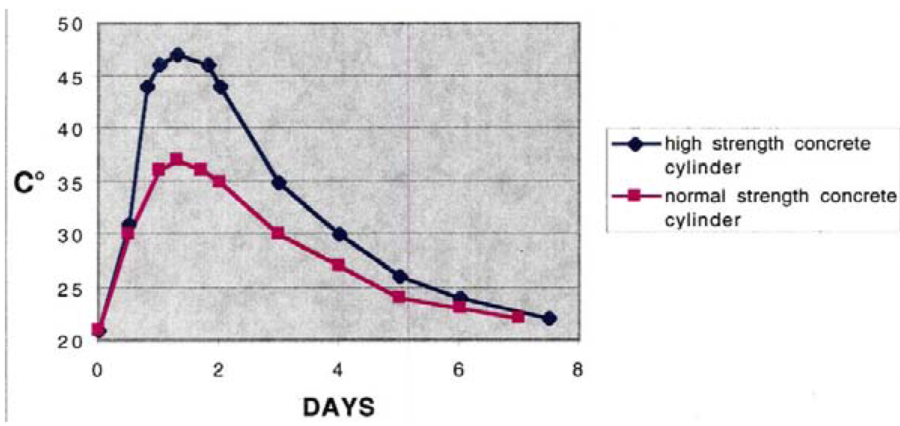

Figure 7-21. 6 x 12 Inch Cylinders Cured Under Same Condition.

Noyo River Bridge

The first bridge with Caltrans specifications requiring an internal cooling system was the new Noyo River Bridge in Fort Bragg, located 140 miles north of San Francisco. The specified concrete compressive strength ranged from 4,300 to 5,700 psi with low permeability required for elements in or near brackish water. Bridge construction began in May 2002 and was completed in August 2005. The mass concrete elements were 39 × 108 × 8.2 ft footings, 9.8 ft diameter columns, 20 × 72 × 12 ft bent caps, and 11 × 85 × 7.2 ft abutments.

The contract special provisions required a thermal control plan that limited peak temperature to 149°F and differential temperature between any two points within the placement was limited to 36°F. Cooling pipes were required in the first placements of the footings, columns, and bent caps. However, if the contractor proposed a plan that did not use the cooling pipes and proved that the plan was effective during the first placements where the cooling pipes were in place only as a back-up system, the cooling pipes would not be required in subsequent placements.

The contractor chose to use 1 inch diameter steel cooling pipes and chilled water from a storage tank as part of the thermal control plan even at the abutments where it was not required in the first placement. The Caltrans Design Engineer for this project noted the ease of fabrication and operation of the system to control temperatures.

San Francisco-Oakland Bay Bridge East Spans

The seismically vulnerable east span of the San Francisco-Oakland Bay Bridge connecting the Yerba Buena Island tunnel to Oakland is being replaced with a new 2.2 mile long bridge. Two of the major structures making up the bridge are the 1.5-mile Skyway and the 2,000 ft self-anchored suspension (SAS) structure. The Skyway, with spans up to 525 ft and box girders up to 30 ft deep and 82 ft wide, was constructed using a pre-cast concrete segment balanced cantilever method. The large bent caps had f'_c of 8,000 psi used cooling pipes to control temperature. All concrete for the Skyway had a specified peak temperature limit of 149°F. The SAS is a single tower signature structure architecturally designed to complement the west span 4-tower suspension bridge linking the island to San Francisco. The span east of the single tower will be 1,263 ft, while the span to the west will be 784 ft. The west end concrete piles, foundation/anchor, and piers were completed in 2004 as a separate contract. The last Skyway segment was placed in December 2006. The 2,000 cubic yards of concrete for the bent cap at the east end (E2) of SAS was placed in 1 continuous pour, while the last of the 5 concrete pours making up the 80,000 cubic yards concrete bent cap on the west end (W2) occurred in February 2009.

Of particular interest are the anchor blocks at W2 at the west end of the structure where the SAS meets the Yerba Buena Island Transition Structure. The two anchor blocks each measuring 63 × 63 × 10 feet help balance the forces in the unequal spans of the signature SAS structure. The maximum peak temperature for the W2 footings/anchors was specified at 122°F to ensure the large mass of concrete would stay intimately in contact with the walls of the excavated rock formation. Low permeability requirement in the concrete for corrosion control resulted in an actual compressive strength that was over 9,000 psi. Each block was poured in one continuous work shift. Liquid Nitrogen as passive control and cooling pipes as active control was used. For one block cooling water was taken directly from the bay while on the other water was chilled water from a storage tank.

The bent caps at W2 on the west end and at the east end of the structure at E2 specified 8,000 psi concrete. (The SAS cable will be looped around the west bent cap through deviation saddles and anchored to the E2 bent cap.) The mass concrete thermal control plan in both bent caps consisted of pre-cooling concrete with liquid nitrogen (passive) and cooling during hydration with active cooling pipes to comply with the temperature limit 149°F.

For these large elements requiring high strength concrete, cooling pipes showed to be a most effective method of controlling temperatures to meet the performance-based specifications. Scaled down mock-ups were required rather than mandatory cooling pipes in the first placement, to validate the contractor's thermal control plans. Figures 7-22 and 7-23 are initial and final photos of the W2 anchor reinforcement and cooling pipe placement. Figure 7-24 is a typical 10 x 10 x 10 ft mock-up cube. Figure 7-25 is a chart of the mock-up temperature curve.

Figure 7-22. Start of Reinforcement / Cooling Pipe Layout W2 Anchor.

Figure 7-23. Complete PVC Cooling Pipe and Reinforcement Layout W2 Anchor.

Figure 7-24. Footing Mock-up 10-ft Cube.

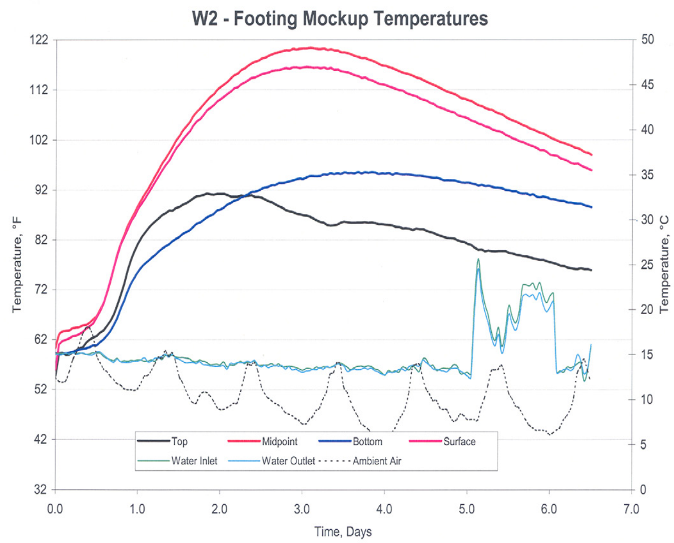

Figure 7-25. W2 Footing Mock-up Temperatures.

Benicia-Martinez Bridge Substructure and Foundation

The new 1.4-mile long Benicia-Martinez Bridge on Route 680 about 15 miles northeast of Oakland, California, has spans up to 660 ft. It was designed as a lifeline structure that will be serviceable after a major seismic event. The portion over water was constructed from 335 cast-in-place single-cell box segments made of high-strength, lightweight concrete. The segments were cantilevered from 11 piers, 10 of which were marine piers rising out of the strait.

The large 8.2 to 9.1 ft diameter piles and massive piers were made of normal weight HPC with a maximum water-cementitious material ratio of 0.40 to limit the permeability of the concrete and protect the reinforcing steel. The specified compressive strength was 5,000 psi for the pier footings, pile caps, and pier walls/columns and 6,500 psi for the pier tables.

There were over 200 normal-weight mass concrete pours. Most, but not all of the pours were cooled with 3/4-inch diameter PVC cooling pipes spaced 2 to 5 ft apart. Overall, thermal control in the substructure and foundations were adequate even though a few placements did exceed the 149°F specified limit, with three placements exceeding 160°F. Cooling pipes were effective, if not indispensable, for these normal-weight mass concrete elements.

Cast-in-Place Segmental Bridge Construction

In constructing a balanced cantilever segmental bridge, a cast-in-place segment, measuring 10 × 20 ft is placed, cured, post tensioned, and then used to anchor and support the traveling forms for the next segment. This is done until the span is constructed. Such construction lends itself to a contractor having concrete achieve a required stressing strength as soon as possible. This leads to a tendency to design high early strength concrete mixes. FHWA guidelines even include steam curing. This tendency if unchecked will lead to concrete mixes that can result in high temperatures during its curing. This turned out to be a particular issue with the lightweight high strength concrete for the Benicia-Martinez CIP segmental bridge. For the Caltrans segmental bridges at Confusion Hill in District 1 and the Devil's Slide Bridge in District 4 that followed, the specifications were written with the idea of limiting concretes that produce enough heat to be Mass Concrete.

Lightweight Concrete Segments & Mass Concrete in Elements with Dimensions as Small as 10 inches

The new Benicia-Martinez Bridge construction brought to light the very special thermal properties of high-strength lightweight concrete. With a high cementitious material content of 980 pounds per cubic yard coupled with a low fly ash percentage of 5%, 10,000 psi lightweight concrete used in the segments generated more heat than the normal weight concrete used in the substructure. Based on past experience, it was not anticipated that the decks, which were as thin as 0.83 ft, or the 1.8 ft thick stems would require thermal control. Only soffits that were greater than 3 ft thick were anticipated to require thermal control. The standard 7 ft requirement for mass concrete was reduced to 3 ft in anticipation of the very high cement content which was induced by the economic incentive to produce a very high early strength in the CIP segment so as to be able to stress and move the traveler forms as soon as possible. After the first placements and continued thermal analyses, however, it became apparent that the temperature rise would be an issue with elements much smaller than 3.3 ft, in fact, all of the high-strength lightweight concrete making up the box cell segments needed thermal control. The entire high-strength lightweight concrete was designated as mass concrete with thermal controls required that limited the peak temperature limit to 160°F. Cooling pipes were used throughout the structure, except in some deck and soffit sections near mid-span. The sections without cooling pipes did use liquid nitrogen as a passive control. The soffit manifold, shown in Figure 7-26, used blue hoses for inflow and red for outflow.

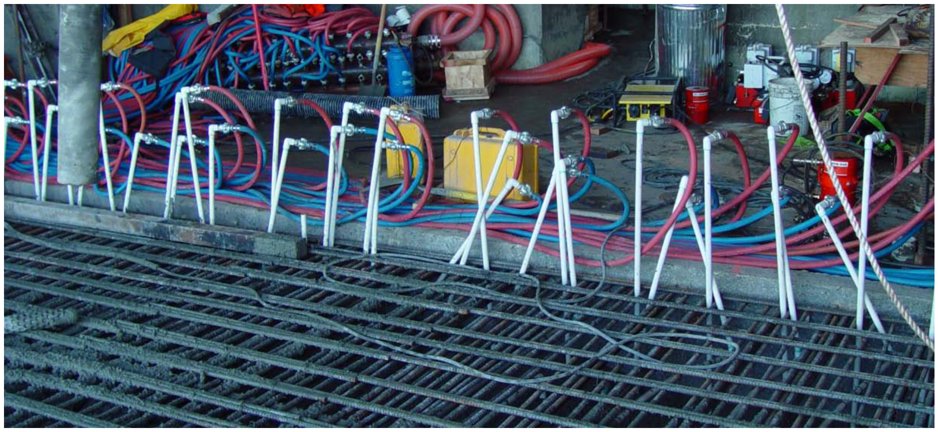

Figure 7-26. Soffit Cooling Pipe Manifold.

The first segment was cast December 31, 2004, and the last was cast October 23, 2006. Of the first 20 segments cast, 15 had elements exceeding 160°F, with four elements over 175°F. Additional temperature measurements at more locations within the segments were taken as construction proceeded. In February 2005, the only segment cast without any cooling pipes reached an alarming 196°F. This was the tenth segment cast, and it became apparent that all segments, though not all deck or soffit components, needed cooling pipes spaced per thermal analyses in addition to precooling to 45°F. After the active controls were implemented, a few element temperatures exceeded 160°F due to occasional problems with the pumping system, but most peaked below 131°F. Precooling with liquid nitrogen and post-cooling with water pumped from the strait flowing through PVC pipes performed quite well, and the cooling pipes appeared to be indispensable for this mix design.

The high strength lightweight concrete for the segments had particular mass concrete properties. The effect of the lightweight aggregate can be seen in the graph shown in Figure 7-27, depicting the temperature profiles of two 3.3 ft concrete cubes. The only difference between the cubes is that lightweight coarse aggregate in one block was substituted with normal weight aggregate on an equal volume basis. All other ingredients were the same (the water and air in the coarse aggregate are assumed to be part of the coarse aggregate).

Both cubes had the same amount of cementitious material and generated heat at the same rate in the same environments; but the cube with the lower mass rose to a signifi antly higher temperature. The variation in temperature occurs because specific heat is the heat capacity per unit mass of a substance. The concrete blocks had almost identical specific heats, but the lightweight concrete had a lower mass. The temperature rise varied inversely with the mass; a larger mass quantity will result in a lower temperature rise as there is more mass to absorb heat.

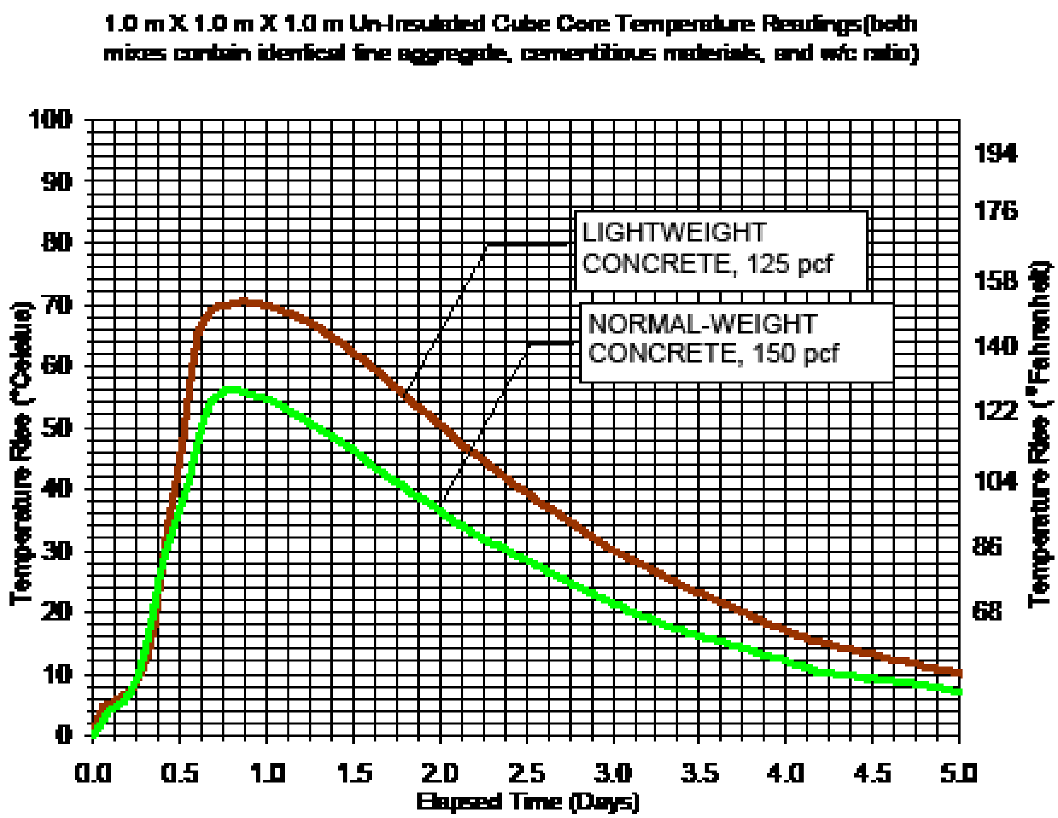

Figure 7-27. Un-insulated Concrete Temperature, Light Weight vs. Normal Weight.

The heat rise ratio and unit weight ratio of lightweight and normal weight concrete are similar. With a unit weight of 125 pcf for the lightweight and 150 pcf for the normal weight, the lightweight cube is 83% (125/150) of the mass of the normal weight cube of the same dimensions. The temperature rise for the lightweight was 128°F, from initial temperature of 66°F to peak of 194°F, while the temperature rise for the normal weight was 99°F, from initial of 66°F to peak of 165°F. The difference in mass resulted in the normal weight having 77% (99/128) of the temperature rise of the lightweight cube.

Since specific heat is the ratio of energy to temperature change for a given material per mass, the energy in the case of the two 3.3 ft cubes is the release of energy from the hydration process. The same cement content and the same type cement (given it was done the same day) are in each cube. This gives the same energy released due to the heat of hydration in each 3.3 ft cube. A similar experiment was performed on 5 ft insulated cubes, but in an effort to mitigate for different amounts of water absorption by aggregates that could affect heat conductivity to the external environment, insulation was placed around both cubes. As shown in Figure 7-28, the heat rise of the normal weight was 83% of the lightweight, exactly the percent of lightweight unit weight to the normal weight concrete. A lesser mass, though possibly having a slightly different specific heat and conductivity value due to water and air in voids in the lightweight aggregate, will have a higher temperature rise. This results in higher temperatures in the low-density concrete. This is an important lesson to keep in mind in pre-identifying mass concrete even though unit weight accounts for this in the equations to predict temperatures.

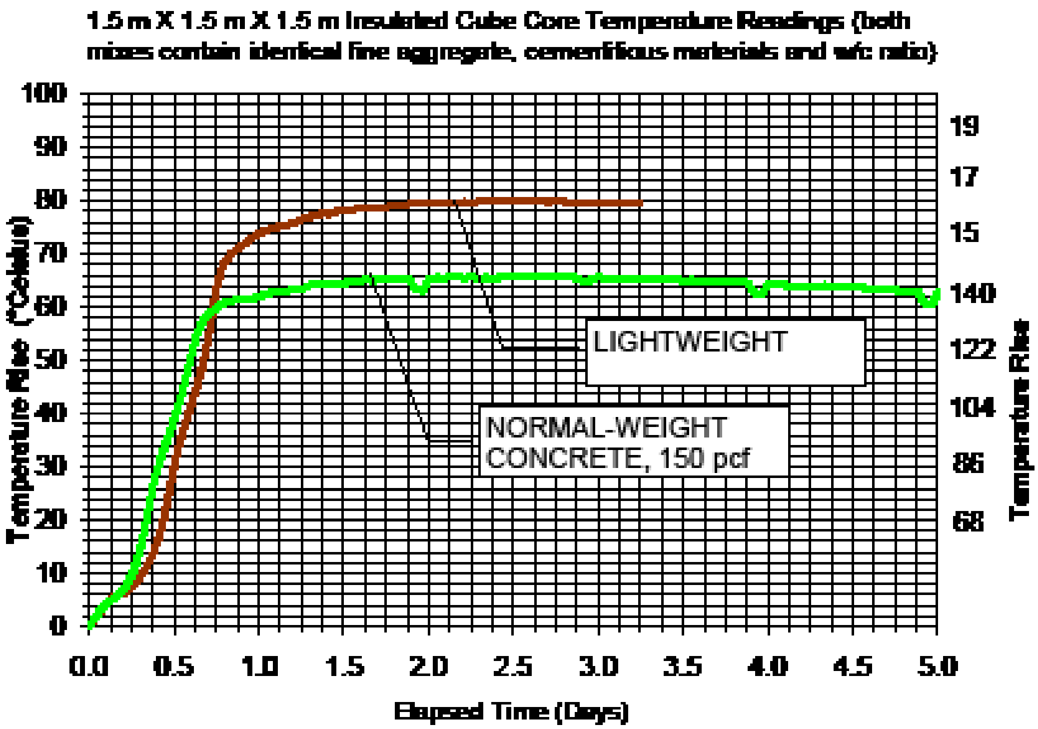

Figure 7-28. Insulated Concrete Temperature, Light Weight vs. Normal Weight.

Figures 7-29 through 7-32 show the thermal histories of the various segment elements. The frequency curve of peak temperatures of stems, soffits and decks are shown. Note the higher temperatures of the deck elements though these are the thinnest sections. This is because cooling pipes were not in many of the deck elements. In contrast cooling pipes were used in about half of the soffits and 318 out of 335 stem pairs. Only the first 17 of 335 stem pairs did not have cooling pipes.

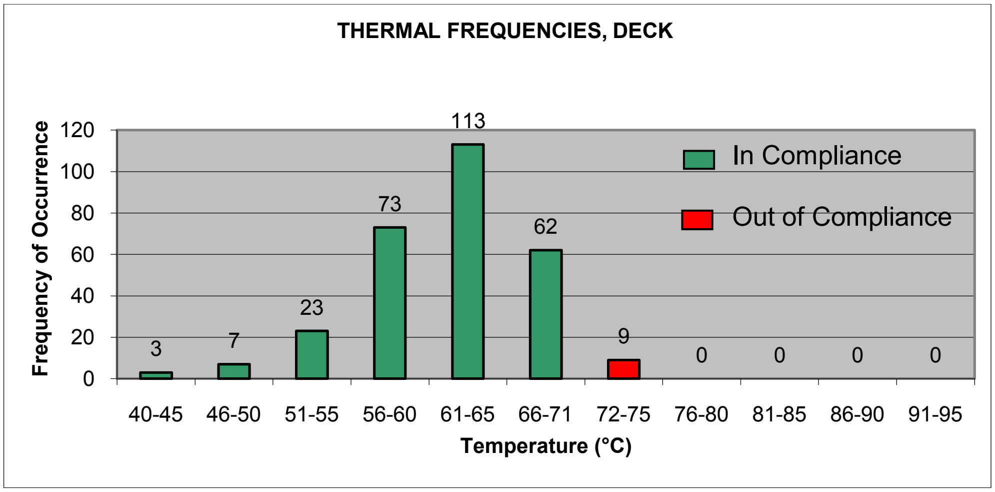

Figure 7-29. Deck Temperatures.

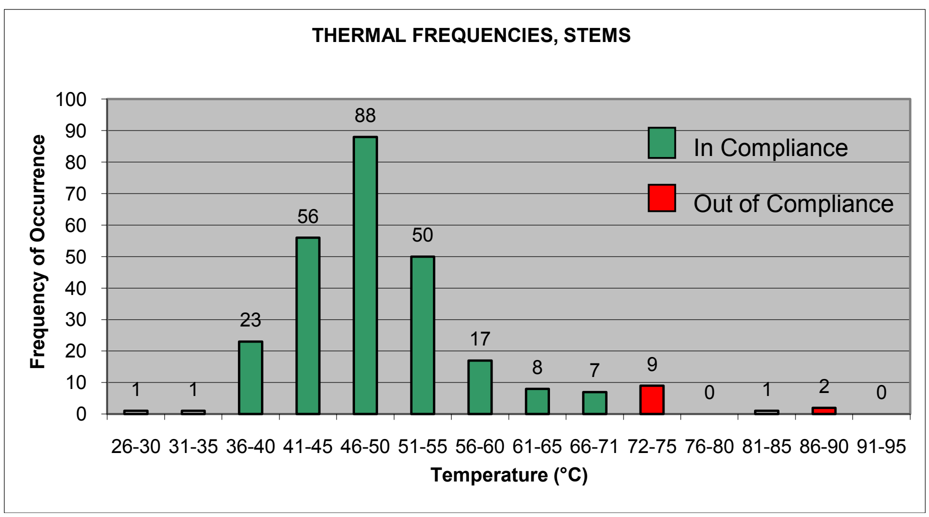

Figure 7-30. Stem Temperatures.

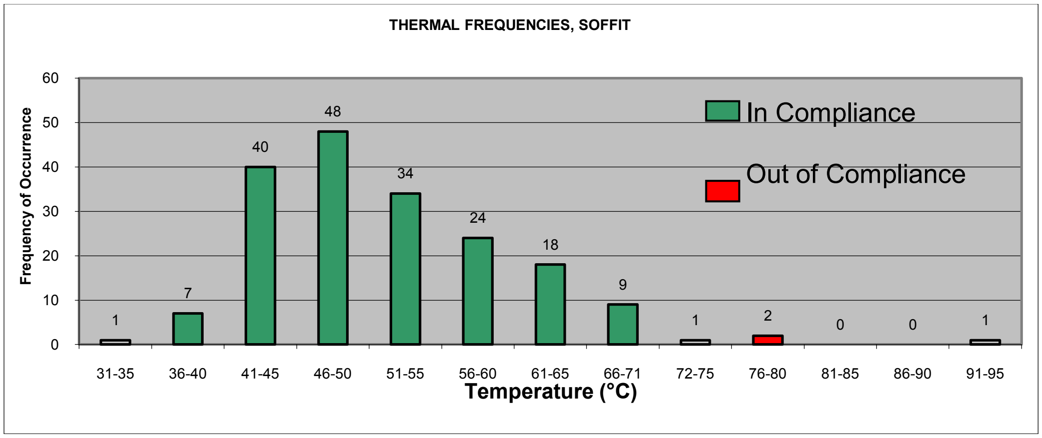

Figure 7-31. Soffit Temperatures.

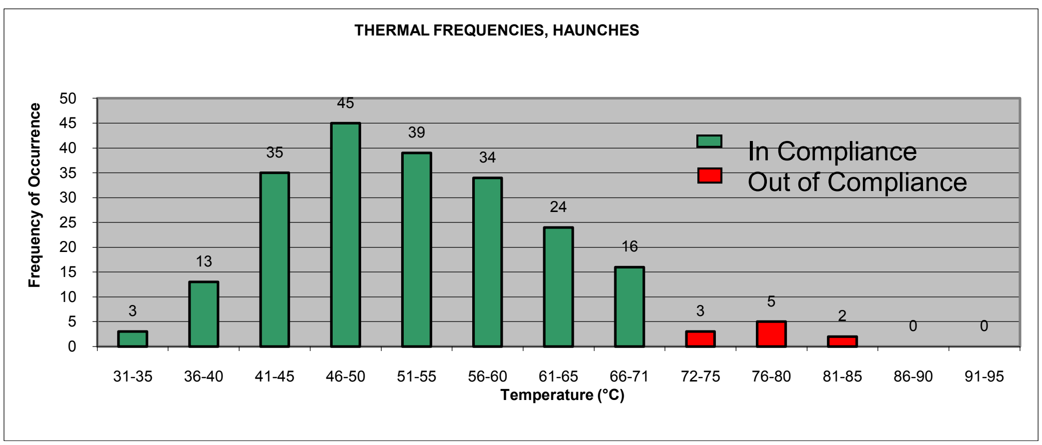

Figure 7-32. Haunches Temperatures.

Devil's Slide / Confusion Hill: A Laconic Approach to Mass Concrete Specifications

Though the issues with the cast-in-place segmental construction of the Benicia-Martinez Bridge were serious, it was recognized that the project's mix design had special thermal characteristics as previously discussed. It was deduced if fly ash was at the typical 25% and cementitious did not exceed Standard Specifications limits of 800 lb/yd^3, elements having a least dimension less than 4 ft would not have a temperature concern. This proved to be a correct assessment when this specification was enforced on the Confusion Hill and Devil's Slide Segmental Bridges. To reduce the incentive for a contractor to gain strength as quickly as possible so as to stress each segment as soon as possible in order to speed up the schedule, a 160°F maximum temperature was specified for any segment. Per specification temperature measurements would be taken in each segment at three locations chosen by the engineer. The maximum measured temperature taken during construction was 135°F for the Confusion Hill Bridge (Figure 7-33) and 142°F for the Devil's Slide Bridge (Figure 7-34). Concrete stressing strength requirements were met within 2 days and sometimes as early as 1 day. The 42-day compressive strength for the segments, which was at 6,100 psi, was exceeded without the need of thermal controls.

Figure 7-33. Confusion Hill Segmental Bridge.

Figure 7-34. Devil's Slide Segmental Bridge.

Acceptance of Active Controls And Return to Passive Control

Given the many factors influencing the characteristics of mass concrete, the methods used to cope with heat are a matter of economics as well as expertise. As contractors became more adept at using cooling pipes to control temperatures in large or high-performance concrete bridge elements, they were selecting this control method on many other Caltrans bridge projects, including some having traditional compressive strength requirements. This was not cost effective.

The Oakland Touchdown is a low-level, post-tensioned, cast-in-place concrete box girder bridge that connects Oakland to the Skyway. In a cost savings move when some large concrete elements were not identified as mass concrete in the bid document, Caltrans initiated change orders that replaced the contractors active thermal control system, which used internal cooling pipes, with a passive thermal control plan based on 50% fly ash concrete mixes. Mock–ups were done to ensure the anticipated outcome occurred in the actual placements. The 1,080 ft long structure has 7 spans over 6 piers. Under the piers are mass concrete pedestals, which sit on mass concrete pile caps that make up the footing. The pile caps vary in size having a footprint from 46 ft square to 52 × 72 ft. The mix had 337 lb/yd^3 of fly ash and 337 lb/yd^3 of Portland cement. The water to cementitious materials ratio was 0.4, the maximum permitted by the specifications for corrosion control. The strength requirement for the pedestal was 5,000 psi at 90 days while the pile cap concrete was 4,350 psi. The average measured strength for all the pedestals was 4,620 psi at 28 days and 5,720 psi at 56 days. Figure 7-35 below shows the 28-day and 56-day strengths for the pile caps. The lowest strengths occurred on samples stored during a 2 to 3-month period when temperature control of the curing room was malfunctioning. A few samples tested at 7 days had average strengths of about 3,000 psi. Concrete from 4 pedestals had average 90-day strengths of 6,225 psi. One 180-day test result was 6,830 psi.

Similar action taken for same reason on the Doyle Drive Bridge in San Francisco resulted in a substantial savings when compared to the contractor proposed thermal controls. For this project a combination of active and passive measures was used. The active system used a limited amount of cooling pipe in the columns (1 to 3 pipes down the approximate longitudinal central axis) along with a low heat mix using limited cement content and 30% fly ash. Savings were also obtained for the footings for the Yerba Buena Island Transition Structure by substituting passive measures for active. Caltrans took responsibility, thus removing risk to the contractor of high temperature issues because of the high degree of confidence in the selected thermal control measures, as was the history prior to the use of High Performance Concrete in mass concrete.

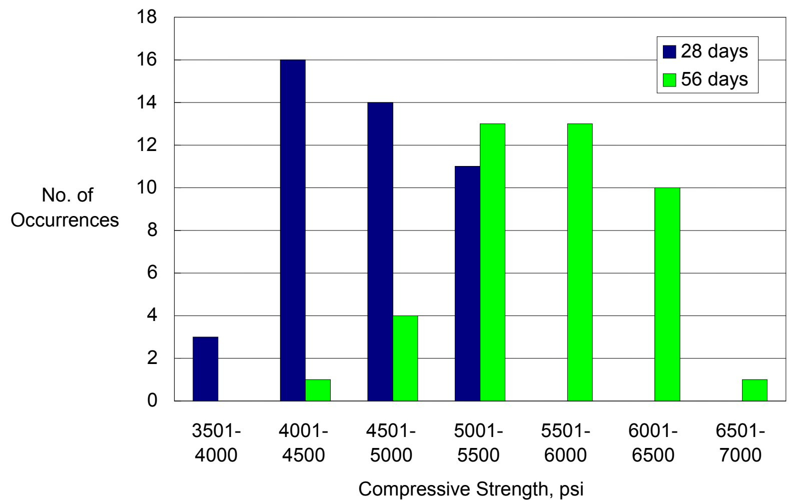

Figure 7-35. Pile Cap Concrete Oakland Touchdown.

Back to Prescriptive Passive Control Specifications & Mass Concrete for Concrete Piles

The data from the larger CIDH piles in the Benicia-Martinez Bridge suggested passive measures might be adequate for CIDH concrete. The pile that exceeded the temperature limits had high cementitious contents. Piles are cylindrical and thus having relatively high surface area to volume dissipate heat to the environment more readily than other common shapes. Also ground temperatures beyond 20 ft deep can be assumed to be no higher than 50°F. Since an assumption for ground temperature can be reliable, researchers at San Jose State University were able to use a finite element analysis, the Schmidt model and actual Q-Drum measurements (heat of hydration for various cementitious contents and combinations), to develop mass concrete specifications for CIDH pile concrete that kept concrete within temperature limits while curing. Subsequently, the accuracy of the calculations was verified against real data where the calculations reasonably matched the results.

The new standard specifications regarding mass concrete for CIDH piles are based on the information generated by the San Jose State research project shown in Figure 7-36.

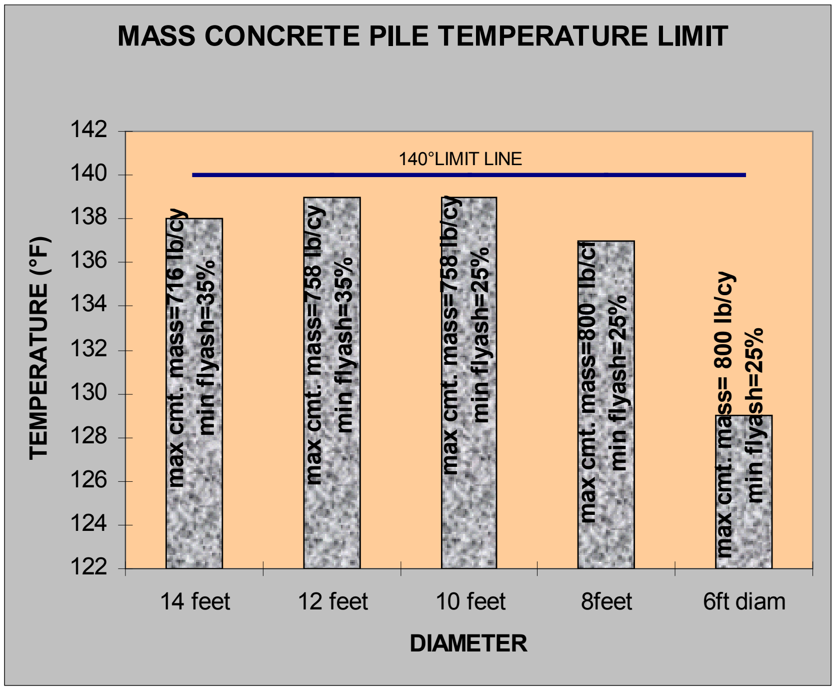

Figure 7-36. CIDH Pile Cementitious Material by Pile Diameter.

The complete set of data is shown in Table 7-4. The specifications require more restrictions on cementitious as the pile diameter increases. Beyond 14 ft in diameter, performance specifications for mass concrete require thermal control plans, mockups, temperature monitoring, and penalties for non-compliance. At the contractor's option, performance specifications may be used for any pile larger than 7 ft in diameter.

Table 7-4. CIDH Pile Cementitious Quantities.

Diameter	Maximum Cementitious		Mimimum Fly ash (%)			
14 feet	kg/ m^3	lb/yd^3	degrees F 15% fly ash	degrees F 25% fly ash	degrees F 35% fly ash	degrees F 50% fly ash
	300	506	126	118	115	112
	350	591	138	127	121	115
	400	675	146	137	133	118
	interpolated 425	717	151	142	138	123
	interpolated 450	760	156	147	143	128
	interpolated 475	802	161	152	148	133
12 feet	300	506	122	116	112	109
	350	591	133	124	119	112
	400	675	141	134	129	116
	interpolated 425	760	146	139	134	121
	interpolated 450	802	151	144	139	126
	interpolated 475	892	156	149	144	131
10 feet	300	506	118	112	109	105
	350	591	128	120	115	109
	400	675	136	129	124	112
	interpolated 450	760	146	139	134	122
	interpolated 475	802	151	144	139	127
8 feet	300	506	113	108	105	102
	350	591	118	115	110	104
	400	675	129	122	118	108
	interpolated 450	760	139	132	128	118
	interpolated 475	802	144	137	133	123
6 feet	300	506	107	103	101	98
	350	591	111	108	104	99
	400	675	120	114	111	103
	interpolated 450	760	130	124	121	113
	Interpolated 475	802	135	129	126	118

Mass Concrete Conclusions

For durable concrete the temperature of curing concrete needs to be evaluated prior to placement. This is a necessary step to ensure the structure will achieve its design life. Caltrans has a vast amount of experience in mass concrete control and has helped to advance the theory and practice. If active or passive thermal control measures are needed beyond normal practices to prevent the concrete from exceeding 160°F, then the concrete is defined by Caltrans as Mass Concrete. The theory and practice of mass concrete is important for evaluation of thermal control plans submitted by contractors, the assessment of plans and specifi ations to determine if mass concrete is applicable, and the determination by Caltrans of the thermal measures to be taken by the contractor when cost effective.

Lightweight Concrete

Lightweight concrete is similar to standard concrete, except that lightweight aggregates (35-70 pcf) are used, resulting in concrete that weighs less than 112 pcf for normal strength concrete and 120-125 lb/ft^3 for strengths above 6,000 psi. Dead loads for girders and bridge decks are lower, offsetting the higher material costs because the spacing of girders and supporting elements such as piers can be increased and foundation demands may be reduced. Seismic loads may also be reduced. Mix designs include Portland cement and SCMs which may reduce permeability and help reduce unit weight, lightweight aggregate, water and admixtures. Lightweight aggregates included in the mix design may be natural like pumice or manufactured by processing natural materials such as expanded shale, slag, clay, pelletized fly ash, or slate into aggregate size particles with a high percentage of void space. Attention is required to specifications when checking density, as oven-dry density and room temperature air-dry density will vary.

The New Benicia-Martinez Bridge completed in 2007, built with balanced cantilever cast-in-place segmental construction used High Performance Lightweight Concrete to meet material design needs which included over water spans exceeding 600 ft as shown in Figure 7-36. The mix design listed in Table 7-5 also had to take into account being pumped as much as 180 ft high and as much as 300 feet horizontally. The f'_c was 6,500 psi, but the Modulus of Elasticity requirements of 3,400 ksi resulted in f'_c over 10,000 psi, shown in Table 7-6. Attention to proper batching and adequate contractor quality control testing resulted in only two failing unit weight quality assurance tests. The compressive strength history shown in Figure 7-37 also indicates consistent batch control.

Table 7-5. New Benicia Concrete Mix.

Material	lb/yd^3
Cement, Type II-V	833
Fly Ash, Class F	49
Metakaolin	98
Normal Weight Sand	1,233
Lightweight Aggregate	858
Water	304
w/cm ratio	0.31

Table 7-6. Lightweight Concrete Properties.

Property	Specified Value	Average Measured Values
Density, lb/cu ft	125±2	125.2
Compressive Strength, psi	6,500 at 28 days	10,370 at 35 days
Modulus of Elasticity at 28 days, ksi	3,400 min.	3,800
Shrinkage after 180 days, %	0.05 max.	0.042
Specific Creep after 365 days, millionths/psi	0.48 max.	0.22
Splitting Tensile Strength psi @ 28 days	450 min.	490 (field tested > 28 days)

Figure 7-37. New Benicia-Martinez Bridge During Construction.

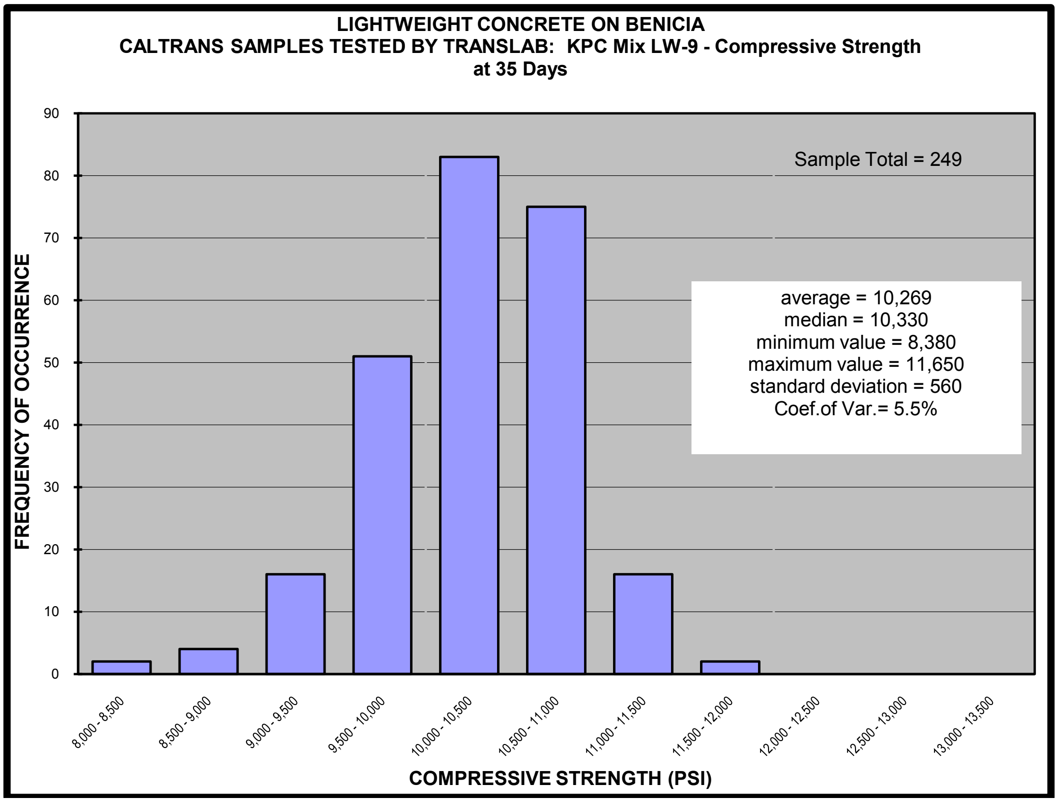

Figure 7-38. New Benicia-Martinez Bridge Concrete Strength History.

Shrinkage Compensating Concrete

After fresh concrete hardens and loses water, shrinkage can occur that will cause surface tension that leads to development of surface cracks. Historical shrinkage controls focused on mix design and curing controls to limit overall shrinkage or reduce the shrinkage rate. The water-cementitious ratio was reduced; free water ultimately bleeds and evaporates, leading to shrinkage. Stiffer aggregates were effectively combined with reduced cementitious materials to limit shrinkage. Cutting expansion joints in newly hardened concrete caused cracking to occur at the joints, leaving the remainder of the panel relatively unaffected by shrinkage. Proper curing practices also limit evaporation-induced shrinkage as does prolonged curing times. The method currently used to control shrinkage is inducing expansion as concrete hardens to compensate for subsequent shrinkage that occurs as concrete dries by use of expansive cement or a shrinkage-reducing admixture.

Expansive cement, containing calcium aluminate (ASTM C 845 Type K), expands after initial set to offset anticipated drying shrinkage. Type K expansive cement is not specified for Caltrans projects. The expansion creates tension in reinforcing steel and compression in concrete. Concrete produced with shrinkage compensating cement possesses properties similar to standard Portland cement concretes. Air-entraining admixtures are also equally effective for freeze-thaw durability. Chemically, the calcium aluminate hydrates with calcium sulfate to produce calcium aluminate sulfate hydrate. The chemical equation is shown below.

$$CA + 3CS + 2CH + 30H \rightarrow C_6AS_3H_{32}$$

Table 7-7 is a summary of concrete characteristics when comparing calcium aluminate concrete to standard Portland cement concrete.

Table 7-7. Type K vs. Portland Cement.

Characteristic	Shrinkage Compensating Cement
Water Demand	Higher
Consistency	Stiffer
Cohesiveness	Better
Final Set	Quicker
Strength	Better
Resistance to abrasion	Better
Sulfate resistance	Similar to Type V Portland Cement

Type K cement requires attention to reinforcement during design to ensure adequate but restrained expansion. The minimum recommended cementitious is 515 lb/yd^3. The inclusion of calcium sulfate leads to higher ettringite formation, which in turn requires additional water and is the source of expansion. Ettringite formation causes fresh concrete to stiffen. Pozzolans may have a negative effect on expansion and should be checked prior to use. As the cohesiveness increases, the risk of segregation decreases. Caution must be taken during placement during extreme weather conditions to avoid water loss because the early stiffening as plastic cracking may occur. After placement, moist cure is required for 7 days to ensure expansion during hydration.

Unlike expansive Type K cements, shrinkage reducing admixtures (SRA) control early-age and long-term shrinkage by reducing the surface tension of fresh concrete. When used, SRAs may slightly retard initial and final set. As expected, when set is retarded, the heat of hydration may also be reduced. Table 7-8 lists SRA effects on concrete.

Caltrans

Table 7-8. SRA Effects on Concrete.

Characteristic	SRA Use vs. Standard Portland Cement Concrete
Shrinkage	30 – 80% reduction
Heat of Hydration	Lower
Final Set	Slower
Strength	Lower to equal
Creep	Lower to equal
Chloride Permeability	Lower to equal
Resistance to frost action	Similar when air entrainment is used

Durability

The construction of expensive structures in various environments with corrosive and abrasive forces requires high durability concretes where design life can exceed 100 years.

- Winter weather with freeze-thaw cycles and heavy vehicle traffic possibly with tire studs or chains results in bridge deck with extreme wear in the wheel pathways.
- Reducing permeability limits the speed that corrosives like chlorides and sulfates can diffuse through concrete pores and attack reinforcing steel. Fine SCMs like silica fume, ultra fine fly ash, and metakaolin combined with low water-cementitious ratios result in concrete with low permeability, which resists flow of corrosive materials.

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A low water-cementitious ratio is the most important criteria. Where durability is a concern, the water-cementitious ratio should be below 0.4. Carbonation (the combination of calcium oxide with carbon dioxide) which eliminates the protective passivated coating on reinforcing steel, is also impeded by dense concrete. The use of SCMs also increases the electrical resistance of concrete, which inhibits the electrical circuit that develops as corrosion occurs. The presence of corrosion pathways caused by micro-cracking is also reduced with SCM usage. Micro-cracks are caused by variation between surface and internal temperatures while curing is minimized by maintaining a constant temperature throughout the curing process.

Fiber Reinforced Concrete

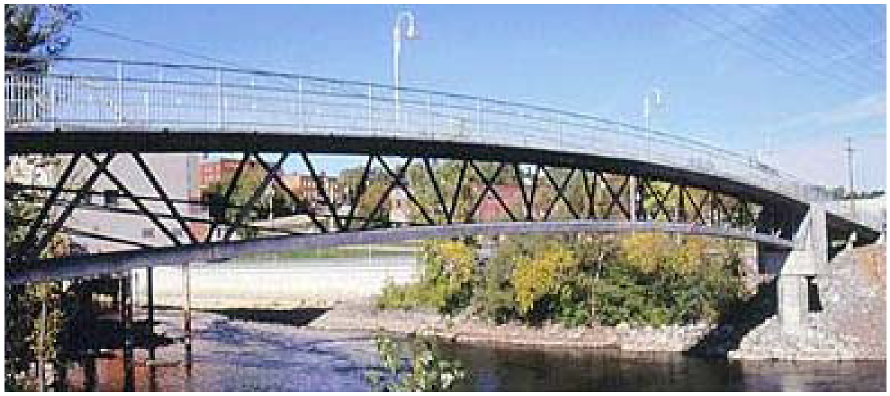
Fiber reinforcement is the addition during mixing of fibrous materials such as steel, plastic, alkali-resistant glass or cellulose to a concrete mix. The use of fiber reinforcement is steadily increasing as a means of increasing concrete durability. The earliest use of fiber dates back to the ancient Egyptians, who included straw in mud bricks for additional strength. After a concrete mix hardens, the fibers bond with hardened concrete and depending on physical properties, provide additional reinforcement. Fibers range from 0.25 to 6 inches in length; they usually have circular or rectangular cross sections, and may be shaped as bars or crimped. Steel fiber use is classified by volume, Low – < 1%, Moderate – > 1% and <2%, and High – >2%. Synthetic fibers with a much lower density seldom exceed 0.2% of volume. When used, fibers improve three-dimensional stability; compressive strengths have exceeded 30,000 psi. Fibers reduce cost by eliminating reinforcing steel but make the mix design less workable. Compared with steel reinforcing, fiber reinforced concretes are less susceptible to chloride attack and carbonation and more durable in freeze-thaw environments.

Reactive Powder Concrete

The Reactive Powder Concrete (RPC) process was patented under the trademark name Ductal® in 1994. Mixes are characterized by compressive strengths in the range of 25,000 to 35,000 psi and flexural strength in the range of 4,000 to 7,000 psi. The surfaces are abrasion resistant. Permeability is very low, with high resistance to carbonation, chloride and sulfate attack due to the small pore structure which also provides for good freeze-thaw durability.

The mix design consists of Portland cement, silica fume, crushed quartz flour, fine silica sand, water, high-range water reducing admixtures and reinforcing by steel or organic fibers. The maximum particle size is 600 microns, and the overall gradation is optimized for maximum density. The low water-cementitious ratio (<0.2) requires use of high range water reducers for workability.

RPC structural elements are more compact and lighter in weight that standard Portland concrete elements. The first bridge made with RPC was a pedestrian bridge in Sherbrooke, Quebec built in 1997. The bridge, a segmental structure, consisted of six precast segments for a 198-ft span as shown in Figure 7-39.



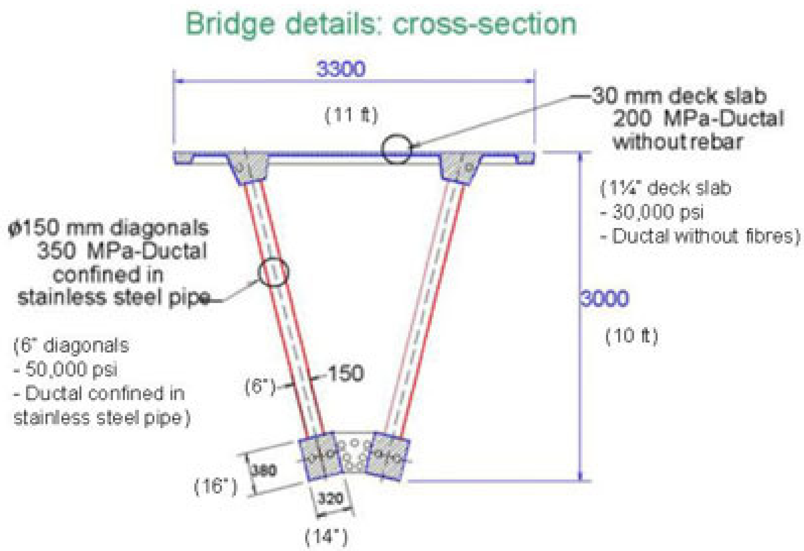


Figure 7-39. Sherbrooke Pedestrian Bridge.[5]

[5] http://www.pci.org/view_file.cfm?file=JL-99-SEPTEMBER-OCTOBER-6.pdf (visited 6/10/10)