



CHAPTER 1

STRUCTURE CONCRETE CHARACTERISTICS

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1 STRUCTURE CONCRETE CHARACTERISTICS

Throughout the history of the California Department of Public Work's Division of Highways and now the California Department of Transportation (Caltrans), concrete has been a major material for California's transportation facilities. Around the world, as in California, constant research into materials and methods has led to gradual improvements in concrete. Over time, the incremental changes have resulted in significant quality differences. Advancements have occurred in:

- The manufacture of cementitious materials
- Use of chemical admixtures
- Mix design procedures
- Batching, mixing, and placing
- Testing
- Concrete compositions that expand the uses of concrete beyond the traditional definition of Portland cement

Portland cement concrete, a 19th Century material, remains an effective material that meets the complex engineering needs of 21st Century bridges and transportation facilities. Bridge engineers having the opportunity to design, build, and maintain California's vast inventory of concrete structures should appreciate this evolutionary advancement and continue to be a part of it.

When Jose de Navarro built the first modern cement mill in 1896 at Roundout, New York, the definition of "good" concrete was advanced. Over 100 years later, hydraulic cements, manufactured with methods developed by de Navarro, are used to bind aggregate into a "good" concrete that is used to construct engineered structures that are as modern today as they were then. Although the definition has changed with new needs and technological advancements, producing "good" concrete has always been a major engineering objective.

"Good" concrete needs to be designed. Duff Abrams, the discoverer of the rule of water-cement ratio, defined "design" in a paper delivered to a Portland Cement Association (PCA) conference in 1918 which still holds true today:

"The term 'design' is used since it is the intention to imply that each element of a problem is approached with a deliberate purpose in view, which is guided by a rational method of accomplishment."



But what is good concrete? There are several qualities associated with concrete from mixing and placing to its final use as a structure or at least portions of a structure. The design engineer specifies a concrete that meets design strength and counteracts known deleterious environmental factors. Selection of ingredients is important to long-term durability. While concrete is being transported and placed or poured and surface finished into the design engineer's envisioned hardened shape, consistency and workability is important. Measures taken immediately after placement and during the early stages of the hardening process are important to concrete performance as a structure or structural element.

Contractors want a workable mix for placement (or pouring) and finishing. Workability generally refers to the ease that materials can be mixed into concrete and the subsequent handling, transporting, and placing with minimum loss of homogeneity. Measure of bleed water is a workability measurement since bleed water results in a lack of homogeneity at the surface. Several workability tests are used to measure the ability of a flowing or self-leveling concrete to resist segregating into its component parts while flowing from a point of discharge over a relatively large distance of travel within the form.

Consistency is related to workability. Consistency is a measure of the geometric stability of fresh concrete. Slump and slump flow tests measure the movement of concrete from the static state of an unconfined cone shape. A Kelly or Penetration Ball measures the distance a sphere shape of a given size and weight sinks into a flat surface of fresh concrete. A drop table is used to measure consistency of mortar. An unconfined cone of mortar is placed on a flat brass disk that is dropped to a sudden stop several times. The flow of the mortar is determined by measuring the diameter of the resulting pancake. Efflux times of grout flowing through an orifice are consistency measurements. Field engineers check concrete mixes for consistency and sometimes workability characteristics when overseeing the placement process.

Plasticity refers to a mix being plastic, which is its ability to be molded into a shape without crumbling or flowing as a liquid. Generally plasticity is not tested though sometimes it is equated with consistency. Mixes that have very stiff or very fluid consistencies are not plastic. Concrete mixes approaching initial set are said to have lost plasticity.

The methods used for placing and consolidating concrete, controlling temperatures during hardening, finishing and curing concrete are important in the selection of the material combinations used for the mix design. Inattentive workmanship and inappropriate selection of materials can lead to placement difficulties, less than optimum concrete strength, inadequate durability and performance, or irregular appearance. Fresh concrete must be placed such that the finished structure concrete meets the specified grade, cross-section, surface smoothness and frictional performance requirements. After hardening, concrete in a chemically stable environment will continue strength development for years. The inherent



volume changes of concrete needs to be minimized and accounted for in the design of the structure. The concrete must be able to withstand the stresses and strains to which it will be subjected throughout its lifetime.

After construction, maintenance engineers inspect structures for changes in deflection, cracking and other signs of deterioration. Durable concrete requires periodic inspection and repair, to maintain a serviceable structure throughout its lifespan.

Good concrete is therefore a concrete that can be cast as easily and efficiently as possible, and upon hardening have appearance, shape, strength, durability, and surface qualities as engineered.

Workability and Consistency

Workability, a performance term, refers to the ease with which fresh concrete may be produced, transported, placed, consolidated and finished without segregating into component parts. Workability is affected by:

- Economic considerations
- Water content
- Types of cementitious materials
- Aggregate gradation
- Aggregate shape and texture
- Aggregate moisture content
- Admixture use
- Temperature
- Placement conditions
- Forming
- Reinforcing steel layout
- Finishing requirements

Workability is dependent on job considerations. The rheological aspect of workability is also referred to as fluidity of flow. Fluid flowing mixtures are efficient when placed from trucks or buckets directly into the forms or when concrete is placed under water, as in wet pile construction, or when the structural elements are densely packed with reinforcing steel. A stiff mixture might be usable when there is wide or open placement, when slip or jump forms are used, or when containment of the concrete within forms is difficult such as combined stem walls and soffit placement.

Economic considerations also have an effect on workability. The most economical concrete mix is the one that meets all required specifications and:

- Balances the costs of materials and placement.
- Maximizes the proportion of aggregate to cement; the optimal aggregate gradation will allow use of the minimum cementitious material specified while producing concrete of standard consistency, meeting fluidity requirements may require use of chemical admixtures that enhance fluidity.
- Minimizes the water-cement ratio, concrete strength will increase as the quantity of concrete increases and as the water-cement ratio decreases.

In general, consistency testing is an indication of workability. However this is not always true. Concrete having the same consistency measurement value may exhibit vastly different finishing characteristics. One mix may be “harsh,” “sticky,” or “bony” compared to another having the same slump. A slump flow value does not indicate whether or not a concrete will segregate. High percentages of crushed coarse aggregate, with elongated, angular, flat surfaces can bind against each other and result in a mix that does not flow around reinforcing steel or into tight spaces. The slump of such a mix can be the same for a mix with well-rounded aggregate that flows easily around reinforcing steel and into tight spaces. Thus workability may be different for the same specified consistency limits.

The ideal workable mixture, from a consistency standpoint, is dependent on the placement parameters. An ideal consistency for concrete placed with a moving form, or where forming is awkward such as the surface having a large grade, or at a wall rising from a horizontal surface placed monolithically, is a mixture that can be readily molded but will resist changing shape if its mold or form is removed prior to initial set. Concrete of this ideal consistency flows sluggishly without segregation or not at all after vibration ceases. Paving a slab, such as a pavement, with a slip form paver requires a stiff mix that sometimes appears to be “crumbly” when dispatched from dump trucks or end dumps. The concrete is restricted to 0 - 1 inch penetration (0 - 2 inches slump). This is constructible because of the energy a slip form paver is able to apply to the mixture. It is desirable because no forms are used and the vertical edges can be as much as 12 inches high. The edges need to maintain the flat vertical plane without forms. Roller compacted concrete is stiffer still as compaction is by the weight of machines used to compact AC or soils. In contrast very fluid, self-leveling mixtures will produce quality concrete even when placed under water or into tight strong forms. Very fluid concrete can only be used where the top surface is level. Self-leveling flowing mixtures may be necessary where access for discharge is limited or areas with highly congested steel reinforcement.

Consistency is roughly proportional to the water content for a given aggregate source with standardized gradation and proportions. Consistency is affected by aggregate gradation, proportions and characteristics and also by the characteristics of admixtures in

use. Accordingly, for a given combination of aggregates and cement content, changes in consistency from batch to batch are usually the result of variations in the amount of free water or admixture use.

The recognized ASTM standard and traditional measure of concrete consistency is the Slump Test (CTM 556), shown in Figure 1-1. The slump test is performed by filling a frustum-shaped metal form with fresh concrete and then measuring the subsidence of the concrete mass after the form is removed. The subsidence, or slump, of the concrete gives an indication of the relative consistency of the concrete mixture. A high slump indicates a fluid mix useful for such elements as CIDH piles. Self consolidating concrete flowability is similarly measured with flow cone and “J ring” measurements.



Figure 1-1. Slump Test.

The Ball Penetration Test (CTM 533), sometimes referred to as the Kelly Ball Test, is a standard consistency test with Caltrans. There is a correlation between the Kelly Ball and Slump Tests. Kelly Ball reports the results as the inches actually displaced by the hemisphere. Slump Test values are typically twice the size of a ball penetration measurement. Thus a concrete with a consistency measurement of 1 inch penetration would have a consistency measurement by Slump Test of 2 inches. The Kelly Ball Test is easier and faster to perform than the traditional Slump Test, and gives uniform results for stiffer mixes. Leveling a portion of the fluid concrete mass, placing a standard spherical weight on the leveled surface, and measuring the depth of penetration of the weight into the concrete constitutes the Kelly Ball Test. The depth of penetration is an indication of the relative consistency of the mixture.



Consolidation

Ideally the in-place concrete needs to be consolidated to make it free of aggregate segregation and air spaces. Cement paste should be in intimate contact with the coarse aggregate, interior faces of the formwork, reinforcing steel and other embedded objects. The consistency of the mix is the key to consolidation. The more water in the paste and the higher the paste content the higher the fluidity. Therefore consolidation of newly placed concrete is associated with the workability, which includes consistency, and the method of consolidating fresh concrete. Manual tamping, rodding and power vibration are consolidation methods. Concrete flows more easily into tight spaces in formwork and reinforcing steel during vibratory consolidation. Vibration reduces entrapped air, helping to eliminate air bubbles and rock pockets, resulting in a denser, more compact concrete. However, over-vibration could cause segregation. (Refer to Chapter 5 for additional information on consolidation.)

With viscosity modifying admixtures, high range water reducer admixtures, and additional fine aggregate in the mix, a self consolidating concrete can be designed that needs little if any vibration or tamping for dense compacted concrete free of segregation and free of voids around steel or in tight spaces.

Bleed Water

As defined by ACI and ASTM C 125 bleeding is “the autogenous flow of mixing water within, or its emergence from, newly placed concrete or mortar; caused by the settlement of the solid materials within the mass...” The presence of bleed water in concrete is a function of water-cement ratio, cementitious contents, aggregate gradations, admixture use, concrete thickness and forming. As concrete solids settle into final position, water segregates and rises to the surface as bleed water. Looked at another way, the aggregate and paste settle and push water to the surface.

Note: If water can flow through the bottom surface of formwork, as in the case where a concrete slab is poured on low saturated sand, gravel, concrete permeable base, or any highly permeable material, bleed water will not appear and there will not be a workability issue.

As the bleed water collects and rises to the surface, micro-channels are cut in the fresh concrete. Concrete strength is lower because micro-channels remain after concrete hardens. The micro-channels also serve a pathway for corrosive chemicals, hastening concrete deterioration and reinforcing steel corrosion. Bleed water may also be trapped in on the lower side of aggregate and reinforcing steel. The voided spaces signify reduced concrete strength, increased permeability and reduced durability.



Proper finishing methods for floor slabs require the bleed water sheen to disappear before working the final steel float and finish. If finish floating is done prior to this, undesirable water is sealed into the top surface of the concrete, resulting in cracking and crazing and premature maintenance and repair costs.

The Hardening Process

The change in concrete state from fluid to solid occurs as a result of a chemical process between cement and water, referred to as hydration. The hydration process begins as soon as the cement contacts water, and ends when the concrete reaches its ultimate strength or water is no longer available. Hydration occurs in three phases, initial set, final set, and progressive hardening. The main strength compounds of cement paste are formed during hydration.

Long term strength gains through the pozzolanic reaction are made possible by adding supplementary cementitious materials (SCMs) to the concrete mix. SCMs are usually made from amorphous silica and alumina oxides. The pozzolanic reaction, is the result of silica or alumina combining with calcium hydroxide, which is a product of the Portland cement hydration process. When SCMs such as fly ash are used as replacement of Portland cement, the development of strength over time maybe slower, extending over a longer time. For this reason the traditional time of 28 days for strength development of mix designs has been increased. On a project that required 8,000 psi or more, the time was extended to 56 days¹. When fly ash use exceeds 35% replacement as a method of controlling temperature in mass concrete the time has been extended to as much as 90 days. For strength requirements from 4,000 to 7,000 psi, 42 days have been allowed for strength development.

Initial Set

The first phase, initial set, is marked by a decrease in consistency as the concrete begins to stiffen considerably. Loss of plasticity can occur gradually as the cement hydrates and the cement paste begins to harden. Or set can occur quickly, a flash set, as with fast setting hydraulic cement. Initial set is when it is no longer possible, with standard placement tools, to place fresh concrete onto the setting concrete, or consolidate setting concrete along the surfaces of a break or crack. The Vicat needle (ASTM C 191) or the Gillmore needle (ASTM C 266), can be used to identify initial set. Both tests measure the penetration, or lack of penetration of a standard diameter needle under standard pressure.

¹ Strength development time is extended to 56 days for any concrete meeting new specifications (2010 Standard Specification, Section 90-1.01D(5)(a))



Initial set is by chemical design. Initial set must not occur before there has been time to place and finish the concrete. Initial set for the vast majority of placements is typically designed to be within two hours of placement. In the case of CIDH piles placed in wet holes the set time for the first concrete discharged into the hole must be for the duration of the entire pour; this can be as much as 12 hours for very large CIDH piles. On the other side of the spectrum where a very rapid strength gain is desired, set time can be only minutes after placing. In this procedure, timing, machinery, and staffing are necessary to get the concrete in place and finished without delay.

As cement particles decrease in size, the hydration rate increases and initial set occurs faster. The set time increases as the ratio of water-cement increases. Set time decreases as temperature increases, and increases with falling temperatures. Admixtures affect set time by design, either hastening or delaying set time. The setting time of Portland cement is regulated by the amount of gypsum added to the clinker prior to final grinding. Without any gypsum, cement will set quite rapidly. The setting time increases roughly proportional to the amount of gypsum added. Although individual cement manufacturers can vary the percentage, the use of gypsum to modify set time typically requires about 3% gypsum (by weight) to the clinker to meet the requirements of ASTM C 150. For rapid-setting hydraulic cement, setting times have been regulated with accelerating admixtures, citric acid and borax.

False Set

Occasionally, a Portland cement concrete mixture will stiffen almost immediately after mixing cement and water without the generation of hydration heat. The phenomenon is known as false set. This can sometimes be a problem when using “mobile mixers” since the contact time during the “mixing” of the water and cement is measured in a few seconds. Stiffening associated with false set can be overcome with additional mixing or when additional water is permitted, by adding water.

If the temperature of the gypsum and clinker becomes excessive during the final grinding stage of the cement manufacturing process, the gypsum may transform chemically into plaster of Paris. When water is added for concrete mixing, plaster of Paris hardens immediately, causing premature stiffening of the mix. Cooling clinker prior to grinding and cooling the grinding mill are methods of reducing the use of plaster of Paris. False set may be caused by excessive temperature when cement temperature exceeds 140°F at the time of mixing. False set may occur in cold weather as a result of over-heating of the water and/or aggregates, or when the heated water mixes directly with the cement. Temperature-related false-set can be reversed by additional mixing, without adding water to restore plasticity. Additional mixing should allow the impacted concrete to set in a normal manner without detrimental effects.



Final Set

The second phase is the final set. Final set time is variable. By design, the time could be less than 1 hour or up to 24 hours later. During final set, the concrete is a soft solid without appreciable surface hardness but will retain shape without formwork support. It will support light loads without indentation; however, it is easily abraded and its surface can be scored or roughened with little effort. The heat from hydration increases significantly at final set. The length of time required for final setting is a function of the cement's characteristics and various mix design factors, i.e., cement content, water-cement ratio, and admixture use. Final set usually occurs in less than 1 hour, but may occur over 24 hours after initial set.

Heat of Hydration

Heat of hydration refers to the heat produced by the exothermic chemical reaction between Portland cement and water. Heat is usually not of major concern when it escapes into the surrounding atmosphere. The temperature rise due to the heat of hydration can be beneficial during periods of cold weather, since it helps maintain favorable curing conditions.

In the case of mass concrete pours, heat expansion and subsequent cooling contraction of the partially hydrated (green) concrete, can result in severe stresses and appearance of thermal cracks. When hydration temperatures exceed 160°F, the hydrates that are formed are unstable and expansive. When one dimension of a concrete placement exceeds 7 feet, a mitigation plan for excessive heat is usually required. In unique situations, the concrete materials or placement methods could create higher temperatures during hydration than normally expected. For example, in the Benicia cast-in-place segmental bridge, the soffit and deck were only 1 foot thick but the high Portland cement content and lightweight aggregate would have produced dangerously high temperatures had preventive measures not been taken.

Strength Development

Concrete gains strength rapidly after placement; then the rate of gain grows progressively slower, but may continue for years. Traditionally, 100% Portland cement concrete mixes with 564 to 658 pounds of Portland cement per cubic yard (pcy) and water-cement ratios of 0.5, required 28 days for strength attainment. The rule of thumb for early breaks was 65% of 28-day strength at 7 days and 80% at 14 days. When high strength concrete was specified with water-cement ratios below 0.4, the 7-day strength was expected to be 80% of the 28-day strength.

Now, with the current practice of using SCMs, non-Portland hydraulic cements, and specialized set control (retard and accelerate) admixtures, the only similarity with the traditional strength versus time graph is that the rate of strength development decreases with



time. Some (not those with $\leq 3,600$ psi compressive strength) concrete mixes with SCMs are allowed 42, 56 and sometimes 90 days to attain design strength because the rate of strength increase maybe slower than 100% Portland cement. Concrete with fast setting hydraulic cement can be designed to have strength in hours instead of days. Using fast setting hydraulic cement can result in rapid strength gains exceeding 5,000 pounds per square inch (psi) in 2 hours; this represents 50% of ultimate strength which will develop progressively for years.

During hydration of Portland cement, water combines with dicalcium and tricalcium silicates, tricalcium aluminates and tetracalcium aluminoferrite to form calcium silica hydrates, calcium alumina hydrates and calcium hydroxide. Water is necessary for hydration of any hydraulic cement and subsequent pozzolanic reactions when SCMs are used. SCMs combine in a pozzolanic reaction with calcium hydroxide, the hydration byproduct, in the presence of water to form calcium silica hydrates. Cementitious materials in concrete do not gain significant strength by drying; they gain strength when water is available for the hydration (chemical) reaction. Therefore once water is unavailable, strength development peaks.

Compressive Strength

Compressive strength is the most universally accepted measurement of concrete quality, because concrete structural elements are designed using a specified concrete compressive strength and strength can be easily tested. During construction, concrete strength is typically determined by breaking standard concrete cylinder samples (6-inch diameter \times 12-inch height) that have been moist cured for 28 days at 70°F. After construction, cores can be taken from a structure at any age and tested for compressive strength.

ASTM allows breaking 4-inch \times 8-inch cylinders, except a test is considered to be three not two cylinders. Cylinders can be cured at temperatures specifically controlled to simulate the temperature of the actual concrete in the structure element. This can be as simple as leaving cylinders on or near the structure or placing them into environmental containers. For the latter, temperature sensors such as a thermocouple are placed inside the structural concrete during placement. The container temperature is regulated to match the temperature of the in-place concrete. When the cylinders are tested, the strengths are comparable to the in-place concrete at the time of the test.

Since the hydration process is exothermic, a fairly accurate estimate of compressive strength can be obtained by integration of the time temperature curve of a curing cylinder. The accumulation of the area under the curve at any point in time can be related to the strength of the concrete at that time. This is known as the Maturity Method. Concrete temperature histories are correlated with strengths in lab tests. This data is used to produce a temperature/time/strength relationship. The temperature/time/strength relationship is unique to the given concrete tested.

When the Maturity Method has been allowed in Caltrans projects, a verification test was also required. For the Bay Area segmental bridge projects, a nondestructive test, such as a Capo Pull-Off or a Windsor Probe was required. For the Capo Pull-Off test, a ring is cast into the concrete. The force required to release the ring from the concrete was correlated to a compressive strength. A Windsor Probe shoots a pin into the hardened concrete. The depth of penetration of the pin corresponds to a compressive strength. If the nondestructive test indicated a significant difference in strength than the Maturity Method, then a cylinder test was required to verify concrete compressive strength. The cylinder test was assumed to be conservative. If cylinders were stored in an environmental container, engineering judgment was needed.

Effect of Water-Cement Ratio on Concrete Strength

The primary factor, besides the quality of the cementitious materials, influencing strength and overall quality of any concrete is the water-cementitious material ratio. Increasing cementitious content lowers water-cementitious material ratio for any given consistency. A secondary but important factor is the quality of the aggregate. Often a ‘high quality’ aggregate is such because it reduces the amount of water needed for a given slump. Thus, well rounded rock and sand make good concrete because it requires less water than a crushed angular rock to produce a desired consistency.

The effects of the water-cementitious material ratio on strength and permeability are shown in Figure 1-2. As the amount of water increases, strength will decrease and permeability will increase.

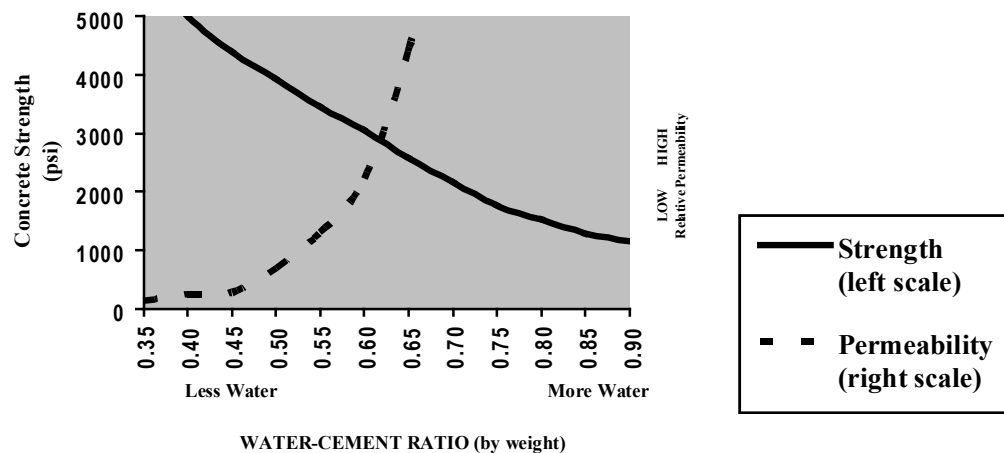


Figure 1-2. Typical Concrete Strength and Permeability Versus Water-Cementitious Material Ratio.



Curing

Obtaining optimum strength depends greatly on the extent to which the cement hydrates during the hardening process. Curing is a process by which a concrete mixture is kept in a moist state following initial setting. It is intended to ensure hydration, as well as to prevent the formation of surface cracks due to a rapid loss of water while the concrete is plastic and low in strength.

Under field conditions, concrete can become partially dry within only a few days after controlled curing concludes. Thereafter, hydration and pozzolanic reactions may occur slowly and intermittently over a period of many years using moisture occurring with rainy weather or prolonged periods of high humidity. For slabs and footings in contact with moist earth, moisture will continually wick through the concrete and hydration may continue indefinitely.

At a minimum, the paste must be kept saturated, or nearly so. If the paste is not kept saturated, the hydration process will cease when all the free water has evaporated from the paste. Thus, water must be added during the curing process to compensate for surface evaporation and for water consumption during hydration.

Strength of Concrete in Existing Structures

When core strengths are not readily available, 5,000 psi can reasonably be assumed for the current strength of concrete in existing concrete structures built between 1930 and the turn of the century. This assumes the construction was proper, the service loads have been within design assumptions, and the concrete is not subject to any durability issues such as alkali-silica reaction, delayed ettringite formation, sulfate attack, freeze-thaw damage, carbonation or other materials related distress.

Most if not all concrete during the time period between 1930–1965 was based on a working stress (service stress) design. Per this method the allowable working stress (f_c) at the outer fiber of concrete was limited to a fraction of the 28-day ultimate compressive strength (f'_c) of the concrete. The 1936 ACI code used 0.40 as the coefficient limiting the calculated stress to a fraction of the specified 28-day compressive strength of the concrete (unless it was adjacent to supports of continuous beams then it could be increased to 0.45). In 1986, the AASHTO Standard Specifications for Highway Bridge Designs still had the coefficient at 0.40. Thus we can assume the maximum designed stress for a bridge was limited to 40% of the 28-day ultimate compressive strength of the concrete ($f_c = 0.40 f'_c$) for bridge structures built between the mid-1930s through the mid-1980s using working stress design method. Typical f_c , the maximum compression extreme fiber stress or allowable working stress, through this period ranged from 1,000 psi to 1,300 psi, with 1,200 and 1,300 being common values. Therefore, this means before leaving the engineer of record, f'_c , the



28-day ultimate concrete strength used in the calculations, ranged from 2,500 psi to 3,250 psi. Using a value for f_c of 1,200 for calculating the beam stresses, we can safely say the mixes were intended to be 3,000 psi at 28 days. An f_c of 1,300 makes f_c' 3,250 psi. (For Ultimate Strength designs, 3,250 psi for f_c' was a common value used on bridge plans produced into the 1990s.)

The practice used then by the California Department of Public Works / Division of Highways to ensure actual minimum concrete strengths, was governed by the class of concrete required by materials specifications. The important concrete material requirements were the minimum sacks of cement per cubic yard along with a limit on total water. The 1954 California Division of Highways Standard Specifications required “Class A Concrete” for structures; this was a 6-sack mix, limiting water to no more than 52 pounds per sack (W/C ratio = 52 pounds/94 pounds = 0.55). This was the same practice for most of the Bay Bridge built 20 years earlier, though there were additional requirements for the Bay Bridge including a lower water limit of 47 pounds per sack in a 6-sack mix design.

The 6-sack requirement for bridge structures was used from about 1970 until the mid-1990s (with the exception of decks which were increased to 7 sacks to enhance abrasion and corrosion protection). The 6-sack requirement continued until units were changed to metric, whether designed by working stress or ultimate stress, for any design assuming 28-day compressive strengths less than 3,500 psi. (Currently, where f_c' is not above 25 MPa or about 3,600 psi, even with the current metric conversions and the use of only Ultimate Stress design methods, the standard specifications are very similar to the 1954 and older standards.) Using the typical cements of that time period, the 6-sack mix specifications usually resulted in concrete near 4,000 psi. When minimum quality aggregates and air entrainment were used, concrete strength was still above 3,000 psi.

Any Portland cement concrete from the early 1950s through 1960s that neared 4,000 psi in 28 days should have reached 6,000 psi by now, while those near 3,000 psi located in areas having lower quality aggregates and air requirements should still exceed 5,000 psi by now. If there are reasons to doubt the 5,000 psi assumption, compressive cylinder cores should be taken and tested to verify strength.

An example supporting the assumed strength is the Cypress Street Viaduct, designed and built in the 1950s (BR. #33-178). After the 1989 Loma Prieta Earthquake caused the Viaduct to collapse, 59 cores were taken and tested. The average strength of concrete from the fallen structures was over 6,600 psi, while only 3 cores tested below 5,000 psi (4,520, 4,870 and 4,890).

Cement manufacturing technology has gradually changed and concrete strengths rose. Ultimate Strength design makes the f_c' the same value as used for the design assumption



with the safety factor applied to the loads. But f'_c was still typically no lower than 3,250 psi and the concrete was still by the same class requirements. So a safe estimate for structures after 1970 can still be 5,000 psi. It should be noted that currently this 5,000 psi value is mostly used as an estimate in flexural analysis. For shear capacity estimates, 3,250 psi is recommended per Caltrans Seismic Design Standards (SDC) 3.6.1.

Volume Changes and Deformations

Not only does concrete strength and elasticity change with time after hardening; the dimensions of concrete structural elements also change. A characteristic of all materials is the volumetric change that occurs immediately with changes in temperature, moisture content, and applied loads. There are also time dependent plastic volume changes resulting from dry shrinkage and time dependent plastic deformation or “creep” caused by continuing applied loads. Concrete shrinks upon drying and swells upon wetting, but swelling is never equal to the original volume so there is an irreversible or a plastic (not to be confused with “plastic shrinkage”) deformation with shrinking. As with strength, volumetric changes due to temperature, moisture content, creep and irreversible dry shrinkage are dependent also on the selection of materials in the mix design.

Even when loads are in the elastic range of concrete there is still some plastic deformation referred to as hysteresis. Hysteresis is primarily due to micro and macro cracking of the concrete as it goes through load cycles. Hysteresis is considered when predicting the behavior of an element during a seismic event as there could be multiple load cycles during the event. Hysteresis that occurs with temperature change is not considered in design practices.

Creep

Just like strength, creep and shrinkage occur rapidly at first and then gradually slow to almost nothing after a few years. The ultimate deflection (creep) for a concrete structure decreases with increased concrete maturity at the time of loading. Cement type and content, aggregate type and size, member shape and size, and reinforcement amount and type (mild or stressed) all affect creep. Generally creep magnitude is proportional to member stress, and measured in the direction of the applied load. As much as 75% of ultimate creep can take place by three months and is assumed to essentially cease after 4 years. Bridge Memo to Designers 7-10, doesn't address creep specifically, but recommends assuming one-half the total anticipated shortening should be out of the structure at 11 weeks when calculating the movement rating. Although the volumetric changes may appear small (measurements are taken by dial indicator reading to the nearest 0.00001 inch for a 1 foot length), the changes are significant to the expectations of structural behavior. Setting camber for a bridge is a result of the ability to predict creep.



Prior to 1958 camber design practices assumed creep would be four times as great as instantaneous deflection and the ultimate deflection of structures closely approximated the assumed deflection. In 1958, our specification for the minimum time before falsework removal changed from 21 days to 10 days. As structures built under the 1958 specifications entered service, many of them began developing sagging decks. It became apparent that the camber design was inadequate for structures built under the 1958 specifications. It was also observed that when falsework remained in place for the previously specified 21 days period that the structure met long-term deflection expectations. Falsework remained in place for 4 months on one structure, the Fields Landing Overhead on SR-1 in Humboldt County and when it entered service, the anticipated deflection never occurred. Adjustments in camber design and construction practices have since been made.

Although predicting the long term shape and volume changes of concrete structures due to creep and shrinkage is an important part of bridge design in general, more precise quantitative prediction of the gradual volume changes in concrete can be vital to the design and performance of segmental concrete bridges. There is a smaller margin for error in long-term volume change for balanced cantilever segmental construction. As segments are placed in opposite directions toward a closure point, the final grades are dependent on volume change and stresses due to the volume changes. Significant stresses can occur in both the superstructure and in the piers due to segments changing volume with time if not properly accounted.

At the level of prediction needed for segmental design and specifications, a reasonable database for at least the shrinkage characteristics of potential aggregates is needed. The first US segmental bridge was Pine Valley done by Caltrans in 1974. Prior to design of this structure, shrink and creep tests were performed on the anticipated mix, which included the aggregate source. The same was true for the quasi-segmental bridge done by Caltrans, Napa River Bridge, in 1976 (quasi since it was not by cantilevered traveling forms but on moving the falsework for each segment).

There have been some notable consequences when volume change is not adequately accounted. One famous example is the Parrotts Ferry Bridge, a segmental bridge built by the Army Corps of Engineers in 1979, where there was a 22-inch deflection in the center span (See Figure 1-3). Other segmentally-constructed bridges in the world with inadequate creep assumptions had more catastrophic results including collapse.



Figure 1-3. Parrotts Ferry Bridge, New Melones River, 22 Inch-Deflection².

² http://highestbridges.com/wiki/index.php?title=Parrotts_Ferry_Bridge (visited June 30, 2010).



Dry Shrinkage

Unlike creep, which is in the direction of load, dry shrinkage is not affected by load direction. Tensile stresses will develop in concrete structural elements when they are restrained from shrinking. Cracking of concrete structures has been investigated for decades, with particular interest in deck cracks caused by shrinkage. Curing practices developed over the last century have been effective in preventing early cracking caused by plastic shrinkage while stress cracking can be mitigated by proper design of the element. However dry shrinkage cracking has been accounted for at best by crack control such as engineering the crack to predetermined locations or spreading them over a large area.

A cast-in-place concrete deck on precast girders adds to the inherent stresses due to dry shrinkage. The 1958 ACI Journal Proceedings, “Tentative Recommendations for Pre-stressed Concrete” by ACI/ASCE Joint Committee 323, was quoted in the March 2004 Concrete International Magazine:

In structures with a cast-in-place slab supported by prestress beams, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of precast beams....When cracking load is significant, such stresses should be added to the effects of the load. (Section 212.4.6 “Shrinkage Stresses”)

Various strategies to mitigate dry shrinkage stresses have been proposed over the years including using shrinkage compensating cements such as Type K, reducing the cement content while reducing water as much as possible, using fibers, and even slowing down the hydration process significantly by replacing up to 70% of the Portland cement with fly ash and waiting months for strengths typically achieved in days. Shrinkage compensating cement initially expands upon curing and thus dry shrinkage results in near net zero shrinkage. To be effective this initial expansion needs to be restrained so that the accompanying dry shrinkage releases stress rather than volume changes. Several decks using Type K or similar cements have been tried over the last few decades. Success of Type K at reducing cracking has been marginal at best. In the 1980s, one method was proposed where concrete was put under vacuum after placement to literally suck water from the concrete; this method was never fully developed.

During the construction of the San Mateo/Hayward Bridge widening project completed in 2002, deck cracking appeared to be due to the cast-in-place deck being placed over precast girders and precast deck bottom panels. Paying more attention to curing details solved some early initial problems with plastic shrinkage. But as time went on cracking was noted in the deck after several weeks. This cracking correlated with the hypothesis that the deck was undergoing a volume change due to shrinkage but was restrained by the “preshrunk”

precast girders and precast deck bottom panels. As noted in the 1958 ACI Journal quote referenced above regarding induced shrinkage stresses, this type of cracking would be typical to this type of construction. The cracks were repaired with methacrylate to bond and seal the cracks at the end of the job.

Toward the end of the San Mateo/Hayward Bridge project a similar phenomena was taking place on a series of bridges on I-80, near the town of Truckee in the Sierra Nevada Mountains, being built as part of a realignment and replacement project. These bridges consisted of placing cast-in-place concrete decks on precast Bulb Tee Girders. Again cracking was noted several weeks after deck placement. It was decided that since there were several deck placements and some lead time before the next placements Shrinkage Reducing Admixtures (SRA) could be evaluated as a possible solution. Between August 2001 and May 2002 the problem of cracking was addressed during stage construction on six bridges on this project located on I-80 in the Sierra Nevada Mountains. Though perfect control of the environment and mix parameters were not fully achieved since this was an ongoing construction project, adequate assumptions could be made and more than reasonable conclusions were drawn. There was a dramatic reduction in cracks when SRA was used (see Figure 1-4). This was repeated on the Angels Crest Bridge in 2008. To date the decks are crack free.

At about this same time testing was being done on SRA being used to control shrinkage in the Skyway portion of the east Spans of the San Francisco-Oakland Bay Bridge. The main purpose of the SRA on this structure was to control geometry of this precast segmental bridge structure. It was apparent from the testing for the Skyway and in the literature that dry shrinkage could be significantly reduced by use of these chemical admixtures.



Figure 1-4. Deck Cracks Before and After Use of SRA.

Shrinkage and Swelling Due to Moisture Change

During the curing period, concrete is maintained in a continually saturated or nearly saturated condition. When curing ceases, the free water within the concrete mass soon evaporates and, as the concrete dries, reversible and irreversible shrinkage occurs. While dry shrinkage may continue for several years, about one-third of the total expected drying shrinkage occurs within the first month and about 90% within the first year. Dry shrinkage ranges from about 600 to 800 millionths per unit of length (approximately 1 inch per 120 feet).

The most important factor influencing drying shrinkage is the water content. Other factors influencing swelling and drying shrinkage are types of cement and aggregates used, and amount and type of reinforcement used.

Non-reinforced concrete shrinks and swells more than reinforced concrete. The reinforcement restricts shrinkage and swelling but does not prevent it. Typical drying shrinkage in reinforced concrete is approximately 200 to 300 millionths per unit of length (or about one-third of the shrinkage for a comparable mass of non-reinforced concrete).

When concrete dries it shrinks more rapidly at the surface than within the mass. This produces stresses, which may cause the formation of a network of cracks extending a short distance inward from the surface (called “crazing” or “map cracking”). Unrestrained, thin slabs, such as concrete paving panels, may warp or curl due to non-uniform drying.

After the initial drying process occurs, the amount of moisture remaining in a concrete mass is proportional to the relative humidity of the surrounding air. A decrease in humidity causes cement paste within the concrete mass to lose moisture and shrink, while an increase will cause it to gain moisture and swell. These cycles produce alternating states of internal compressive and tensile stresses. Overall, these incremental volume changes and their resulting stresses are small and of little consequence for most concrete.

Thermal Expansion and Contraction

Concrete expands and contracts as temperatures rise and fall. The following equation is used to estimate the length change for a concrete mass resulting from a known temperature change:

$$\Delta L = \alpha L_1 (T_2 - T_1)$$

Where: ΔL = Estimated length change
 L_1 = Original concrete mass length in inches
 T_1 = Original concrete mass temperature (°F)



$$\begin{aligned} T_2 &= \text{Final concrete mass temperature } (^\circ\text{F}) \\ \alpha &= \text{Thermal expansion/contraction coefficient } (1/^\circ\text{F}) \end{aligned}$$

The magnitude of the thermal expansion/contraction coefficient is influenced by several factors including the type of aggregate, the amount of cementitious materials in the mix, the water-cementitious material ratio, the age of the concrete, and the relative humidity. Of these factors, aggregate type has the greatest influence.

Aggregate thermal expansion/contraction coefficients vary from $3.8 \times 10^{-6}/^\circ\text{F}$ for limestone to $6.6 \times 10^{-6}/^\circ\text{F}$ for quartz. Typically, a value of $5.5 \times 10^{-6}/^\circ\text{F}$ is used. For a 100-foot concrete span that has undergone a 77°F temperature change this equates to 1/2 inch change in length.

Durability

Durability refers to the ability of concrete to withstand the adverse effects of environmental weathering and transportation demands during its service life without serious deterioration. Durability involves the consideration of a number of related factors, such as resistance to weathering, abrasion, resistance to chemical attack, and physical deterioration.

Permeability can have a large impact on durability. Permeability is the rate (distance/time) that molecular-sized particles migrate through porous substances like concrete. Water and other liquids, liquid borne ionic compounds such as chlorides and sulfates, and gases such as carbon dioxide are examples of materials that permeate through concrete and react with concrete components in the process. For concrete, permeability is controlled by density of cementitious paste and porosity of aggregate, the ratio of paste to aggregate and the bond between paste and aggregate. As permeability increases, durability decreases.

Susceptibility of concrete and reinforcing steel to chemical attack is a function of design, cementitious materials, water-cementitious material ratio, admixtures, and curing processes. However, the most significant factor is the water to cementitious material ratio. When the water-cementitious material ratio is lower than 0.4, permeability is reduced, outer chemical attack threats are reduced, and durability is greatly enhanced. Chemical ions (primarily sulfates and chlorides) advance through concrete as a function of permeability, low permeability equates with more resistance to ion flow. Surface treatments that seal concrete surfaces can also be effective in reducing chemical caused deterioration. Silane has been shown to be an effective sealer in this regard.



Carbonation

Carbon dioxide and water begin combining with calcium hydroxide and to a lesser extent with calcium silica hydrates in concrete after it reaches final set. Although carbonation increases concrete strength, it also leads to corrosion of reinforcing steel. As hydration occurs in fresh concrete, hydroxides form as a byproduct. The presence of hydroxides raises the pH of concrete to 12. As the pH passes 11.5, reinforcing steel develops a passivating layer of iron that protects the steel from further corrosion. Carbonation reduces the alkalinity of concrete. When the pH drops below 11.5, the passivating layer is destroyed and corrosion can proceed.

Sulfates

The greatest danger from external sulfates occurs when sulfates and ground water are present. The sulfates usually are found in the form of magnesium, sodium, potassium or ammonium sulfate. Sulfates may also come from sulfuric acid produced by industrial operations or decay of organic matter. Sulfate ions will either cause expansion and cracking of concrete or deterioration of the concrete as the concrete loses bonding strength. External sulfates are countered by quality concrete with low water-cementitious material ratio and low permeability. In high sulfate environments, the concrete's water-cementitious material ratio should be no more than 0.4.

For 57 years the Portland Cement Association and the Division of Highways/Caltrans evaluated concrete embedded in a sulfate rich soil water basin maintained at Caltrans Laboratory facilities. (From 1940 until 1958 the water basin was at the laboratory complex on H Street. The laboratory complex moved to the current location on Folsom Boulevard in 1958.) Samples were placed in the sulfate rich soil so only the bottom half of each sample was covered. The basin was then flooded and allowed to dry. There were 10 to 15 wet dry cycles per year; the wetting and drying cycles in the sodium rich solution provided for the most severe sulfate exposure. Over the years the concrete variables tested included: water-cement ratios; cement content; air entrainment; cement fineness, varying cement chemistry within a type of cement; SCMs such as slag, fly ash, silica fume and calcined shale; and curing techniques such as water cure, stream curing and curing compounds.

After 57 years it was found that the reduction in water-cement ratios was “the most effective means of improving the performance of concrete in sulfate exposure conditions, regardless of cement type or composition” (Portland Cement Association’s “Concrete Technology Today”, David Stark, Volume 18, #2, July 1997). The type of cement and SCM constituency is of greater importance only when water-cement ratio exceeded 0.45. In those cases, Type V cement resists sulfate attack best because of the lower percentage of calcium aluminate. Also, Silane sealer proved to be effective in mitigating sulfate damage to concrete having water-cement ratios in the 0.45-0.55 range.

Chlorides

Chloride ions associated with deicing chemicals and salt water are carried by water as it permeates through concrete, eventually coming into contact with reinforcing steel. As ions reach the reinforcing steel, an electrochemical process, a corrosion cell will form and the steel will corrode into iron oxide. Rust occupies four to six times the volume of steel, resulting in expansive stresses on concrete and surface spalling and diminishing reinforcement cross-sectional area. A typical spall is shown in Figure 1-5.



Figure 1-5. Corrosive Girder Spalling.

When the chloride/water concentration in a concrete element reaches the corrosion threshold (the concentration at which corrosion activity can occur), a corrosion cell will be established along a bar of reinforcing steel. Corrosion begins as a single anodic area forms on the bar; the rest of the bar will be cathodic. As the chloride/water concentration increases, another anodic area will form, either on the same bar or on an adjacent bar. Development of anodic areas continues as the chloride concentration increases, until most of the steel is anodic and the entire structural element is filled with numerous, battery-like corrosion cells.

Early attempts at rehabilitation of corrosion damaged bridge decks were driven by the conventional wisdom of the time, which focused on the removal and replacement of all concrete in areas where visible and undersurface delamination were evident. This procedure left large areas of chloride-contaminated concrete still in place, and even though the repaired deck was covered with a protective membrane (and in some cases a concrete overlay) to inhibit further moisture penetration, corrosion, along with subsequent development of corrosion induced fracturing, continued, and for some structures, continues to this day. Corrosion persists in repaired concrete for two reasons. First, when an anodic area is disturbed by removing the chloride-contaminated, fractured concrete and replacing it with relatively salt-free concrete, the electric potential is reversed; that is, steel in the undisturbed portions of the deck, formerly cathodic, becomes anodic, whereas steel within newly patched areas, formerly anodic, becomes cathodic. As long as moisture and oxygen are present, the corrosion process will continue unabated. Second, the membrane systems used during the 1970s and early 1980s were not 100% impermeable.

The concrete overlays installed during the same time period also experienced chloride penetration. As a consequence, the barrier systems were not effective in preventing further chloride contamination. Of greater importance, however, is the fact that even with a totally impermeable deck protective system, moisture in amounts sufficient to fuel the corrosion process will migrate upward from the unprotected underside of the deck, or inward from the exposed lateral surfaces of the deck and girder stems.

Today, it is understood that the only certain cure for a chloride-contaminated structural element is total replacement. However, replacement is not currently a viable option for most of the structures where corrosion continues to occur. In view of this, it is evident that rehabilitation will remain a necessary maintenance effort for many years to come.

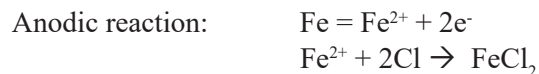
Corrosion is an electro-chemical process. For an electro-chemical cell to function, three elements are necessary: an anode, where corrosion takes place; a cathode, which does not corrode but which maintains the ionic balance of the corrosion reactions; and an electrolyte, which is a non-metallic solution capable of conducting an electric current by ionic flow. When the chloride concentration within concrete reaches the corrosion threshold (the concentration at which corrosion activity can occur), a corrosion cell will be established

along a bar of reinforcing steel. Initially, only a single anodic area will form on the bar; all of the other steel (on either side of the anodic area) will be cathodic. Subsequently, due to changes in moisture or chloride content, another anodic area will form, either on the same bar or on an adjacent bar. Development of anodic areas continues as the chloride concentration increases, until most of the steel is anodic and the entire element is filled with numerous, battery-like corrosion cells.

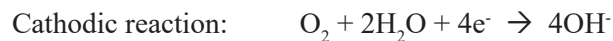
Concrete that is not contaminated with chlorine has an inherent resistance to corrosion attack because of its high pH value, caused by calcium hydroxide. The high pH inhibits the corrosion process, and is gradually neutralized by the presence of soluble chlorides. Once the corrosion threshold is reached, corrosion will occur if oxygen and moisture are present.

When a metal is placed in an electrolyte, a self-generated electro-chemical activity will occur if there is a potential difference between two areas of that metal. This activity (i.e., a flow of ions from the anodic to the cathodic areas of the metal) will result in corrosion at the anodic area. The process is not unlike the activity that occurs in a wet cell battery as it produces an electric current. That is, the potential difference between two electrodes in a battery, or between two areas along a single metal bar, causes a current to flow through the electrolyte from one electrode, or one area, to the other. In a battery the electrical circuit is completed through external physical connections; in a reinforcing steel bar, the circuit is completed through the bar itself.

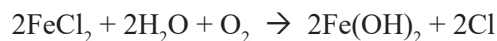
Iron, since it is relatively high in the electromotive force series, can easily enter into solution, which liberates electrons at the anode.



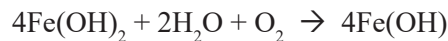
To maintain chemical equilibrium, electrons must be consumed at the cathode, and provided oxygen and moisture are present, hydroxides will be formed.



The free hydroxide replaces chlorine and ferrous hydroxide is deposited at the anodes, where it combines with water and oxygen to form ferric hydroxide.



Once liberated, chlorine ions are free to cycle around and attack additional iron molecules.





High quality aggregate and cement paste have the greatest effect on resistance to chloride intrusion. Aggregate particles should be sound (hard and strong) and of low porosity. The cement paste should be dense, watertight and completely embed the individual aggregate particles. Additionally, thorough mixing, optimum vibration to ensure maximum consolidation, and adequate curing are essential to the production of concrete that resists chlorides. Other standard preventive procedures include concrete cover (usually 2 - 2 1/2 inches or more in extreme exposure climates), low water-cement ratio (less than 0.40), use of supplementary cementitious materials such as silica fume that reduce permeability, and use of epoxy-coated reinforcement to keep chloride ions separate from reinforcement. Polyester concrete and similar topical coating overlays are also utilized to limit chloride-ion penetration.

Resistance to Freezing and Thawing

Of the many naturally occurring concrete disintegration mechanisms, repeated freeze-thaw cycles are the most severe. In normal concrete, both the hardened paste and the aggregate particles are vulnerable to the destructive effects of freeze-thaw cycles. If concrete is subjected to long periods of wet weather prior to freezing, it will almost certainly be damaged when freezing occurs. The cement paste is vulnerable because, when saturated paste is frozen, expansion of the free water in the paste produces a volume increase that exceeds the amount of expansion that the paste can withstand without damage. Porous aggregate particles may also absorb moisture. The absorbed moisture will produce expansive forces capable of bursting the aggregate particles if freezing occurs while the aggregate pores are filled (or nearly filled) with water. The rate of damage caused by concrete freeze-thaw cycling is directly related to the porosity of the cement paste.

Air-entrained concrete is used in areas subjected to freeze-thaw cycles. Air-entrained concrete contains billions of tiny air bubbles that are uniformly dispersed throughout the cement paste as the concrete is mixed. These air bubbles create a system of spherical air voids in the hardened concrete. The voids produced by entrained air reduce the permeability of the cement paste, which reduces the amount of free water present in the concrete. As the free water in the air-entrained concrete freezes, it expands through the void system and exerts pressure against the entrained air, which then compresses to relieve the pressure. When thawing occurs, the compressed air forces the water back into its original space. This process continues during repeated freeze-thaw cycles, providing long-term protection against deterioration. Entrained air protects only the cement paste; it provides no protection to the aggregate. Consequently, the ability of air-entrained concrete to resist freeze-thaw action will depend on the porosity of the aggregate particles and their degree of saturation. Typical before and after photos of freeze-thaw damage are shown in Figure 1.6.



Figure 1-6. Freeze-Thaw Damage, Before and After Photos³.

Alkali Aggregate Reaction

In some areas the native aggregate contains silica that chemically reacts with the sodium and potassium oxides, Na_2O and K_2O , found in cement. All cement specified for use on State highway projects meets the requirements for “low alkali” cement. Additionally, aggregates that have not received an innocuous rating will require the use of more SCMs than innocuous aggregates. Such aggregate is commonly called reactive aggregate, and the chemical reaction that occurs is known as the alkali-silica reaction (ASR), which results in a degenerative expansion of concrete. The alkali-silica reaction produces a gel that absorbs moisture and swells. The expansive swelling results in surface deterioration, cracking, spalling, and eventually complete disintegration of the concrete, as shown in Figure 1-7.

³ Investigation into Freezing-Thawing Durability of Low-Permeability Concrete with and without Air Entraining Agent, Kejin Wang, Gilson Lomboy, Robert Steffes, National Concrete Pavement Center, June 2009 (visited 9/10/10).



Figure 1-7. Concrete Deterioration Due to Alkali-Silica Reaction⁴.

A similar reaction occurring between dolomitic limestone and alkali, called an alkali-carbonate reaction, also results in expansive aggregate through the breakdown of dolomitic limestone aggregates. At this time, the potential of alkali-carbonate reactions are recognized, but not considered relevant to Portland cement concrete.

Thomas Edison Stanton, shown in Figure 1-8, Materials and Research Engineer for the State Division of Highways, is credited with discovering ASR in 1940. Stanton published a paper in the proceedings of the American Society of Civil Engineers⁵ stating that “excessive expansion of concrete may occur through chemical reactions between cements of relatively high alkali content and certain mineral constituents in some aggregates, such as shales, cherts, and impure limestones.” Stanton implemented the use of low alkali cements.

⁴ Reactive Solutions — An FHWA Alkali-Silica Reactivity News Publication Volume 3, Issue 3.

⁵ Stanton, T.E. (1940). “Expansion of concrete through reaction between cement and aggregate,” *Proceedings of the American Society of Civil Engineers*, Vol. 66, No. 10, pp. 1781-1811.



Figure 1-8. Thomas Stanton.

From 1940 to 1985 alkali-silica reaction was addressed by requiring low alkali cements. Alkali content was limited by specifications to cement that contained no more than 0.6% (by weight) of total alkali, calculated as Na_2O plus $0.658\text{K}_2\text{O}$. In 1955, structures built with reactive aggregate and low alkali cement were inspected and found to be in good shape. In 1958 the Division of Highway's Materials Engineer, Bailey Tremper, reported in the Highway Research Board Highway Research Report C-18 that the sole use of low-alkali cement to prevent reactive aggregate damages "was fully satisfactory judging from fairly comprehensive field surveys." By 1960 all cement used in highway facilities was required to be low-alkali cement. Shortly after, it was discovered that in the Bishop area the lower limit needed to be 0.3%; this was referred to as "Low-Low Alkali" cement. The highly reactive Bishop aggregate requiring 0.3% alkali was an exception, and the 0.6% limit was thought to be adequate for other concrete. Later low-low alkali cement became unavailable and concrete using these aggregates required 15% fly ash in addition to low-alkali cement. This however did not appear to be adequate either.

In 1985, the Simi Valley Freeway was rapidly deteriorating due to reactive aggregate. After field investigations, bridges in the Santa Barbara and San Luis Obispo area were determined to have reactive aggregate damage. A Statewide field study revealed more structures damaged by reactive aggregate. In 1986, ongoing projects were required to add 15% fly ash to the

cement content. Specifications were written so that all contracts required the aggregate to be tested per ASTM C 289 for reactivity or its potential. Any aggregate found to be reactive or potentially reactive would require 15% of cement to be replaced with fly ash. Aggregates known to be highly reactive required another 15% fly ash (30% total) by weight of the required cementitious content.

By 1992 as a result of a research project initiated at Caltrans Transportation Laboratory, it appeared the 15% replacement was not always adequate, sometimes even making the ASR more detrimental (see Figure 1-9). In 1996, when aggregate was found by ASTM C 289 to be potentially reactive, an additional 15% was required, making the total fly ash requirement 30%. Given the variability of results of ASTM C 289 for many aggregates the concrete industry proposed having a universal requirement for fly ash in all concrete. In 1997, specifications required all Caltrans cast-in-place concrete to be at least 25% fly ash. There were alternatives available in certain situations, such as 10% silica fume in corrosive environments or if the fly ash contained less than 2% calcium oxide, then only 15% fly ash was required.

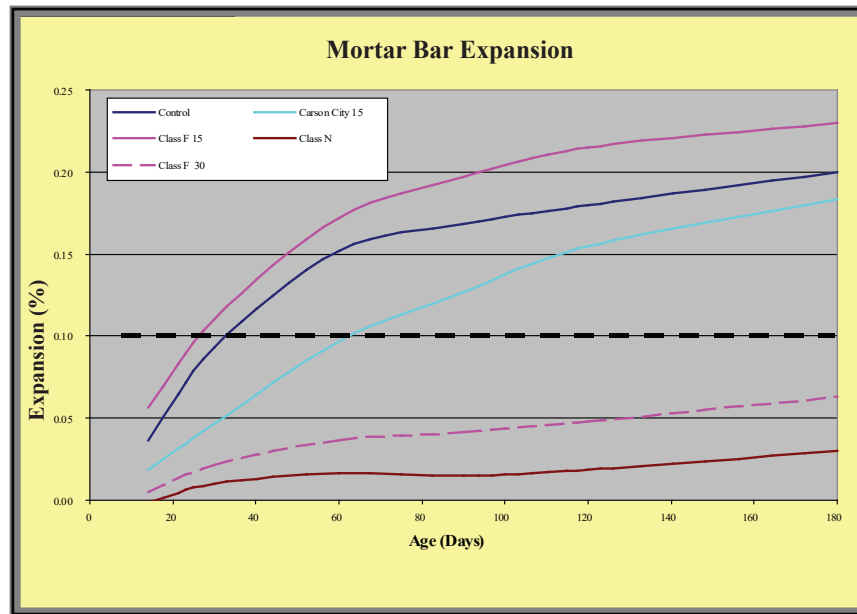


Figure 1-9. Mortar Bar Expansion Test Results.

Caltrans Materials Engineering and Testing Service (METS) currently maintains a source list of innocuous aggregates for concrete. Source testing must pass a combination of California Test 554 and either ASTM C 1260 or ASTM C 1293. For these aggregates the minimum amount of required fly ash is lower. Ground granulated blast furnace slag, with its wider use, is also acceptable as a solution to ASR mitigation.



Delayed Ettringite Formation

Ettringite is the common name for hexacalcium aluminate trisulfate hydrate, $(\text{CaO})_6(\text{Al}_2\text{O}_3)(\text{CSO}_3)_3 \times 32\text{H}_2\text{O}$. Ettringite typically forms during the hardening phase within all concrete. (In cement chemistry the notation would be $\text{C}_6\text{A}_3\text{S}_3\text{H}_{32}$ where C stands for CaO, A for Al_2O_3 , S for SO_3 , and H for H_2O .) Since it normally forms during the hardening process there are no disintegrative forces on the concrete. But at high curing temperatures ettringite formation is inhibited. Gradually, after the hardening, the molecules combine to form the ettringite, but when the formation is delayed, the resulting crystal exerts pressure within the concrete shape and causes disintegrative cracking. This phenomenon is called Delayed Ettringite Formation or DEF.

The only agreed upon way to prevent DEF is to avoid high curing temperatures. The temperature limit varies between agencies. At present, Caltrans uses 160°F as the upper limit.

Abrasion

Naturally occurring abrasive elements include wind, flowing water, floating ice, and debris. Man-made abrasive sources include moving traffic, snow plows, and tire chains. Abrasive forces can wear away the cement and aggregate particles or dislodge individual aggregate particles from the concrete. The water-cement ratio, mixing, placing and finishing techniques, and curing conditions all have a greater influence on resistance to abrasion than does aggregate quality. Soft aggregates require more cement to reduce wear, while hard aggregates require just enough cement to secure the aggregate in place against the abrasive action.



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