# HIGH-PERFORMANCE CONCRETE

Transportation Operations and Systems Research and Development Partnership
Agreement No. 359629
Work Order 3

PB2000-105208

FINAL REPORT

Prepared for

Commonwealth of Pennsylvania
Department of Transportation

By

Dr. Paul J. Tikalsky
and
Dr. Andrew Scanlon

The Pennsylvania Transportation Institute
The Pennsylvania State University
Transportation Research Building
University Park, PA 16802-4710

March 2000

This work was sponsored by the Pennsylvania Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of either the Federal Highway Administration, U.S. Department of Transportation, or the Commonwealth of Pennsylvania at the time of publication. This report does not constitute a standard, specification, or regulation.

PTI 2K16

Technical Report Documentation Page

| 1. Report No. | 2. Government Accession No. | 3. Recipient's Catalog No. |
|---|---|---|

| 4. Title and Subtitle | 5. Report Date |
|---|---|
| High-Performance Concrete | March 2000 |
| | 6. Performing Organization Code |
| 7. Author(s)<br>Paul J. Tikalsky and Andrew Scanlon | 8. Performing Organization Report No.<br>2K16 |
| 9. Performing Organization Name and Address<br>The Pennsylvania Transportation Institute<br>Transportation Research Building<br>The Pennsylvania State University<br>University Park, PA 16802-4710 | 10. Work Unit No. (TRAIS)<br><br>11. Contract or Grant No. 359629 |
| 12. Sponsoring Agency Name and Address<br>The Pennsylvania Department of Transportation<br>Bureau of Planning and Research<br>555 Walnut Street, 6th Floor Forum Place<br>Harrisburg, PA 17101-1900 | 13. Type of Report and Period Covered<br>Final Report<br><br>14. Sponsoring Agency Code |

**15. Supplementary Notes**
COTR:




**16. Abstract**

The primary goal of this research project was to evaluate PennDOT's current concrete mixture designs for performance characteristics and provide specific recommendations on the effective use of concrete with high-performance characteristics. Highway concrete mixtures in Pennsylvania are largely designed for strengths between 23 and 31 MPa (3,300 and 4,500 psi) and for resistance to freezing and thawing. While strength and freeze-thaw resistance are important in Pennsylvania, other parameters impact the long-term performance of concrete in highway applications. Concrete can be developed to address economic considerations, as well as multiple combinations of strength, permeability, modulus, cracking tendency, abrasion resistance, freeze-thaw resistance, alkali-aggregate reaction, internal and external sulfate attack, workability, construction scheduling, traffic openings, or other criteria.

The report defines HPC in the context of the Pennsylvania Department of Transportation; describes the characteristics and benefits derived from the use of HPC; evaluates the current state of the practice in Pennsylvania; and identifies the performance criteria that benefit PennDOT bridges, structures, and concrete pavements. It also provides a series of recommendations for consideration for the Commonwealth of Pennsylvania.


| 17. Key Words | 18. Distribution Statement |
|---|---|
| high-performance concrete, concrete mixtures, concrete strength, concrete durability | No restrictions. This document is available from the National Technical Information Service, Springfield, VA 22161 |

| 19. Security Classif. (of this report) | 20. Security Classif. (of this page) | 21. No. of Pages | 22. Price |
|---|---|---|---|
| Unclassified | Unclassified | 67 | |

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

# TABLE OF CONTENTS

## TABLE OF CONTENTS (Continued)

# LIST OF FIGURES



# LIST OF TABLES

# 1. INTRODUCTION

Transportation agencies across the United States invest more than $5 billion on concrete bridge repair and renovation annually. These projects not only consume resources, but cause significant delays for the motoring public on highways and roads. The important variables influencing bridge and pavement performance must be controlled and studied in order to optimize factors that result in the most durable bridges and pavements. The Pennsylvania Department of Transportation (PennDOT) is in a unique position to implement high performance technology and to create a database of long-term service-life characteristics.

Several states have conducted initial work to implement high performance concrete (HPC). HPC showcases have been held in Texas, Nebraska, New Hampshire, Washington, Colorado, Ohio, Virginia and Florida. These showcases have demonstrated the use of long-span concrete bridges, high-strength girders and piers, new structural shapes for structural members, and early opening techniques. Using these documented experiences to improve design in Pennsylvania and extend HPC technology by prolonging the life of the infrastructure is the next logical step. Other states will benefit from Pennsylvania's leadership in demonstrating the durability aspects of HPC, and Pennsylvania will benefit from the advances of other states.

As part of its commitment to improve the cost effectiveness of the highway infrastructure in Pennsylvania, PennDOT seeks cost-effective strategies for using high-performance concrete in bridges and structures. The primary goal of this research project was to evaluate PennDOT's current concrete mixture designs for performance characteristics and provide specific recommendations on the effective use of concrete with high-performance characteristics. Highway concrete mixtures in Pennsylvania are largely designed for strengths between 23 and 31 MPa (3,300 and 4,500 psi) and for resistance to freezing and thawing. While strength and freeze-thaw resistance are important in Pennsylvania, other parameters impact the long-term performance of concrete in highway applications. High-performance concrete refers to concrete mixtures that are developed by considering the total performance of the concrete. These concrete mixtures determine what can reasonably be designed, constructed, and maintained under service considerations.

Several examples illustrate the benefits that can be obtained from using high performance concrete:

- Concrete with a compressive strength between 55 and 90 MPa (8,000 and 13,000 psi) is not currently used in design for highway applications, but could be used to reduce the dead load and number of girders used in concrete bridges. This would allow designers and prestressed concrete manufacturers to use beams in excess of the current limit of 45 meters (150 ft) and subsequently compete with steel bridges for long-span bridges.
- Low-permeability concrete can be used to reduce the susceptibility of pavements and bridge structures to corrosion, scaling, freeze-thaw deterioration, and sulfate attack, thus increasing the life expectancy of the infrastructure.
- Rheological properties can be optimized to improve construction operations and avoid problems with congestion and honeycombing.
- Heat of hydration, maturity, and shrinkage properties can be optimized to reduce cracking and subsequent deterioration in pavements and bridge decks, and increase the efficiency of prestressed girders by reducing losses in the prestressing force.

Performance-based concrete specifications identify the desired engineering and construction properties, while allowing the concrete industry to optimize mixture designs. Concrete can be developed to address economic considerations, as well as multiple combinations of strength, permeability, modulus, cracking tendency, abrasion resistance, freeze-thaw resistance, alkali-aggregate reaction, internal and external sulfate attack, workability, construction scheduling, traffic openings, or other criteria.

Chapter 2 of this research report defines HPC in the context of the Pennsylvania Department of Transportation. The characteristics and benefits derived from the use of HPC are described in Chapter 3. Chapter 4 evaluates the current state of the practice in Pennsylvania and identifies the performance criteria that benefit PennDOT bridges, structures, and concrete pavements. Finally, Chapter 5 provides a series of recommendations for consideration for the Commonwealth of Pennsylvania.

# 2. DEFINING HIGH PERFORMANCE CONCRETE

There is often confusion over the definition of high-performance concrete. The definition is dependent on the state of the concrete industry in any particular geographic area and expectations of the specifying agencies. What is considered conventional concrete in some areas may be considered high-performance concrete in other areas. For high performance concrete, the desired properties of concrete must be clearly specified, delivered, and measured. Normal weight and strength concrete manufactured with conventional processes for transportation applications is expected to be transported, cast and consolidated with manual and mechanical equipment. It is expected to carry factored structural loads to 28 MPa (4,000 psi) and generally to resist freezing and thawing environments.

Any concrete that satisfies criteria to overcome the limitations of conventional concretes may be referred to as high performance concrete (HPC). HPC may include concrete that benefits the construction process (e.g., reduces construction time to permit rapid opening of roads or bridges without compromising service life performance); provides substantially improved resistance to environmental influences (e.g., reduced permeability to resist the ingress of chemical species); or features improved mechanical properties for structural applications (e.g., high strength to reduce section size or span length). Therefore, it is not possible to define HPC without considering the performance requirements of the intended use of the concrete. HPC uniquely defines the structural elements' intended use in construction, and their resilience to deterioration and environmental extremes.

Forster (1994) defined HPC as "a concrete made with appropriate materials combined according to a selected mixture design and properly mixed, transported, placed, consolidated, and cured so that the resulting concrete will give excellent performance in the structure in which it will be exposed, and with the loads to which it will be subjected for its design life." The American Concrete Institute's (ACI) task force on high performance concrete refined that definition further to the following two statements (Russell 1999):

**High performance concrete** is concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices.

**A high-performance concrete element** may be defined as that which is designed to give optimized performance characteristics for a given set of load, usage and exposure conditions consistent with the requirements of cost, service life, and durability.

The definitions are clear statements to guide the general understanding of high performance concrete for the Pennsylvania Department of Transportation. To further refine the definition for engineering applications, a concrete should be considered a high-performance concrete if it incorporates specific engineering characteristics for a particular application and environment. The engineered concrete properties can be improved to benefit three major areas:

1) the construction process,
2) the mechanical properties of concrete elements, and
3) the durability of concrete structures.

Properties that improve the construction process improve the quality of the constructed material, overcome a particular difficulty in casting concrete, or expedite the construction. The improvement of mechanical properties can be used to optimize structural sections or systems for loadings or serviceability requirements. The increased durability of concrete impacts the life expectancy and life cycle cost and the maintenance of the transportation infrastructure. A summary of the benefits is provided in Table 1.

The Pennsylvania transportation infrastructure offers a unique subset of HPC properties. The environment in which concrete structures and pavements exist is varied, but it is considerably harsher than that for most building applications. Table 2 provides a list of typical environmental factors experienced in Pennsylvania.

**Table 1. Benefits from high-performance concrete.**

| BENEFITS TO CONSTRUCTION PROCESS | | |
|---|---|---|
| •Flowing/ nonsegregating<br>congested areas<br>vibration and compaction are restricted<br>defect-free surfaces | •Early age strength<br>high early strength,<br>very early high strength | •Decreased variability<br>•Consistency<br>setting time |
| •Pumpable<br>low friction losses<br>nonsegregating<br>uniform discharge | •Reduction of fluid forces | •Curing procedures<br>•Materials<br>resources<br>time |
| •Early form removal | •Improved sawing procedures | •Improved finishing procedures |

| BENEFITS TO MECHANICAL PROPERTIES | | |
|---|---|---|
| •Higher strength<br>•Reduction of dead load<br>•Optimize prestressing<br>•Extending span length | •Higher or lower elastic modulus | •Reduced strain<br>•Creep<br>•Shrinkage |
| •Increased shear strength | •Improved fatigue resistance | •Increased toughness |
| •Increased abrasion or surface hardness | •Reduced thermal expansion | |
| **BENEFITS TO DURABILITY** | | |
| •Reduced permeability, chloride penetration, and sulfate intrusion, and level of water saturation | •Reduced cracking potential,<br>heat of hydration, tensile strain,<br>drying shrinkage, and autogenous shrinkage | •Chemical resistance<br>•Sulfate attack<br>•Alkali silica reaction<br>•Delayed ettringite<br>•Acid attack |
| •Reduction in steel corrosion | •Increased freeze-thaw resistance | •Reduced shrinkage |
| •Improved fire resistance | •Physical Resistance to abrasion/erosion<br>scaling resistance | •Lower or higher density, |

**Table 2. Environmental conditions in Pennsylvania.**

| Environmental Condition | Typical Range in Pennsylvania |
|---|---|
| Temperature* | -12° to +31°C |
| Humidity* | 28% to 88% |
| Precipitation* | 0.9 – 1.1 meter/year |
| Freeze-Thaw** | 10 to 30 cycles per year |
| Deicing Salts** | 5.6-8.4 metric tons/lane km |
| Sulfate Exposures** | 0.0-0.2 percent $SO_4$ |

*see appendix C

** *Estimated from prior research and reported data to PennDOT.*

# 3. CHARACTERISTICS OF HIGH-PERFORMANCE CONCRETE

For the purposes of the transportation uses of concrete in Pennsylvania, high performance can be further defined into specific performance characteristics. Table 3 defines 13 specific characteristics and performance grades for concrete used in the transportation infrastructure in Pennsylvania. Each characteristic has three grades of performance that can be specified, depending on the anticipated exposure and desired design life. Grade I represents good-quality concrete for transportation applications. The higher grades represent higher performance levels to increase the life of the highway infrastructure, reduce maintenance, or optimize structural systems. The high- performance concrete requires contractors and engineers to use advanced knowledge of concrete materials, performance, and quality control.

Each of these performance characteristics must consider the technical, economic, and practical ability of a contractor to attain the desired HPC properties. Because many characteristics of high performance concrete are interrelated, a change in one characteristic usually results in changes in one or more of the other characteristics. Consequently, if several performance characteristics have to be taken into account in producing concrete for an intended application, each of these characteristics must be clearly specified in the contract documents and cannot be contrary to the other characteristics, e.g., high strength and low modulus.

Each of the characteristics in Table 3 benefits certain components of the highway infrastructure. However, the specification of a performance grade requires that engineers and contractors understand the exposure conditions and the potential deleterious effects. It is not necessary to specify the performance grade of all characteristics or all types of potential deterioration.

When considering a 10-meter-tall concrete pier for a 30-meter-overpass, the designer might specify minimum compressive strength, resistance to alkali-silica reaction, permeability, freeze-thaw resistance, workability, and, if the surrounding soil or water contains moderate levels of soluble sulfates, sulfate resistance. The designer does not need to specify modulus of elasticity,

**Table 3. Classes of performance for high-performance concrete.**

| Performance Characteristics | Standard Test Method | 3. Proposed HPC performance grade | | |
|---|---|---|---|---|
| | | 1 | 2 | 3 |
| Freeze-thaw durability (relative modulus after 300 cycles) | AASHTO T161 Proc. A | $60\% \le X \le 80\%$ | $80\% \le X \le 90\%$ | $90\% \le X$ |
| Scaling resistance (visual rating after 50 cycles) | ASTM C 672 | $X=2,3$ | $X=1$ | $X=0$ |
| Abrasion (wear in mm) | ASTM C994 | $2.0>X\ge 1.0$ | $1.0>X\ge 0.5$ | $0.5>X$ |
| Alkali-silica reaction (expansion) | AASHTO T303 | $X<0.10\%$ At 14 Days | | |
| | ASTM C441 | | $X<0.10\%$ At 56 Days | $X<0.05\%$ At 56 days |
| Chloride penetration (coulombs) | AASHTO T 277 | $4000\ge X>2500$ | $2500\ge X>1500$ | $1500 \ge X$ |
| Compressive strength MPa (ksi) | AASHTO T 22 | $24\le X<32$ $(3.5\le X<4.6)$ | $32\le X<55$ $(4.6\le X<8.0)$ | $55\le X<82$ $(8.0\le X<12.0)$ |
| Strength ratio $\frac{28\ \text{day}\ f_c}{7\ \text{day}\ f_c}$ | AASHTO T 22 | 1.15 | 1.25 | 1.40 |
| Elastic modulus GPa (Msi) | ASTM C 469 | $20\le X<30$ $(2.9\le X<4.3)$ | $30\le X<45$ $(4.3\le X<6.5)$ | $45\le X$ $(6.5\le X)$ |
| Shrinkage (microstrain) | ASTM C 157 | $800>X\ge 500$ | $500\ge X200$ | $200\ge X$ |
| Sulfate resistance (expansion) | ASTM C1012 | $X<0.10\%$ At 6 months | $X<0.10\%$ At 12 months | $X<0.10\%$ At 18 months |
| Tensile strength MPa (psi) | ASTM C78 | $4>X\ge 5$ $(580\le X<720)$ | $5>X\ge 6$ $(720\le X<870)$ | $X>6$ $(870\le X)$ |
| Workability $X$ = mm (in.) | ASTM C143 | $50>X\ge 125$ $(2>X>5)$ | $125>X\ge 200$ $(5>X>8)$ | $200<X$ $(8<X)$ |
| Creep coefficient $\nu$ (strain/strain) | ASTM C 512 | $3.0\ge \nu>2.0$ | $2.0\ge \nu>1.4$ | $1.4\ge \nu$ |

tensile strength, shrinkage, scaling resistance, or abrasion resistance. In such a case, the concrete producer would provide the latest test values for the specified performance values to the engineer for review and approval before the mixture is used. In the quality control and quality assurance programs, the minimum compressive strength and strength ratio (28-day: 7-day) may be specified for approval and payment purposes.

This type of performance specification allows contractors to develop concrete mixtures that meet PennDOT's needs, yet optimize materials for economic and competitive reasons. To make these types of evaluations, engineers and contractors must understand the nature of the deterioration mechanisms and the means by which deterioration can be avoided or mitigated.

A detailed discussion of performance characteristics is necessary to fully explain the performance grades and the potential benefits for the highway infrastructure. The succeeding discussion summarizes the mechanisms of deterioration and the nature of the concrete properties for the reader. It includes an explanation of conditions under which an engineer would specify a particular grade of performance.

## 3.1. FREEZE-THAW RESISTANCE

Concrete structures exposed to freezing and thawing cycles are susceptible to a slow progressive form of deterioration. All concrete contains enough water to freeze during cold weather. Water expands during the transition from liquid to solid. The volumetric increase is more than 9 percent. Pore water in the capillary spaces and free water around the aggregates expand and generate stresses in excess of 220 Mpa, well beyond the tensile or compressive strength of concrete. A small amount of freezable water (pore water) is enough to damage concrete. Concrete that is saturated during freezing cycles is likely to undergo severe deterioration and contribute to the acceleration of other forms of deterioration (corrosion, alkali-silica reactivity [ASR], sulfate).

The resistance to freezing and thawing cycles is obtained by casting concrete that has a uniformly distributed air void structure in the paste portion of the concrete. The air void structure provides space for the solid water to occupy during the freezing cycles. The proximity of air is important because freezing water cannot travel through capillary voids in the hardened paste any great distance as compared to the quantity of freezing water. The entrained air bubbles, 0.1 to 1-mm in diameter, must be closely spaced together and non-interconnected. The average spacing factor of bubbles must be less than 0.20 mm (0.008 inches), as defined by American Society for Testing and Materials (ASTM) C457. While concrete with spacing factors as high as 0.25 mm has been shown to be resistant to freeze-thaw environments, the performance data is not as definitive as with the lower factors.

The system of air in concrete is affected by the mixture constituents, as well as by the mixing and transporting of the concrete. In lean concrete mixtures, the sand gradation affects the air-void structure. Very fine sand particles decrease the quantity of entrained air, while increased particles 0.5 to 4.75 mm in diameter tend to increase the air content. This same effect is seen in the quantity of cementitious material. Increasing amounts of cementitious material and using fine cementitious material decreases the air content of concrete. Increased water contents tend to increase air contents, but not entrained air contents. The excess water forms pockets of water beneath aggregate particles, which eventually become entrapped air voids. Chemical admixtures typically change the dosage of air-entraining agent needed to obtain specific air content. Water reducers, retarders, and accelerators typically increase the air content, while some high-range water reducers and other admixtures may decrease the air content. The most common air-entraining agent is neutralized Vinsol™ resin, an agent has been used for more than 50 years. However, synthetic air entraining agents have been developed in the past 15 years. Each of these chemicals is different and interacts with chemical admixtures and fine material differently.

A higher performance characteristic for freezing and thawing resistance is primarily needed in concrete that may be saturated during the freezing cycle. The highest level of performance should be used when the concrete is saturated and undergoes repeated freeze and thaw cycles. Freeze-thaw resistance is measured in terms of the relative dynamic modulus of elasticity after

300 cycles of freeze and thaw (AASHTO T161). Grade 1 in Table 3 is for a relative modulus of elasticity less than 80 percent; if it is between 80 percent and 90 percent, the freeze thaw resistance is grade 2. For grade 3, the relative dynamic modulus of elasticity is greater than 90 percent. Grade 1 applies to structural members exposed to freeze-thaw environments in an unsaturated condition (e.g., bridge girders). Grade 2 applies to members that are periodically saturated during freezing (e.g., footings, railings, elevated piers). Grade 3 applies to concrete that is typically saturated during freezing (e.g., pavements, bridge decks, piers at the water line).

Improved resistance is obtained by increasing the entrained air content, maintaining a spacing factor below 0.20 mm, slightly reducing the water cement ratio in the concrete mixture, and thoroughly mixing the concrete.

## 3.2. SCALING RESISTANCE

Deicing salt is applied to road surfaces to reduce the incidence of accidents by delaying the formation of ice and snow on the highway surface. Due to the presence of salt on bridge decks and pavements, concrete members that are exposed to salt are subjected to the forces of surface scaling. Surface scaling is caused by several mechanisms, but is primarily due to the porosity of the surface layer and differential stresses caused by the unfrozen surface water and frozen subsurface water. The scaling resistance of concrete is affected by a variety of variables. Early finishing, excessive bleeding, overworking the surface during finishing, lack of curing, early exposure to freezing temperatures or carbon monoxide, and exposure to deicing salts are the major contributing factors to scaling. The early finishing or overworking of the plastic concrete surface of concrete prevents bleed water from reaching the surface and increases the w/cm ratio and porosity of the concrete immediately below the surface. Excessive bleeding increases the porosity of the surface concrete and allows deicing salts to enter through the bleed water channels, increasing the depth of chloride penetration. Exposure of the concrete to freezing temperatures or fumes from gas heaters destroys the integrity of the surface during the early days of hydration. Exposure of concrete to deicing salts in the first 6 months after placement decreases the durability of the concrete by exposing the concrete to the formation of chloroaluminate salts in the surface layer when the concrete is most susceptible to chloride penetration. The scaling of concrete also accelerates other forms of deterioration by weakening the surface layer and allowing other detrimental species to enter the concrete.

Scaling is measured in terms of a visual rating of the surface after 50 cycles or the concrete in service. This is detailed in ASTM C672. If the visual rating of the surface is poor, the scale is either 4 or 5. This is grade 1 performance in Table 3 and is primarily for concrete that is not part of the highway driving surface and not exposed to deicing salts (e.g., bridge girders, piers, foundations). Grade 2 is a class of concrete that is exposed to either abrasive forces or concentrations of deicing salts (e.g., pier caps, railings, box culverts). Surfaces exposed to concentrations of deicing salts and abrasive forces should be designed for maximum resistance to scaling. Grade 3 should be used to protect highway surfaces and areas that will be exposed to heavy concentrations of salts.

Improved resistance of concrete to scaling is obtained by reducing the water-cement ratio in the concrete, air entraining, reducing the concrete's permeability, and moist curing it. Curing conditions are very important in developing a scale-resistant concrete pavement or bridge deck. The contractor should provide a moist environment for the concrete that is greater than the evaporation rate for the first 7 days (10 days for bridge decks). For example, for an 80°F day with 50 percent relative humidity, a wind speed of 12 MPH, and a concrete temperature of 88°F, the contractor would have to provide at least 0.25 lbs/ft$^2$/hr of moisture for curing. If the contractor provided a wind break to 0 MPH wind speed, the requirement would drop to 0.05 lbs/ft$^2$/hr (see Figure 1).

Reducing the potential level of saturation at freezing is also important and relates to the need for low-porosity concrete. Finishing operations and sealing operations should not be conducted too early. The addition of silica fume or other pozzolans will typically benefit the scaling resistance.

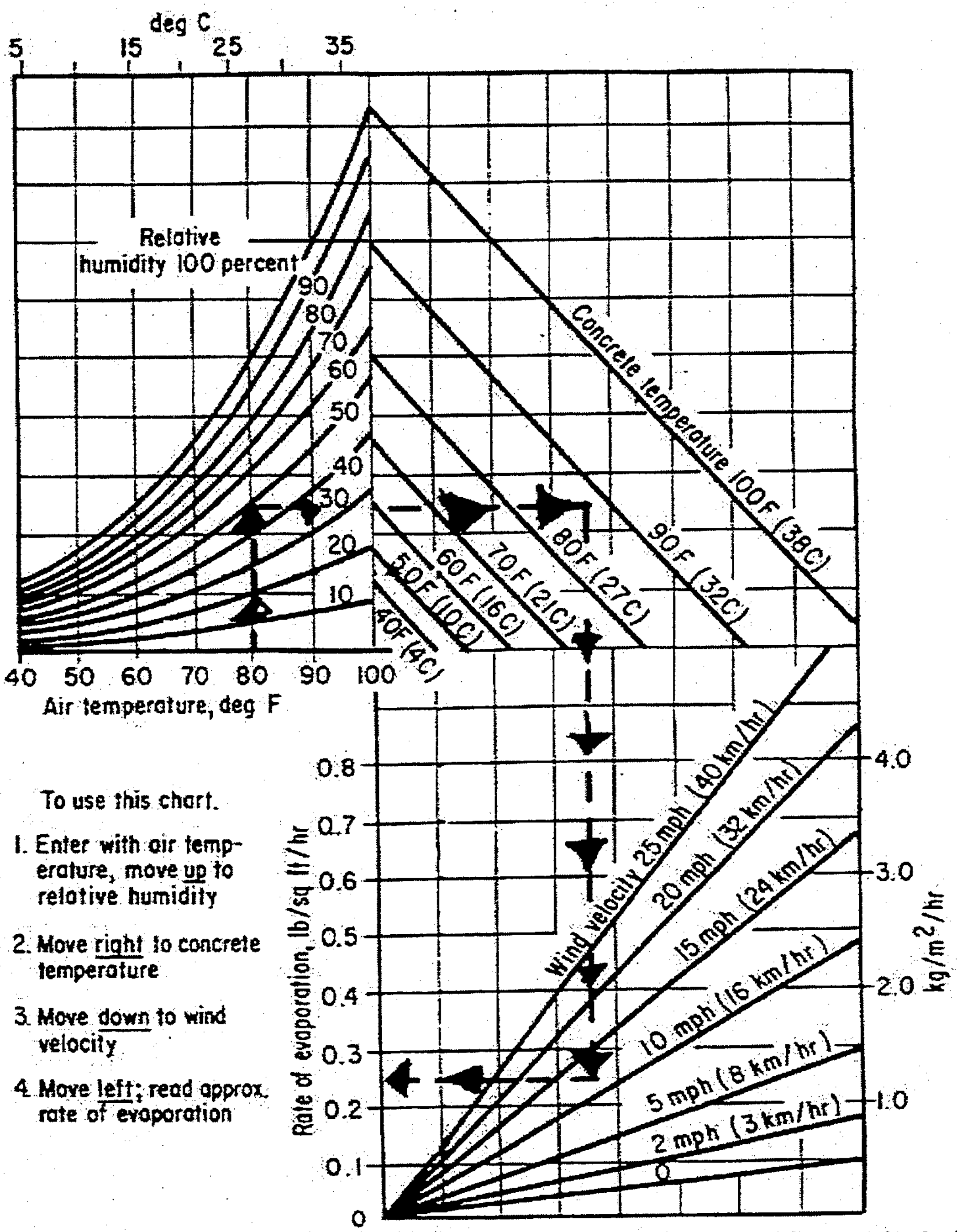


**Figure 1. Rate of evaporation chart for concrete construction.**

## 3.3. ABRASION RESISTANCE

Erosion of concrete surfaces subjected to acceleration and deceleration of heavy vehicles, chains, tire studs, and hydraulic scour is the primary reason for considering abrasion-resistant concrete. Abrasion-resistant concrete can preserve road surfaces, bridge decks, piers, and culverts. The combination of attrition, scraping and percussion loadings, known as abrasion, is resisted by complex strength interactions between aggregate and cementitious paste. Abrasion problems are often associated with soft aggregates, low-strength, weakened surfaces from inadequate curing and finishing, or the over manipulation of plastic concrete.

To improve abrasion resistance, the concrete aggregate should be hard and the compressive strength should be specified at a higher level. Siliceous aggregates are typically the most resistant to abrasion, whereas limestone and granite provide moderate resistance, and lightweight aggregate and blast furnace slag provide the least resistance. Curing and finishing have a large impact on the abrasion resistance. Wood and magnesium floating tears the concrete microsurface, allowing it to bleed; as a result, the surface is not abrasion resistant. Steel floating or a hard trowel finish is required to harden the surface. Abrasion resistance and skid resistance are not easily accommodated simultaneously. Using a siliceous sand and/or coarse aggregate, a low w/cm ratio to strengthen the concrete matrix, and extended curing is the best way to provide abrasion resistance for pavement and bridge surfaces, while finishing techniques may be better for hydraulic structures.

Abrasion resistance is measured in terms of the average depth of wear. For Table 3, grade 1 abrasion resistance, the concrete mixture can be optimized using greater coarse aggregate proportions and coarse aggregates that meet minimum levels of LA abrasion resistance (e.g., pavements and bridge decks). For a higher resistance, grade 2-abrasion resistance can be obtained by developing mixture designs using moderately hard aggregates with w/cm ratio at or below 0.40 (e.g., acceleration and deceleration zones and tidal zones). For grade 3-abrasion resistance, hardened aggregates must be used with a hardened trowel finish (cavitation zones, areas where chains and studded tires are used).

## 3.4 ALKALI-SILICA REACTION RESISTANCE

Alkali-silica reactivity (ASR) is often linked with an insidious cracking of the concrete. The cracking is the result of a volumetric change in the concrete through the imbibing of water by alkali silica gel. Certain reactive forms of silica (e.g., opal, chalcedony within chert, tridymite and crystobalite, strained quartz, and silica-based glass) react with alkali metals and hydroxyl ions to form a gel that has the ability to imbibe water. The gel grows into voids and provides internal pressures in cracked concrete. Understanding the formation and growth of the gel is beyond the scope of this report, but the gel is a complex matrix of amorphous alkali silicate hydrate that forms through a diffusion-controlled reaction. External or pore water is imbibed through osmotic forces. It is widely understood that a relative humidity greater than 80 percent will provide enough external water to drive the swelling of the gel. ASR can be avoided by using non-reactive aggregates, using low-alkali portland cements, low-calcium fly ash, silica fume or ground granulated blast furnace slag, or lithium admixtures. Non-reactive aggregates and low-alkali portland cements are not widely available in some markets. However, the use of appropriate proportions of fly ash, ground granulated blast furnace slag, and silica fume is an inexpensive solution that has a long service record. These supplemental cementitious materials reduce the total mass of soluble alkalis and assist in forming non-swelling gels.

ASR resistance is measured using the AASHTO T303 and ASTM C441 methods in Table 3. Grade 1 concrete has ASR of 0.1percent after 14 days in AASHTO T303, or 0.15 percent in ASTM C441 (beams not exposed to moisture). Grade 2 concrete should meet a higher level of performance—0.10 percent in ASTM C441 (e.g., in walls, slabs and beams exposed to moisture and a known reactive aggregate). Grade 3 concrete performance should be used for extended life structures (greater than 50 years) with known reactive aggregates. More comprehensive guidelines are provided in the proposed "AASHTO Guide Specification for Highway Construction–Portland Cement Concrete Resistance to Excessive Expansion Caused by Alkali-Silica Reaction," attached in Appendix A.

The most effective means to diminish the effect of ASR is to use 20-30 percent low-calcium fly ash, 35-50 percent ground granulated blast furnace slag, or 5-10 percent silica fume or metakaolin. The use of alternative cementitious materials provides several mitigating effects. They reduce the total of soluble alkalis from the portland cement, reduce the total calcium hydroxide in the hardened paste, reduce permeability, and incorporate alkalis into the cementitious compounds. If reactive aggregates are used, keeping the alkalis below 2.5 kg/m$^3$ for elements not directly exposed to water, or below 1.8 kg/m$^3$ for structures directly exposed to water will also provide suitable ASR resistance.

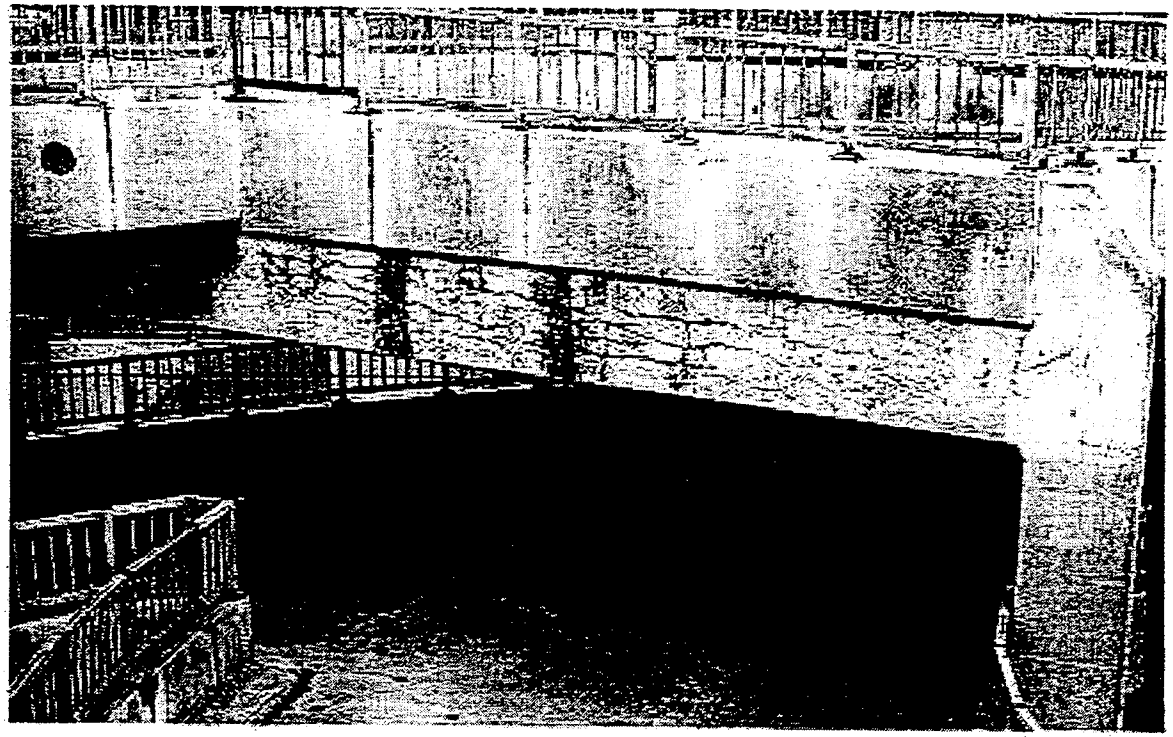

**Figure 2. Alkali silica reaction damage on overpass.**

## 3.5. CHLORIDE PENETRATION RESISTANCE

Degradation of concrete reinforcement and the concrete surface layer is accelerated by chloride penetration. Chloride ions participate in the electrochemical process of steel corrosion by destroying the passivating layer of steel in high pH environments and by increasing the strength of the electrolytic pore water solution. Concrete is normally an excellent passivating material because of its high concentration of hydroxyl ions, high pH, and its dense protective nature. However, when the pore water adjacent to the reinforcing steel becomes concentrated with chloride ions, the oxidation-reduction reactions of steel corrosion are no longer inhibited. The result is corrosion of the reinforcing steel. Chlorides also prevent the water near the surface of concrete from freezing at 0°C. The chloride salts have multiple effects on the surface layer of the concrete. They lower the freezing temperature of water by as much as 5-10°C to prevent the formation of ice in moderate winter temperatures. In lowering the temperature, chloride salts also encourage the saturation of the surface layer of concrete before freezing occurs. The saturated layer will be subjected to a more severe freeze-thaw cycle than an unsaturated concrete and potentially deteriorate at a faster rate. In addition, the stress gradient between the frozen water in the base concrete and the unfrozen surface layer provides an interface that is susceptible to scaling action.

Chloride penetration can be measured by coring and conducting extraction techniques to determine the chloride concentrations. However, this is must be done over a long period of time (5 to 50 years) and is not practical for specifying concrete mixtures. The rapid chloride permeability test, AASHTO T277, is a measure of the mobility of chlorides and the potential for chloride penetration. The amount of the electric charge that passes through the concrete element in a specified time is expressed in coulombs. This test has a successful history in plain concrete and in concrete that contains air entraining agents and water reducers. However, nitrite- and nitrate-based admixtures have been shown to interfere with the measurement and therefore should not be used with these types of admixtures. For HPC grade 1, the value of the charge passing through the concrete is between 2500- 4000 coulombs. This is less than what is typically delivered by current PennDOT specifications, but can be easily obtained by ready mix concrete producers throughout Pennsylvania. Grade 1 performance should be the minimum level specified for all exposed reinforced concrete. For concrete exposed to directly to deicing salts (e.g., pavements and curbs), and for reinforced concrete indirectly exposed to deicing salts (e.g., pier caps and wing walls), grade 2 levels should be specified. This is a rapid chloride permeability between 1500- 2500 coulombs. The highest level of performance, grade 3, should be specified for reinforced concrete elements directly exposed to deicing salts (bridge decks) or indirectly exposed to concentrated salt solutions (culverts, drainage structures). The level of rapid chloride permeability for grade 3 is less than 1000 coulombs.

Decreasing the water- cementitious materials ratio and using blends of cementitious materials result in a decreased diffusion coefficient for a given concrete. Partial substitution of portland cement by ground granulated blast furnace slag, fly ash, metakaolin, or silica fume provides the most effective means of reducing the permeability and diffusion coefficient. Addition of a small amount of a superplasticizer also has been shown to decrease the diffusion coefficient. Grade 1 permeability can typically be obtained by using mixtures containing minimal amounts of ground granulated blast identify acronym furnace slag, ground granulated blast furnace slag (GGBFS) (35 percent) or fly ash (20 percent). Grade 2 permeability levels typically require slightly higher percentages of GGBFS or fly ash, or low amounts of silica fume (5-7 percent) or metakaolin (5-7 percent), with water cement ratios below 0.45 and moist curing for 7 days. To achieve grade 3 performance, the concrete will typically contain a higher percentage of silica fume (10 percent) or a combination of fly ash and silica fume with a w/cm ratio below 0.45 and moist curing for 7-10 days.

## 3.6. COMPRESSIVE STRENGTH

Compressive strength is perhaps the characteristic of high performance concrete most sought after by structural engineers. Higher strength concrete permits designers to create smaller member sizes or reduce the number of girders, beams, columns, or piers. This results in a reduction of the overall weight of the structure and more effectively uses prestressed concrete technology. As a result, there may be substantial reduction in the cost of the structure. States such as Virginia and Texas have reported cost savings in excess of 13 percent on highway bridges. The production of high-strength concrete cannot be accomplished by all contractors in Pennsylvania without an investment in additional quality control. While Pennsylvania has some excellent sources of aggregate and cementitious materials, this is not sufficient to produce concrete with a compressive strength in excess of 55 MPa (8 ksi). The production of durable high-strength concrete is not obtained by simply adding more cement. Producers must pay close attention to the selection of the cementitious materials and aggregates. All portland cements and fly ashes are not the same. The sequence in which the materials are charged into the mixer, the precise amount of water in the aggregates, and the water added to the mixture are also important. The aggregate size, gradation, and content must be optimized and producers must have the capability to store, discharge, and weigh multiple sources of cementitious materials. The initial concrete temperature must be controlled by cooling aggregates in the summer and heating them in the winter. Lastly, the producer must have a quality control and record keeping system that allows them to monitor the concrete strength batch by batch over time.

Concrete compressive strength is measured using AASHTO T22. This has to be modified for high-strength concrete. Neoprene capping is not typically used for concrete over 55 MPa (8 ksi) and these strengths require stiffer and higher capacity testing machines. Testing laboratories must use 70-durometer neoprene caps or sulfur caps for testing high-strength concrete. While some laboratories in Pennsylvania have this equipment, most field laboratories, concrete producers and local testing laboratories do not possess these machines. For HPC grade 1, the compressive strength is between 25 and 40 MPa. This covers most of the current PennDOT A, AA, and AAA concrete. Grade 1 can currently be produced by approved ready-mix concrete producers. Grade 2 is strength between 40 and 60 MPa. This level of performance is currently used for prestressed concrete in Pennsylvania. However, structural designers may find that this level of concrete strength may reduce the dead load of structures or reduce the amount of concrete in substructures. Grade 3 compressive strength is completely new to PennDOT projects. There has not been a project that has consistently used 70 to 80 MPa concrete. This grade of concrete is most likely to be used in the optimization of tall piers and prestressed concrete girders.

The production of high-strength concrete is accomplished through the careful selection of cementitious material combinations. The materials must hydrate in such a manner as to create a dense calcium silicate hydrate gel through the hydration of portland cement and pozzolans. The cement must produce enough calcium hydroxide to react with the pozzolans without creating weak aggregate/paste interfacial zones. The concrete also cannot overheat or the cementitious reactions will self-desiccate or "burn out." Additions of both fly ash and silica fume are often used to control the rate of reaction. The aggregate choice and gradation are very important in high-strength concrete. Concrete producers will be using smaller maximum-size aggregate and more continuous gradations than typically used for conventional strength concrete. Most importantly, the concrete producers must maintain stricter quality control to make the fine adjustments that are required in developing and maintaining a high-strength concrete design.

## 3.7. ELASTICITY

The modulus of elasticity is a fundamental material property that relates directly to the stiffness of a structural element and structural system. In concrete, a higher value of the modulus of elasticity implies higher strength in both tension and compression. It also implies higher flexural rigidity (EI) from both the higher modulus and the higher load required to impose the cracked moment of inertia. The increased flexural stiffness results in lower deflection values and therefore the possibility of longer spans. The primary use for higher modulus concrete is in prestressed concrete beams. The higher modulus can decrease elastic losses and optimize the prestressing steel. This optimization reduces the congestion within prestressed concrete, improving the probability of higher quality and uniform production. Higher modulus is not always beneficial. Higher modulus concrete will accumulate more stress from environmental strains, such as thermal gradients and shrinkage. In these conditions, the higher modulus of elasticity is detrimental to structural performance, as these strains may prematurely crack the section and subsequently reduce the flexural rigidity.

The modulus of elasticity is primarily a function of the compressive strength of the concrete, the modulus of elasticity of the coarse aggregate, and the quantity of the coarse aggregate. To moderately raise the modulus of a concrete mixture, concrete manufacturers can reduce the w/cm ratio and increase the compressive strength. However, to substantially increase the modulus of elasticity, a higher modulus aggregate must be used. The change in aggregate will also affect the thermal coefficient of expansion, creep, and shrinkage properties.

The modulus of elasticity, "E", is measured using ASTM C469. For HPC grade 1 the modulus of elasticity is between 20 to 30 GPa. This is the modulus that is currently delivered to PennDOT using the specifications for Type A, AA, and AAA concrete. While it is not regularly measured in highway projects, it has been measured in the LTPP program and in trial mixture designs. Grade 2 has a modulus of elasticity between 30 to 45 GPa. This is the modulus that is currently being delivered in prestressed concrete beams. The highest level of modulus, grade 3, is greater than 45 GPa. This level of modulus could be used to optimize long-span prestressed concrete girders and tall slender piers, by reducing prestress losses and increasing stiffness.

The production of concrete with a higher modulus in the interest and within the capabilities of prestressed concrete manufacturers in Pennsylvania. The high level of control that these manufacturers provide enables them to produce trial mixture designs and concrete beams that meet higher-than-normal elastic moduli. The cost of the higher modulus is a function of the cost of the transportation of aggregates.

## 3.8. SHRINKAGE RESISTANCE

Shrinkage can be divided into several causes and mechanisms, e.g., plastic shrinkage, autogenous shrinkage, drying shrinkage, and carbonation shrinkage. Plastic shrinkage cracks occur in the first 8 hours after placement and is an avoidable source of deterioration in concrete construction. Plastic shrinkage cracking is caused by the rate of evaporation exceeding the rate of bleed water production. The mitigation of plastic shrinkage cracking is obtained by limiting the evaporation rate or maintaining a moist curing environment. Autogenous shrinkage is the shrinkage caused by the hydration reactions of portland cement. This type of shrinkage induces tensile stresses that contribute to later cracking from drying shrinkage or applied stresses. Autogenous shrinkage can be mitigated by reducing the rapid rate of hydration at early ages. Drying shrinkage is the most widely sited cause of cracking in concrete flatwork. The loss of capillary water within the cementitious gel that binds together the aggregate in concrete causes a volumetric change. This volumetric change leads to tensile strains and eventual cracking of the concrete surface. The process of mitigating this type of shrinkage is highly dependent on the coarse aggregate content and type, total water content, extended curing times, proper joint placement, and selection and quantity of portland cement. Carlson (1938) showed the effect of aggregate type on shrinkage by using constant cement content, aggregate content, and water content. This is shown in Table 4. The effect of water content on drying shrinkage is shown in Figure 3. For a fixed w/c ratio, the water content increases, and the drying shrinkage increases with an increase in cementitious content.

Carbonation shrinkage is caused by the self-desiccation involved in the formation of calcium carbonate in hardened concrete. Elements exposed to moist conditions in highway applications have reduced carbonation shrinkage because of the solubility of calcium carbonate. Relatively dry (RH ~ 65%) closed areas with concentrations of carbon dioxide are the most susceptible to the formation of calcium carbonate. This may affect tunnels, underpasses in Pennsylvania. Mitigating carbonation shrinkage can be accomplished by maintaining low permeability and low water content and avoiding finely ground cementitious materials.

The result of restrained shrinkage is cracking in concrete. Cracks allow the penetration of salts and water that results in steel corrosion and, in the long term, structural failure. To minimize cracks, crack width or joint openings, the coarse aggregate content should be optimized, the water content must be controlled, and the concrete must be cured properly. Restricting crack width/ joint openings allows aggregates to transfer loads across the opening and allows crack sealant to function as designed.

Drying shrinkage (and autogenous shrinkage) is measured using ASTM C157. Concrete with grade 1 shrinkage will have shrinkage between 500 to 800 microstrains. (This is typical of concrete measured for AA concrete measured by the PennDOT Materials and Test Division and measured on the trial mixtures at Penn State). Shrinkage values for HPC grade 2 are between 200 and 500 microstrains. This type of concrete would substantially reduce drying shrinkage cracking in bridge decks and the prestressing losses in girders. Shrinkage values for HPC Grade 3 are less than 200 microstrain. This would be similar to Type K cement or concrete containing shrinkage compensating admixtures. Grade 3 would eliminate cracking in concrete flatwork from shrinkage or minimize prestress losses related to shrinkage.

Concrete producers in Pennsylvania are capable of minimizing shrinkage by optimizing mixture designs. The shrinkage can be reduced by choosing lower heat cements, reducing the water content, reducing the cementitious content, and increasing the coarse aggregate content. This is difficult with the current concrete specifications, which specify the minimum cementitious content, and do not allow equal mass replacement of all supplementary cementitious materials.

**Table 4. Effect of type of aggregate on shrinkage of concrete (Carlson 1938).**

| Aggregate Type | 1-year shrinkage, percent |
|---|---|
| Sandstone | 0.116 |
| Slate | 0.068 |
| Granite | 0.047 |
| Limestone | 0.041 |
| Quartz | 0.032 |

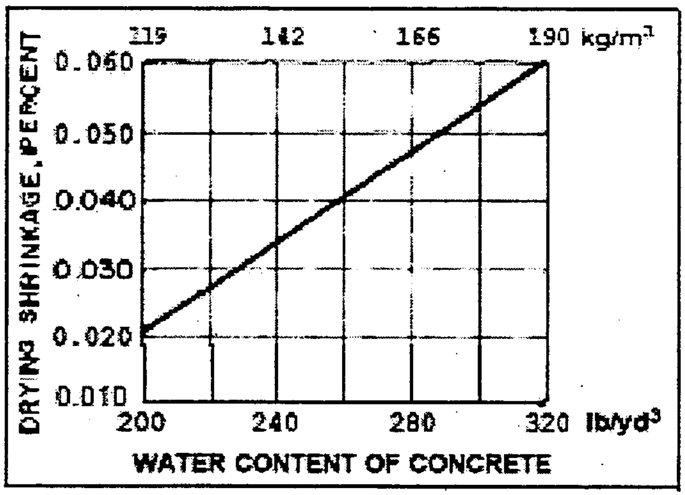


**Figure 3. Effect of water content on the drying shrinkage of concrete.**

## 3.9. SULFATE RESISTANCE

The sulfate resistance of concrete is largely dependent on the aluminates in the portland cement, particularly the tricalcium aluminate content ($C_3A$) of the portland cement. In a sulfate-rich environment, the monosulfoaluminate phases of the hydrated paste transform into ettringite, a calcium trisulfate aluminate hydrate phase. This transformation is highly expansive and induces internal tensile strains that are much larger than the tensile strain capacity of concrete. If only portland cement is used as the cementitious content, it is advisable to restrict the cement to Type II cement in sulfate environments. For most sulfate environments, a Type II cement should be blended with a 20-25 percent Class F fly ash, 35-50 percent ground granulated blast furnace slag, or 7 percent silica fume. These cementitious combinations resist the formation of ettringite in sulfate environments by stabilizing the monosulfate phases, reducing the permeability of concrete, and consuming a portion of the available calcium hydroxide in the cementitious paste. In most cases, fly ash and ground blast furnace slag will reduce the cost of the concrete mixture while provide high sulfate resistance.

Sulfate resistance is measured using ASTM C1012 method. Grade 1 corresponds to mild sulfate resistance and is applicable when measurable amounts (0.0 to 0.1 percent in soils or 0 to 150 ppm in water) of sulfates are detected. Grade 2 should be specified in moderate environments where the sulfate concentration in the soil is 0.1 to 0.2 percent or 150 to 1500 ppm in water. Grade 3 is reserved for severe sulfate exposure environments, greater than 0.2 percent in soil or 1500 ppm in water. Structures most susceptible to sulfate attack in the highway infrastructure are foundations, substructures, box culverts, and bridge piers. Structural elements in the splash zones in sulfate-rich water, seawater, or above the freeze line in sulfate-rich soils should be considered at least Grade 2.

Concrete producers in Pennsylvania already produce sulfate-resistant concrete, where specified. Type II cement, fly ash, ground granulated blast furnace slag, and silica fume are available throughout the Commonwealth.

## 3.10. TENSILE STRENGTH

The tensile strength of concrete is typically between 7 and 11 percent of the compressive strength of the concrete. While tensile strength is not used in reinforced concrete calculations, it contributes to the design of prestress concrete girders and contributes to the cracking resistance of concrete. The tensile strength increases moderately with increases in compressive strength. Steel fibers can be used in cases where large increases in tensile strength are desired.

Concrete tensile strength is measured using flexural strength (ASTM C78 or C293) or splitting tensile strength (ASTM C496). All of these tests slightly overestimate the actual tensile strength. In addition, tensile failures are brittle and often progressive. This is the reason that designers of prestressed concrete typically use $3\sqrt{f_c'}$ as a conservative estimate of tensile strength, where the accepted tensile strength of concrete is estimated between $6\sqrt{f_c'}$ and $7.5\sqrt{f_c'}$.

For HPC Grade 1, the tensile strength is adequate for most unreinforced pavements. Grade 2 should be specified in heavy-duty pavements and prestressed concrete beams. Grade 3 tensile strength is appropriate for high-strength prestressed concrete girders or specialized pavements for overweight trucks in acceleration or deceleration zones.

## 3.11. WORKABILITY

The workability of concrete is largely dependent on the water content of a given concrete mixture; however, chemical admixtures play an important role in plasticizing concrete mixtures for special conditions. The typical pavement mixture has a slump of 50 to 75 mm (2 to 3 inches) and the slump of other structural concrete ranges from 50 to 250 mm (2 to 5 inches). This level of workability provides paving machines, pumps, and other standard concrete placement equipment with the ability to handle concrete in a reasonable manner. To obtain these levels of fluidity, the concrete producer limits the water content of the mixture to the specified water-to-cementitious materials ratio and adds small amounts of water-reducing admixtures, as needed.) The purpose of specifying slump, or workability, is to ensure that the concrete materials do not segregate and the contractors can place the concrete with their equipment and skilled work force. Once the concrete is in place, a reasonably fluid mixture can be compacted and finished without honeycombing or entrapping large voids.

There are occasions where the concrete forms become congested with reinforcing steel, prestressing tendons, inserts, and hardware that require more fluid concrete. The design and control of such concrete requires high-range water reducing admixtures and adjustments to the concrete mixture proportions. The contractor must take precautions to avoid segregation and leakage from the forms, prevent blow-through in pumping the concrete, as well as size aggregate to flow through the congested areas. This type of concrete requires a higher level of expertise and quality control to ensure its proper placement. An even higher level of expertise and quality control is required to produce flowing/self-compacting concrete for structural elements that are not readily accessible by vibrators, chutes, pump hoses, or tremies. These hidden areas of box sections, columns, repair details, and other complicated formwork present unique circumstances in concrete construction. This type of concrete requires contractors to develop special mixtures and quality control procedures to assure the workmanship during placing. While this type of self-compacting concrete requires extra effort to produce, it provides durable and reliable concrete structures, and may substantially reduce labor, eliminate vibration noise, and encourage innovation of the construction systems in concrete construction.

Flow or workability is typically measured in terms of slump using the ASTM C143 method. While this is not the only means by which to measure the flow of concrete, it is a method that contractors are familiar with and it is suitable for defining various grades of concrete flow. Workability grades can be classified as the following:

- Grade 1 for slump values between 50 -125 mm (2 - 5 in),
- Grade 2 for slump values between 125 - 200 mm (5 - 8 in.), and
- Grade 3 for slump values greater than 200 mm (8 in).

Grade 1 is the slump for most standard construction. This includes most levels of workability currently specified by the Department of Transportation, including pavements, bridge decks, substructures and foundations. Grade 2 workability would be specified for conditions under which the concrete forms are severely congested, less than 37 mm (1.5 in.) between reinforcing or form surfaces, and in places of limited access. This is typically dependent on the contractor's forming method and the means of concrete delivery. Grade 3 workability should be specified only in areas where access is extremely limited and where there are blind areas that cannot be vibrated or placed directly. This type of concrete must flow around obstacles without segregation and must be self-compacting. The contractor and concrete producer must work together to develop and place these specialized mixtures. Slump tests are not appropriate as a quality control tool for these types of mixtures. The fluidity of self-compacting concrete must be measured and documented by a V-Funnel flow test or L-Box tests. The V-Funnel test measures the flow of concrete through a hydraulic shape to calculate its viscosity. The L-Box test measures the flow of concrete through a congested construction assembly. Both have been used in projects and research related to self-compacting concrete, but neither is standardized and ASTM or AASHTO standard.

The benefits of Grade 2 and 3 workability mixtures are that they 1) eliminate the restriction on the height of a lift, 2) eliminate the necessity for construction scaffolding for vibration, and 3) allow placement of the bottom and walls of box section members at the same time

While all concrete producers in Pennsylvania will not immediately be capable of delivering Grade 2 or 3 workability concrete, many areas of the Commonwealth and contractors can benefit from the advantages. Grade 2 workability is typically obtained by using high-range water reducers from PennDOT-approved suppliers. These supplies and products already exist and can be used in nearly every district of the Commonwealth. Grade 3 workability, self-compacting, flowing concrete requires changes in the basic nature of the mixture designs. Typically, this type of concrete requires very fine aggregates, high-range water reducers, and chemical adhesive admixtures that prevent the cementitious paste from being washed away. This type of product has not been approved for use in PennDOT mixtures and will require approval, as well as the development of a quality-control protocol for field applications.

## 3.12. CREEP

Creep of concrete is a time-dependent deformation that occurs under sustained load. Creep is considered in design of bridge structures in connection with deflections and prestress losses. Parameters affecting creep are discussed in standard references such as *ACI Committee 209 Report* (ACI 1999). ACI 209 refers to ASTM C 512 for determination of creep properties based on laboratory tests for specific concrete mixtures. The equation for creep given in AASHTO Load and Resistance Factor Design (LRFD) Specifications is based on a modified form of the ACI 209 equations and includes specified 28-day structural compressive strength, $f_c'$, as a parameter. A limited amount of work has been done on creep of high-strength concrete, as reported by ACI Committee 363 (1997). Their report indicates that creep of high-strength concrete is reduced significantly relative to normal-strength concrete, but that because of higher sustained stress levels, the total creep of different strength concretes will be about the same. Zia et al. (1993) reported that observed creep strains for high-strength concrete, minimum compressive strength = 69 MPa (10,000 psi), ranged from 20 to 50 percent of that of conventional concrete, minimum compressive strength of 28 MPa (4,000 psi).

Creep properties may be specified in two ways. The creep coefficient, $\upsilon_t$, is defined as the ratio of creep strain to initial strain under a constant stress state. The creep strain is the time-dependent strain given by the difference between total strain and initial strain. ACI 209 indicates that the normal range of the ultimate creep coefficient, $\upsilon_u$, is between 1.30 and 4.15 with an average value of 2.35 for standard conditions. Correction factors are provided for other than standard conditions. Creep can also be specified in terms of specific creep, $\delta_t$, defined as creep strain per unit stress. The creep coefficient and specific creep are related through the modulus of elasticity as follows:

$$v_t = \delta_t E_{ci}$$

It is recognized that concrete creep is a highly variable property and that many of the correction factors proposed involve parameters that are themselves highly variable and often not known at the design stage. For these reasons, design equations for deflection and prestress loss are normally based on empirical expression involving, in most cases, only the specified compression stress. In spite of the simplifications inherent in this approach, these empirical expressions appear to provide satisfactory results.

HPC grade 1 has a creep coefficient value between 2.0 and 3.0, typical of normal-strength concrete at sustained loadings less than 50 percent of the compressive strength. Grade 2 has values between 2.0 and 1.4. These values are primarily attained through the decrease of w/cm ratio and the increase in compressive strength. Grade 3 is very low creep concrete with creep coefficients less than 1.4. This is attained in high- strength concrete for special load applications.

# 4. EVALUATION OF CURRENT PENNDOT SPECIFICATIONS AND CONCRETE PERFORMANCE

The Pennsylvania Department of Transportation has more than 95 years of experience constructing roads and bridges. Through this time, the standards for highway and bridge design and construction have evolved into the current department guidelines and specifications. It is valuable to understand the current level of performance delivered by the PennDOT specifications for concrete. The current level of performance can be extracted from reviewing the Commonwealth's Department of Transportation Specification, Publication 408, as well as several recent PennDOT research and investigation contracts and comparing them to state-of-the-art practices in concrete construction.

The Pennsylvania Department of Transportation has used several forms of high- performance concrete in recent years. Bridge decks with Type K cement and corrosion-inhibiting admixtures (Ferrogard and DCI) have been cast to reduce the potential for deck cracking and subsequent corrosion of the reinforcing steel. In addition, PennDOT has several items in the specifications that have the intended and unintended effects of improving the performance of concrete.

Each of the performance characteristics outlined in this report is implicitly or explicitly addressed in the current specification. The means by which the current specification addresses some the characteristics is not always apparent, but it should be clearly recognized that PennDOT's specifications for concrete have served the Commonwealth for many years. Many of the provisions are well researched and represent high quality practices in the specification of concrete for highways and bridges.

Table 5 is a copy of Table 3 from this report. Table 6 provides a grade for the average concrete produced for PennDOT using the current practices. The practices and quality of construction vary throughout the Commonwealth by district and by particular job; Table 6 provides a general grade based on quality control tests, anecdotal information from districts, and past performance studies. This average grade focuses on generally accepted practices and does not reflect problems that have arisen from construction mistakes, omissions, or errors. Using Tables 5 and 6, this chapter provides a summary of the current state of practice in Pennsylvania for each of the 14 performance measures and a discussion of current concrete specifications.

**Table 5. Grades of performance for high-performance concrete.**

| Performance Characteristics | Standard Test Method | Proposed HPC performance grade | | |
|---|---|---|---|---|
| | | 1 | 2 | 3 |
| **FT**<br>Freeze-thaw durability (relative modulus, 300 cycles) | AASHTO T161 Proc. A | 60%≤X≤80% | 80%≤ X≤90% | 90%≤ X |
| **SR**<br>Scaling resistance (visual rating of surface 50 cycles) | ASTM C 672 | X=2,3 | X=1 | X=0 |
| **AB**<br>Abrasion resistance (wear depth, mm) | ASTM C994 | 2.0>X≥1.0 | 1.0>X≥0.5 | 0.5>X |
| **AS**<br>Alkali-silica reaction | ASTM C1260 | X<0.10%<br>At 14 Days | | |
| | ASTM C441 | | X<0.10%<br>At 56 Days | X<0.05%<br>At 56 days |
| **CP**<br>Chloride penetration, Coulombs | AASHTO T 277 | 4000≥X>2500 | 2500≥X>1500 | 1500≥X |
| **CS**<br>Compressive Strength, MPa (ksi) | AASHTO T 22 | 24≤X<32<br>(3.5≤X<4.6) | 32≤X<55<br>(4.6≤X<8.0) | 55≤X<82<br>(8.0≤X<12.0) |
| **SD**<br>Strength ratio<br>$\frac{28\text{ day } f_c}{7\text{ day } f_c}$ | AASHTO T 22 | 1.15 | 1.25 | 1.40 |
| **ME**<br>Elasticity, GPa (Msi) | ASTM C 469 | 20≤X<30<br>(2.9≤X<4.3 ) | 30≤X<45<br>(4.3≤X<6.5 ) | 45≤X<br>(6.5≤X) |
| **SH**<br>Shrinkage (microstrain) | ASTM C 157 | 800>X≥500 | 500≥X200 | 200≥X |
| **SU**<br>Sulfate resistance (expansion) | ASTM C1012 | X<0.10%<br>At 6 months | X<0.10%<br>At 12 months | X<0.10%<br>At 18 months |
| **TS**<br>Tensile strength MPa (psi) | ASTM C78 | 4 >X≥5<br>(580≤X<720) | 5 >X≥6<br>(720≤X<870) | X > 6<br>(870≤X ) |
| **WK**<br>Workability mm (in.) | ASTM C143 | 50>X≥125<br>(2>X>5) | 125>X≥200<br>(5>X>8) | 200<X<br>(8<X) |
| **CC**<br>Creep coefficient<br>ε/ε | ASTM C 512 | 3.0≥V>2.0 | 2.0≥V>1.4 | 1.4≥V |

**Table 6. Summary of performance grades for PADOT 2000 specification.** (See Table 5 for definition of performance grades.)

| **Performance Characteristics** | **Estimated Current Performance Grade** (*lowest* **0-1-2-3** *highest*)‡ | | | |
|---|---|---|---|---|
| | **AAA**<br>Deck slabs | **AA**<br>Parapets, diaphragms, shear blocks, abutment backwalls, u-wings above bridge seat, and cheekwalls, pavements | **A**<br>Precast Box Culverts, piers, abutments below bridge seat, pedestals, wingwalls, retaining walls and footings | **Prestressed**<br>Bridge girders |
| **FT**<br>Freeze-thaw durability | 2 | 2 | 2 | 2 |
| **SR**<br>Scaling resistance | 1 | 1 | 1 | NA |
| **AB**<br>Abrasion resistance | 1 | 1 | 1 | NA |
| **AS**<br>Alkali-silica Reaction | 1 | 1 | 1 | 1 |
| **CP**<br>Chloride penetration | 0 | 0 | 0 | 0 |
| **CS**<br>Compressive strength | 1 | 1 | 1 | 2 |
| **SD**<br>Strength ratio | 1 | 1 | 1 | 0 |
| **ME**<br>Elasticity | 1 | 1 | 1 | 1 |
| **SH**<br>Shrinkage | 1 | 1 | 1 | 2 |
| **SU**<br>Sulfate resistance | 1 | 1 | 1 | 1 |
| **TS**<br>Tensile strength | 1 | 1 | 1 | 2 |
| **WK**<br>Workability | 1 | 1 | 1 | 1 |
| **CC**<br>Creep c++++oefficient | 1 | 1 | 1 | 2 |

NA – not applicable
‡ Grades are estimated from limited data available from QC/QA records and past PADOT research.

## 4.1. FREEZE-THAW RESISTANCE

Pennsylvania's climate provides for repeated freezing and thawing cycles for most of the highway infrastructure. Therefore, the current specification requires all exposed concrete to have entrained air. The specification requires 6.0 percent $\pm$ 1.5 percent total air content in the plastic state or 3.5 to 8.0 percent entrained air in the hardened concrete. The specified level of air is the industry standard and consistent with ACI recommendations to resist freezing and thawing in Pennsylvania. Under severe conditions of deicing salts and moist exposures (bridge decks, pavements), the American Concrete Institute's recommendations (ACI 201.2R) are for 5.5 to 6 percent air content.

### Evaluation

The requirement for hardened concrete to contain between 3.5 to 8.0 percent entrained air is very broad by technical standards. The presence of either 3.5 or 8 percent air content in hardened concrete shows an extreme problem in quality control. In addition, 3.5 percent air content is not sufficient to provide freeze-thaw durability in pavements or bridge decks and is detrimental to the scaling resistance to concrete. The technical standard for hardened concrete should be 9 $\pm$ 1 percent of mortar fraction of the concrete. This relates to a minimum of 4 percent entrained air in hardened concrete for AA and AAA concrete mixtures in Pennsylvania. In addition, the entrained air must have a spacing factor equal to or less than 0.20 mm (0.008 in). If the concrete exhibits deficiencies related to air content, or is suspected by the Engineer to have deficiencies, the hardened concrete should be tested in accordance with PTM No. 623. Voids less than 1 mm in diameter should complies more than 4 percent of the concrete volume and they should have a spacing factor equal to or less than 0.20 mm (0.008 inches).

## Summary

- **Current**: Require 6 percent air and durability factor as low as 60 percent after 300 cycles. Allow 3.5 to 8 percent in hardened concrete.
- **Proposed**: Require durability for 80 percent for bridge decks, pavements and other structures subjected to saturated freezing and deicing salts. Provide at least 4 percent entrained air content in hardened concrete and a spacing factor less than 0.20 mm (0.008 inches)
- **Impact**: no cost impact; most air-entraining admixtures already meet this requirement, ASTM C260.

## 4.2. SCALING RESISTANCE

PennDOT does not have a standard of performance or specification requirements regarding scaling resistance. The department indirectly addresses the scaling resistance of concrete by specifying air-entrained concrete. While this is only one aspect of providing scaling resistance, it is the most important one.

## Evaluation

The current level of resistance to scaling can only be estimated from anecdotal evidence. Considering the level of chloride intrusion is known to be relatively high in Pennsylvania bridges and pavements and the incidence of reported scaling ranges from moderate to low, the overall performance of concrete in Pennsylvania's highways is good. There are three measures associated with scaling resistance:

(a.) proper air entrained void system,

(b.) adequate moist curing conditions, and

(c.) a period of drying before the first application of deicing salts (6-9 months).

The scaling resistance of concrete could be increased using extended moist curing techniques and by avoiding the use of deicing salts in the first year. In light of the low to moderate evidence of scaling, the current measures seem adequate. Any system-wide change in specification would have a negative financial effect for low to moderate return, since scaling is rarely the sole cause of concrete failure. However, in urban areas and on bridge decks where scaling is a known problem and salt application rates are high, scaling resistance could be improved by extending curing times and avoiding salt applications in the first year. While extended curing may add $2.69/m$^2$ ($0.25/ft$^2$) to the cost of construction, it may avoid the cost of resurfacing or diamond grinding bridge decks in the future.

## Summary

- **Current**: No current standard or specification.
- **Proposed**: No change in most pavement applications. Extended moist curing and reduction of salt exposure in the first year would be beneficial.
- **Impact**: Extended curing would have a $2.69/m$^2$ ($0.25/ft$^2$) impact on construction costs for the bridge deck surface, but would increase the life of the bridge deck.

## 4.3. ABRASION RESISTANCE

The department does not have a standard of performance for concrete abrasion resistance. As an indirect measure, the department requires the abrasion loss of Type A coarse aggregate be less than 40 percent in an LA abrasion test, PTM No. 622. PennDOT addresses the abrasion of acceleration and deceleration zones and in hydraulic applications on a case by case basis, usually after damage has occurred.

## Evaluation

There is a need to specify abrasion resistant concrete in acceleration and deceleration zones to avoid costly repairs and resurfacing. This is particularly important in areas that have a grade changes associated with the intersection. This type of remedial work affects the operation of the busiest intersections and is evident throughout the state. Specifications should require 28 MPa (4000 psi) compressive strength and use of siliceous sand in these applications is warranted and additional curing measures should be taken.

## Summary

- **Current**: No current standard or specification
- **Proposed**: No change for most concrete. Use 28 MPa (4000 psi) compressive strength mixtures in acceleration and deceleration zone that are on steep grades or on interstate highways. Extend moist curing for 7 days in these zones.
- **Impact**: No immediate cost impact for most concrete. Extended curing would have a $2.69/m$^2$ ($0.25/ft$^2$) impact on the abrasion resistant zones of pavement or bridge decks, but increase the life of these elements.

# 4.4. ALKALI -SILICA REACTION

The department has been using pozzolans to mitigate alkali silica reaction for several years and implemented steps in the construction specifications, Publication 408, in July 1999 by adding subsection 704.1 (h) *Mix Designs Using Potentially Reactive Aggregate*. This specification change provided a means of evaluating potentially reactive aggregates and provides guidelines for the design of concrete mixtures. The mitigation techniques specified in the specification are

(a) use low alkali Portland cement (less than 0.6%),
(b) 50 percent reduction in expansion from the control using 15 to 25% fly ash, or
(c) 50 percent reduction in expansion from the control using 25 to 50% ground granulated blast furnace slag.

## Evaluation

The revised specifications are equivalent to the proposed grade 1 ASR performance. The potential to improve the specification even further exists with little or positive economic impact on the department. The reduction of only 15 percent of the total portland cement content, when using fly ash, maintains the alkalis in the concrete. An equal mass replacement of 20 to 25 percent of the portland cement has the potential to reduce the price of concrete by $2.87/m$^3$ ($2.16/yd$^3$) (see Appendix B). The current specification permits portland cement used with pozzolans to have an alkali content of 1.4 percent. This alkali content is very high, and it should be decreased to 1.0 percent to meet higher grades of resistance. Since very few portland cements have equivalent alkali contents in excess of 1.0 percent, this has no financial impact on the department's concrete. The use of silica fume is permitted under special conditions and metakaolin is not currently permitted to mitigate alkali silica reaction. Both pozzolans (silica fume and metakaolin) should be permitted.

### Summary

- **Current**: Use fly ash or ggbfs to mitigate ASR; however, only reduce the cement content by 15 percent when using fly ash.
- **Proposed**: Allow an equal mass replacement of fly ash to mitigate ASR.
- **Impact**: An equal mass replacement of 20 to 25 percent of the portland cement has the potential to reduce the price of concrete by $2.87/m$^3$ ($2.16/yd$^3$)

## 4.5. CHLORIDE PENETRATION

The department has a testing standard for the potential for chloride permeability, but does not have a specified or acceptance standard of performance or maximum chloride penetration limit within the scope of construction specifications or in the approval of concrete mixtures. The department samples concrete for AASHTO T277 testing, Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration, in selected circumstances for informational purposes. In a recent survey of 40 Pennsylvania bridges (with an average age of 10 years), 120 samples were measured for chloride ion permeability. The level of chloride permeability was an average of 4530 coulombs. In addition, 15 percent of reinforcing steel was already exposed to chloride levels above the 0.73 $kg/m^3$ (1.2 $lbs/yd^3$) corrosion threshold.

## Evaluation

The diffusion of chlorides and the increased potential for scaling and corrosion that accompany the higher chloride concentration is a major problem with the Pennsylvania infrastructure. The measure of chloride permeability in Pennsylvania is extremely high for AAA concrete, the Commonwealth's highest grade. The value of 4530 coulombs at 10 years is more than double the maximum specified levels at 28 days in Virginia, New York, and Texas, the states that use the AASHTO T277 test in mixture approval. It is reasonable to expect the permeability of concrete to be below 4000 for A and AA concrete and below 1500 coulombs for bridge decks and other reinforced structures exposed to deicing salts. The cost of the current level of permeability is very high. It is the single most important item decreasing the life of bridge decks and structures exposed to water or deicing salts. With approximately 25,000 bridges in Pennsylvania, the state, counties, and townships will replace between 800 to 1000 bridge decks a year for the next 25 years.

## Summary

- **Current**: No standard or specification
- **Proposed**: Use AASHTO T277 in the mixture design approval process without potentially inhibiting admixtures, as they are not typically drastically changing permeability.
- **Impact**: This is the single most important change that will improve the long-term life of the Pennsylvania infrastructure. There is no immediate cost or reduction in cost, but long-term life will be improved, averting future construction and reconstruction.

## 4.6. COMPRESSIVE STRENGTH

The PennDOT specifications for 28-day structural design compression strength are appropriate for the stated applications. The structural design compressive strengths are between 20 and 28 MPa (3000 to 4000 psi) for A, AA, and AAA concrete. The 28-day minimum mix design compressive strength for A, AA, and AAA is 1.7 to 3.4 MPa (250 to 500 psi) greater than the 28-day structural design compressive strength. The structural design compressive strength of prestressed concrete is between 28 and 55 MPa (4000 to 8000 psi). The mixture design approval process requires that the concrete average 6.9 MPa (1000 psi) over the minimum mix design compressive strength. This is 10.3 MPa (1500 psi) over the structural design compressive strength.

### Evaluation

The compressive strength of concrete is substantially over designed in all classes of concrete. This consistently leads to high cement contents and the associated shrinkage cracking and heat of hydration-related strain. The minimum mixture design compressive strength should be equal to the structural design compressive strength, $f_c$'. The average compressive strength of a mixture, $F_{cr}$', should be a function of the required minimum compressive strength and the standard deviation of the mixture, based on statistical records. The value of Fcr' should be the greater of the following two equations:

(1.) $F_{cr}=f_c'+1.33\sigma$

(2.) $F_{cr}=f_c'+2.33\sigma-3.4$ MPa

For a concrete producer with good quality control, this would lead to an average compressive strength of 32.5 MPa (4700 psi) for AAA concrete, $f_c' = 28$ MPa (4000 psi), instead of the current 37.9 MPa (5500 psi). The implications of such a change in the specification would be the following:

- A reduction in cracking in all decks and pavements.

- A reduction in the cost of concrete for all A, AA and AAA by approximately $\$2.14/m^3$ ($\$1.60/yd^3$).

This includes only the cost of cementitious materials, not the associated savings from the reduction in admixtures. (see Appendix B).

However, the minimum mix design compressive strength is not consistent with the 28-day design compressive strength. The difference between the design structural strength and the mix design strength in the specification is not technically or statistically correct. There is anecdotal information that the difference is due to the inability of all concrete producers to deliver compressive strengths on a regular basis. This should be considered a separate problem and addressed in payment reductions. The current specification should require the same minimum mix design compressive strength and structural design compressive strength, $f_c'$. This still requires concrete producers to provide an average compressive strength equal to $F_{cr}$ in the above equations. These are the accepted limits from the ACI and AASHTO and are the equation by which all structural reliability is based in the United States, for both buildings and bridges.

## Summary

- **Current**: Concrete compressive strengths are typically specified 3.4 MPa (500 psi) higher than the required minimum structural strength. There is no statistical deviation or monitoring. Minimum cement contents are specified.
- **Proposed**: Minimum compressive strength should be equal to the minimum structural compressive strength. This is true is nearly every other state. Variations should be determined according the statistical methods described in ACI 318 or AASHTO guidelines. The minimum cement contents should be lowered in line with industry standards and the supplementary cementitious materials (fly ash, ggbfs, silica fume, etc.) should be included in the minimum amount of cementitious materials.
- **Impact**: This change will improve the overall quality of concrete durability for AAA and AA concrete and will decrease the cost of concrete to the department by approximately $\$2.14/m^3$ ($\$1.60/yd^3$).

## 4.7. STRENGTH RATIO

The department has no guidelines on the development of compressive strength. The Commonwealth's specifications only require producers to report the 7-day and 28-day compressive strength of the trial mixtures. The department does not limit the heat of hydration of portland cement or combinations of cementitious materials. Data collected from PennDOT's approved mixture designs shows that the average AA or AAA concrete has a 7-day compressive strength that is 78 percent of the 28-day compressive strength. This same data also shows that the average compressive strength of AA concrete at 7 days is more than 3.4 MPa (500 psi) over the required 28-day structural design compressive strength. Likewise, the average 7-day compressive strength of AAA concrete is more than 5.5 MPa (800 psi) greater than the required 28- day structural design compressive strength.

### Evaluation

Throughout the Commonwealth, concrete compressive strengths specified for 28 days are being met at 7 days or less. This is a result of specification language that accepts concrete early if the strength is met early. This practice is a major factor contributing to the cracking of concrete from thermal strains developed within concrete structures. The rapid hydration of cementitious systems increase the rate of the hydration of portland cement, thereby creating less durable paste structures and microcracking from autogenous shrinkage and thermal gradients. The current specification encourages this strain accumulation and associated cracking by allowing contractors to accelerate the hydration and strength development of concrete. The specification permits the contractor to meet the 28-day compressive strength in 7 days. This practice has the benefit of allowing early opening of bridges and pavements, but it reduces the long-term strength development and durability of concrete. Concrete hydrating under normal conditions will have between 60 and 70 percent of the 28-day compressive strength after 7 days. Concrete hydrating under conditions that are more rapid generates autogenous shrinkage strains, thermal gradients, and coarse cementitious paste structures. These structures are permeable and often have substantially microcracking. Considering the department's desire to construct long-life structures, this issue becomes an important aspect of quality assurance for high performance concrete. The cost associated with this is related to the long term durability of bridge decks and pavements. This should be weighed against the need to open pavements and bridges within the first 7 days after construction. If the need to open the bridge or pavement early is greater than the need for its long-term durability, the current practice is warranted. If the need for long-term durability of a pavement or bridge deck is greater, the specifications should encourage a steady hydration of the cementitious system without high early age strength. The concrete should not be accepted with 7-days compressive strength data. The concrete should be required to have a 20 to 35 percent strength gain between 7 and 28 days. The specifications should encourage the long term strength gain and reduce the incentives for rapid early strength gain unless these is an imperative need for the opening of the pavement or structure in 7 days.

## Summary

- **Current**: No current standard or specification.
- **Proposed**: Encourage 20-35 percent strength increase between 7 and 28 days.
- **Impact**: This would slow down the hydration and stop much of the early age cracking. This has no direct economic impact.

## 4.8. MODULUS OF ELASTICITY

The department does not require a specific modulus of elasticity for any class or application of concrete. Designers use the ACI and AASHTO prediction equation without the corrections for aggregate type to estimate stiffness and deflection of structures.

## Evaluation

The current treatment of modulus is consistent with the practices in the industry. There is little effort in the highway industry to optimize structures using modulus of elasticity. Although this has been used for 10 years to optimize building column and beam deflection, the same techniques have not been applied to bridges. There is no immediate financial benefit to increasing or decreasing the modulus of elasticity. The primary application would be in reducing the deflections and losses in prestressed concrete. However, this is not currently the controlling factor in the design of prestressed concrete girders or other elements.

### Summary

- **Current**: No current standard or specification
- **Proposed**: No proposed changes.
- **Impact**: No impact.

## 4.9. SHRINKAGE

The department has no specification or guidelines for reducing shrinkage cracking. The specifications have several indirect measures that increase shrinkage in concrete to the detriment of long term durability of pavements, bridge decks, and other structures. These measures include the following:

- Coarse aggregate content for A and AA concrete is only required to be above 0.37 $m^3/m^3$ (9.93 $ft^3/yd^3$) and there is no coarse aggregate requirement for AAA concrete.
- The specification requires nominal curing procedures with respect to shrinkage reduction.
- The specification requires between 385 and 456 $kg/m^3$ (6.75 – 8.00 $sks/yd^3$) of portland cement for AAA concrete.
- The specifications increase the minimum cementitious content above 400 $kg/m^3$ when using more than 15 percent fly ash.

### Evaluation

The department is in need of major measures to reduce the shrinkage of concrete. Coarse aggregate content, volume of cementitious materials, heat of hydration, and curing of concrete are the most important variables in reducing drying and autogenous shrinkage in pavements and bridge decks. The low minimum coarse aggregate content required in A and AA concrete, 0.37 $m^3/m^3$ (9.93 $ft^3/yd^3$), and the lack of any limit for AAA concrete are major contributing factors to shrinkage cracking in Pennsylvania's highways. The average volume of coarse aggregate in the surveyed districts was 0.39 $m^3/m^3$ (10.56 $ft^3/yd^3$) for AA concrete and 0.38 $m^3/m^3$ (10.28 $ft^3/yd^3$) for AAA concrete. These correspond to Coarse Aggregate Factors (CAF) of approximately 0.63 and 0.61 for AA and AAA concrete, respectively. The recommended CAF for No. 57 coarse aggregate and approved fine aggregate sources would be greater than 0.69, which would correspond to 0.43 $m^3/m^3$ (11.5 $ft^3/yd^3$). Considering the Commonwealth has a wide variety of aggregate combinations, this ratio may drop as low as 0.41 and still produce high quality concrete mixtures. This should be the lower target for shrinkage resistance concrete. Since coarse aggregate is less expensive than fine aggregate in most areas of Pennsylvania, this change would decrease the shrinkage potential of concrete and decrease the cost of concrete by \$0.43/$m^3$ (\$0.32/$yd^3$) (see Appendix B).

The cementitious content of concrete should be reduced because the high volume of cement paste is a major cause of the shrinkage in concrete. The reduction in cementitious contents will reduce the cost for all A, AA and AAA concrete by approximately \$2.14/$m^3$ (\$1.60/$yd^3$).

The department allows membrane-forming curing compounds on nearly all structures and pavements. The application rates for these types of curing measures are not easily controlled or assured in the field, especially in moderate-to-high-evaporation environments. As such, they provide very little resistance to shrinkage cracking. When wet curing measures are used, they are only required for 96 hours in pavements and for 7 days or until the compressive strength is met for bridge decks and other structures. Wet curing should be used on all bridge decks for at least 7 days and membrane curing compounds should not be used on reinforced concrete elements. Pavements should be wet cured in ambient temperatures above 20°C (77°F) or in a relative humidity below 80 percent.

## Summary

- **Current**: No direct standard or specification. The specification stipulates a minimum CAF of 0.61 and minimum cement content for AAA concrete of 385 kg/m$^3$ (6.75 sacks/yd$^3$).
- **Proposed**: Increase the coarse aggregate content of the concrete mixtures and reduce the cement content requirements. Require moist curing during hot or dry weather.
- **Impact**: The impact of the changes would be a decrease in the cost of concrete by \$0.43/m$^3$ (\$0.32/yd$^3$) from the increase in coarse aggregate and a reduction in the cost of concrete by another \$2.14/m$^3$ (\$1.60/yd$^3$) for the reduction in cementitious content.

## 4.10. SULFATE RESISTANCE

PennDOT addresses the sulfate resistance of concrete in the classical method. The department limits the $C_3A$ content of portland cement or permits the use of blended cementitious materials to meet performance characteristics defined by ASTM C1012.

## Evaluation

The current specification provides a good means of ensuring the sulfate resistance of concrete exposed to sulfate rich soils and water. However, there are insufficient guidelines to assist engineers in assessing the need for specifying sulfate resistance concrete. There is the need to permit greater than 15 percent fly ash as a equal mass replacement for portland cement. This will provide higher sulfate resistance at less cost and without encouraging shrinkage cracking from excess paste. The impact of reducing the portland cement by 20 to 25 percent has the potential to reduce the price of concrete by \$2.87/m$^3$ (\$2.16/yd$^3$).

## Summary

- **Current**: Current specification is sufficient; however, increase cementitious contents with the use of fly ash to mitigate sulfate attack.
- **Proposed** : Allow greater amounts of fly ash for sulfate resistance without increasing the total cement content.
- **Impact**: Impact would decrease the cost of concrete by \$2.87/$m^3$ (\$2.16/$yd^3$) and provide higher sulfate resistance, lower shrinkage and lower permeability.

## 4.11. TENSILE STRENGTH

The department does not require a specific tensile strength for A, AA or AAA concrete. Previous versions of the construction specifications required flexural strength, indirect tension, tests to be used in quality control and quality assurance. Designers use the ACI and AASHTO prediction equations for the estimation of cracking and tensile strength in prestressed concrete. This limits the allowable tensile stress, ft, to $3\sqrt{f_c'}$.

## Evaluation

The current treatment of tensile stress is consistent with the practices in the industry. While it is widely recognized that tensile strength is an important parameter in the design of pavements, only the state of Texas uses flexural strength as a Quality Control/Quality Assurance (QC/QA) tool for pavement construction. The problems of curing and handling large flexural specimens led to the demise of the tests. There is little effort in the highway industry to optimize structures for tensile stresses. The prestressed concrete industry consistently looks at splitting tensile tests to determine the indirect tensile strength of concrete to optimize tendon layout, prestressing sequence and crack control. There is no immediate financial benefit to increasing or decreasing the tensile strength of concrete. The primary application would be in reducing the cracking in pavements and bridge decks. However, this is not currently a design consideration in the design of these elements.

## Summary

- **Current**: No current standard.
- **Proposed**: No change. Tensile stresses could be monitored for prestress, but no specification change is required.
- **Impact**: No impact. Monitoring tensile strength would improve prestress loss and deflection calculations.

## 4.12. WORKABILITY

The new workability specifications allow the contractor to select a slump to accommodate the labor and equipment on a particular project. The department changed its slump requirements in 1999 to allow the contractors to select any slump up to 125-mm (5 inches) when the mixture does not contain water-reducing admixtures. The maximum slump limit is increased to 165 mm (6.5 inches) and 200 mm (8 inches) respectively for mixtures containing water-reducing admixtures and superplasticizers. The contractor must specify the slump in the quality-control plan and must meet the slump within ±40 mm (1.5 inches).

## Evaluation

This is a very good step forward, but the limits should be more carefully considered. The use of 165 mm (6.5 inches) slump in mixtures containing water reducers is too high. This level of workability is obtained by so-called "mid-range water reducers, not by the most conventional water reducers. The specification should limit the slump for non-congested areas to a maximum of 125 mm (5 inches). The specifications permit the contractor to use superplasticizers, high-range-water-reducing admixtures, at their discretion for any particular application. The specification should be more discriminating by limiting this to congested areas and limited access areas within formwork.

## Summary

- **Current**: Contractor specifies slump within limits.
- **Proposed**: Maximum limit of 125mm (5 inches) should be specified unless there is proof of congestion. In congested areas, higher slumps should be permitted using superplasticizers.
- **Impact**: No cost impact; quality will be easier to monitor with clear limitations and expectations for the slump as a QC/QA test.

# 4.13. SPECIFIC CREEP

The department does not consider creep in its specifications, but refers to it in its design manual (DM-4). There is no specific requirement for creep in the manual and values chosen by engineers are not related to concrete mixture designs for specific elements. The creep in prestressed concrete beams induced from axial loadings are neglected and typically included in a lump sum loss prediction.

## Evaluation

Because of the high variability of creep, simple empirical rules are normally used in design to check long-time deflections and prestress losses. In most cases, these rules provide satisfactory performance. PennDOT recently replaced the Bureau of Public Roads equations for prestress loss with the more recently developed AASHTO rules. These requirements can be considered as being consistent with the current state-of-the-art for design. As higher strength concretes come into use, particularly for prestressed concrete, the simple empirical rules may not be appropriate. In these cases more refined calculations may be necessary to obtain realistic estimates of deflection and prestress losses. In such cases, the proposed performance classes for creep, which correspond approximately to the corresponding strength classes, may be used.

## Summary

- **Current**: Use AASHTO Design Guidelines
- **Proposed**: No design changes; however, allow empirical data for creep.
- **Impact**: No cost impact; empirical data will improve prestress loss predictions.

# 5. HIGH PERFORMANCE CONCRETE MIXTURES AND BENEFITS

## 5.1. EXTENDED LIFE CYCLE

Improved long-term performance is an important benefit of HPC. The PennDOT considers the initial cost the major factor in selecting bridge types and details. A closer look at durability and the life cycle cost implications should be taken in the future as an important factor in making such choices. HPC structures with longer design lives have the potential to be designed and constructed for a first cost near that of conventional concrete structures. Consideration of life-cycle costs should favor the choice of HPC.

The key to high performance and long-term durability lies in the microstructure of the cementitious material. A superior product can be engineered by increasing the materials packing efficiency. A high-packing density with a clear understanding of the thermo- mechanical and chemical processes involved in degradation processes and transport mechanisms can lead to a fine microstructure with low permeability and a high resistance to the penetration of aggressive elements of the environment. Such a fine microstructure can be obtained by using supplemental cementitious materials, such as fly ash, silica fume, blast furnace slag, or natural pozzolans.

Concrete durability is related to chemical resistance, dimensional stability and physical endurance, in addition to water tightness. Lower shrinkage of HPC is advantageous in reducing cracking and the subsequent ingress of chloride ions and detrimental chemical species. With the water tightness associated with less internal cracks of HPC, physical attacks such as wetting and drying, corrosion, carbonation, abrasion, and permeability will be minimized. The refined microstructure created by pozzolanic reactions reduces the potential for salt and water ingress, improves the sulfate resistance, increases long-term strength gain, and mitigates the expansions associated with alkali silica reaction and delayed ettringite formation..

The cost of the materials used in constructing high-performance concrete structures is not prohibitive. Table 7 provides the approximate cost of various cementitious materials used in high performance concrete. While some cost more than portland cement, others are considerably less expensive. The capital cost of ready-mixed concrete material in a concrete bridge is likely to fall by 5 to 10 percent if the concrete producer optimizes required strength, permeability, and shrinkage. This is more than a $1000 savings on a 4-lane, 30-meter- (100-foot) long bridge deck. If the reinforcing steel were replaced with stainless steel clad reinforcing, the capital cost of the reinforcing steel would increase by approximately 90 percent (see Table 8), but the labor and handling of reinforcing steel compared to epoxy coated rebar would drop. If a 30-meter- (100 feet) long, 4-lane bridge deck has a reinforcing ratio of 0.01, this would increase the cost of the bridge by approximately $8000 and more than double the bridge's life expectancy.

**Table 7. Approximate cost of cementitious materials.**

| Cementitious Materials* | Cost $/MT | Cost $/ton |
|---|---|---|
| Portland Cement | 66 | $60 |
| Ground Granulated Blast Furnace Slag | 60 | $55 |
| Fly Ash | 33 | $30 |
| Silica Fume | 440 | $400 |
| Metakaolin | 330 | $300 |

- Relative costs of cementitious materials are highly dependent on transportation costs and vary according to availability and market demand.

**Table 8. Approximate cost of reinforcing materials.**

| Reinforcement * | Cost $/kg | Cost $/lbs. |
|---|---|---|
| Reinforcing Steel (Grade 60 bar) | 0.61 | 0.28 |
| Epoxy Coated Reinforcing Steel | 0.70 | 0.32 |
| Galvanized Reinforcing Steel | 1.10 | 0.50 |
| Stainless Steel Clad Reinforcing Steel | 1.32 | 0.60 |
| Glass Fiber Reinforced Polymer Bar | 2.20 | 1.00 |
| Stainless Steel Reinforcing Steel | 3.52 | 1.60 |

* market prices change with production rates; prices based on 1999 survey in *Concrete Construction.*

## 5.2. REDUCTION OF LONG-TERM MAINTENANCE COST

Low-permeability concrete that cracks will not provide good long-term performance, and as a result will require long-term maintenance costs. HPC will reduce cracks due to shrinkage, alkali sulfate reaction, heat of hydration, shear, and flexural cracks to such a large degree that the need for maintenance will be sharply reduced.

By reducing the long-term maintenance needed for a structure through an extended life cycle, the overall cost of the structure throughout its life cycle can be greatly reduced.

## 5.3. EASE OF CONSTRUCTION, QUALITY CONTROL, AND EXPEDITION OF PROJECT COMPLETION

A fundamental question with respect to the realization of the intended quality of concrete is, Should it be achieved via a *sophisticated quality management system* or via a *robust mix tire*? To answer this question, several factors have to be kept in mind, such as the skill of the site workers, the place of the easiest and most reliable control, and the climatic conditions. The answer from HPC is very clear: self-compacting concrete is the most reliable.

Conventional concrete restricts construction in many ways, such as restricting on the height of a lift, requiring of construction scaffolding for vibration, and requiring separate placing of the bottom and walls of box section members. Using the self-compacting concrete makes consolidation unnecessary.

Such a construction method, in which concrete is just pumped into built-in forms that are assembled with steel reinforcement fabricated and constructed in a plant, is promising. The development of this construction systems is anticipated and justified by a variety of aspects, such as a reduction in the number of workers on site, a reduction in the number of skilled workers required, constant number of workers required throughout the construction period, shortened construction period, non-interference by the elements, and improved safety. The possibility of such new structural systems is becoming stronger. (For instance, a steel-concrete panel structure that has been impractical due to the placeability problem has been realized.)

## 5.4. REDUCTION OF TOTAL NUMBER OF MEMBERS AND TOTAL VOLUME OF MATERIAL

HPC with higher strength values is receiving greater attention for use in bridge structures in North America. For fixed girder dimensions, the increased concrete strength allows a reduction in the number of girders used. This will result in a lower unit cost for a given length structure. The utilization of longer span lengths for a multi-span structure results in the need for fewer piers and foundations. The reduced number of girders will also result in a lighter superstructure and hence reduce the dead load carried by the substructure.

The tensile strength of HPC increases with an increase of compressive strength. This is beneficial in the design of prestressed concrete members such as bridge girders where the tensile strength may control the design. The reduced creep of high-strength HPC is also beneficial in reducing prestress losses in bridge girders. Consequently, utilization of HPC in prestressed concrete girders results in economic benefits.

## 5.5. INCORPORATION OF INNOVATIVE MATERIALS

Pozzolans are mineral admixtures of fine siliceous or aluminous particles that are capable of reacting with lime at normal temperatures and forming cementitious products similar to those produced from portland cement hydration.

***Silica fume*** is a waste byproduct of the production of silicon and silicon alloys, which are extremely fine and glassy particles with 85 to 98 percent of silica dioxide ($SiO_2$). Silica fume offers a great potential as a replacement for cement, and seems to be applicable when high-strength concrete is needed. It is possible to make high-strength concrete without silica fume at compressive strengths of up to about 14,000 psi (98 MPa). Beyond that strength level, however, silica fume becomes essential, and even at lower strengths of 9,000-14,000 psi (63-98 MPa), it is easier to make high-strength concrete with silica fume than without it. Thus, when it is available at a reasonable price, it should generally be a component of high-strength HPC mixtures.

***Fly ash*** is a byproduct from combustion of coal or lignite in thermal power plants. It has been used extensively to produce low-heat concrete as well as high-strength concrete. When fly ash is used, it serves as a source of reactive silica for the pozzolanic reaction to reduce the heat of hydration and to contribute to some strength of the hardened concrete. Benefits of fly ash and slag cement application include strength gain with age, permeability, and increased durability.

***Ground granulated blast furnace slag*** is a byproduct of steel mills and can be classified into two general varieties: air- and water-cooled slag.

***Chemical admixtures*** are used to modify properties of mortar, fresh concrete, and hardened concrete to accommodate specific working conditions and serviceability. There is a wide variety of chemical admixtures suited for many different purposes. These include improving workability, pumpability, strength development, or reduction of slump loss, segregation, expansion rate, and others.

*Alternative reinforcement*

Polymer fibers have gained wide acceptance as a means of reducing the size of cracks from drying or plastic shrinkage. In addition, plastic fibers reduce the occurrence of subsidence. Pennsylvania increased the cover requirements in the late 1970s to distance the reinforcing steel from the deicing salts and to eliminate subsidence cracking. However, the lack of reinforcing near the surface creates more shrinkage cracks and wider cracks. Polymer fiber creating a potential to reduce the cover by reducing the risk of subsidence.

## 5.6. POTENTIAL MIXTURE DESIGNS OF HIGH-PERFORMANCE CONCRETE

As the transportation infrastructure in Pennsylvania grows, the potential exists to create a long-lasting highway system by optimizing concrete pavements and bridge structures. This task will use the information from this research to develop AAA concrete mixture designs that minimize the life-cycle costs for the new highways in Pennsylvania.

Concrete properties and performance are related to the mixture proportions, mixing procedure, transportation, placing, and curing method. Changing the source of portland cement can increase the compressive strength of a concrete mixture by more than 20 percent. Changes in the coarse aggregate source and content can change the compressive strength by up to 10 percent. The use of supplemental cementitious materials can decrease the permeability of the concrete, provide long-term structural strength, decrease the cost of concrete, and reduce the heat of hydration in the concrete. For this research project, several mixture designs were run to demonstrate the effects of the cementitious variables. Mixtures 1 through 6 were designed to meet grade 2 freeze-thaw durability, grade 1 workability, and grade 2 alkali-silica reaction durability. This is shown in Table 9. The two AAA mixtures are ready-mixed concrete from approved PennDOT sources with the same cementitious materials and aggregates.

Mixtures 1, 2, 3, and 5 all qualify as AAA concrete under current PennDOT specifications, except that mixtures 3 and 5 have lower cementitious materials contents. Mixtures 3 and 5 meet the required 28-day structural compressive strength for AAA concrete and both are designed to be low-permeable mixtures. The permeability of mixture 3 is well below 1000 coulombs and the ratio of 28-day $F_c$ to 7-day $F_c$ is greater than 1.50. This mixture would qualify as grade 1 compressive strength, grade 3 chloride penetration, grade 3-strength ratio and grade 2 modulus. Table 10 shows the estimated HPC performance grade for 10 categories.

## Table 9. Summary of potential HPC concrete mixtures (SI units).

| Factor | Units | Laboratory Trials | | | | | | Current | |
|---|---|---|---|---|---|---|---|---|---|
| | | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 | Mix 6 | AAA | AAA |
| Cement Factor | $kg/m^3$ | 455 | 455 | 340 | 340 | 340 | 350 | 480 | 400 |
| Pozzolan percent | mass percent | 5.9 | 6.3 | 5.9 | 22.1 | 6.4 | 35 | 20 | 35 |
| Coarse Aggregate Factor | percent DRUW | 0.7 | 0.7 | 0.66 | 0.66 | 0.66 | 0.69 | 0.66 | 0.67 |
| Water Factor | gal/sack | 4 | 4 | 4.9 | 4.9 | 4.9 | 5.0 | 4.2 | 4.9 |
| Air Factor | percent | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 |
| w/cm | | 0.35 | 0.35 | 0.43 | 0.43 | 0.43 | 0.44 | 0.38 | 0.43 |
| Cement | $kg/m^3$ | 428 | 426 | 320 | 265 | 318 | 227 | 384 | 260 |
| Silica Fume | $kg/m^3$ | 27 | | 20 | | | | | |
| Metakaolin | $kg/m^3$ | | 29 | | | 22 | | | |
| Fly Ash | $kg/m^3$ | | | | 75 | | | 96 | |
| GGBFS | $kg/m^3$ | | | | | | 123 | | 140 |
| Coarse Aggregate | $kg/m^3$ | 1142 | 1142 | 1077 | 1077 | 1077 | 1112 | 1077 | 1083 |
| Fine Aggregate | $kg/m^3$ | 569 | 571 | 759 | 752 | 760 | 691 | 541 | 649 |
| Water | $kg/m^3$ | 160 | 160 | 147 | .147 | 147 | 159 | 184 | 171 |
| AEA-MBVR | ml/kg | 50 | 50 | 50 | 50 | 50 | 19 | 50 | 50 |
| Water Reducer | ml/kg | 200 | 200 | 50 | 38 | 38 | 88 | 700 | 75 |
| 7-day $F_c$ | MPa | 36.9 | 42.9 | 24.3 | 22.5 | 22.3 | 29.9 | 44 | 38.4 |
| 7-day E | GPa | 25.2 | 29.0 | 22.7 | 13.8 | 19.3 | 30.6 | -- | -- |
| 14-day $F_c$ | MPa | 47.1 | 49.0 | 32.9 | 26.4 | 26.7 | -- | -- | -- |
| 28-day $F_c$ | MPa | 47.5 | 47.4 | 37.8 | 31.0 | 29.0 | 37.21 | 51.2 | 48.1 |
| 28-day E | GPa | 30.7 | 27.4 | 27.7 | 27.0 | 24.3 | 32.7 | 29.0 | 28.2 |
| 28-day $F_c$: 7-day $F_c$ | | 1.29 | 1.10 | 1.56 | 1.38 | 1.30 | 1.24 | 1.16 | 1.25 |
| 28-day Permeability | Coulombs | 678 | 527 | 693 | 6430 | 2919 | -- | -- | -- |
| Estimated Cost | $\$/m^3$ | 97 | 94 | 86 | 76 | 84 | 83 | 88 | 87 |
| Relative Cost of CM | $\$/m^3$ | 43 | 41 | 32 | 23 | 31 | 27 | 33 | 30 |
| Estimated HPC Performance Grades | FT | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| | SR | 2 | 2 | 2 | 1 | 1 | 2 | 1 | 2 |
| | AS | 2 | 2 | 2 | 2 | 2 | 2 | 1 | 2 |
| | CP | 3 | 3 | 3 | 0 | 1 | 2 | 1 | 1 |
| | CS | 2 | 2 | 2 | 1 | 1 | 2 | 2 | 2 |
| | SD | 2 | 0 | 3 | 2 | 2 | 1 | 1 | 2 |
| | ME | 2 | 1 | 1 | 1 | 1 | 2 | 1 | 1 |
| | SH | 1 | 1 | 1 | 2 | 1 | 2 | 1 | 1 |
| | SU | 1 | 1 | 2 | 2 | 2 | 2 | 1 | 2 |
| | WK | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |

## Table 10. Summary of Potential HPC Concrete Mixtures (U.S. customary units).

| Factor | Units | Laboratory Trials | | | | | | Current | |
|---|---|---|---|---|---|---|---|---|---|
| | | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 | Mix 6 | AAA | AAA |
| Cement Factor | sks/cu. Yd. | 8 | 8 | 6 | 6 | 6 | 6.25 | 8.45 | 7 |
| Pozzolan percent | mass percent | 5.9 | 6.3 | 5.9 | 22.1 | 6.4 | 35 | 20 | 35 |
| Coarse Aggregate Factor | percent DRUW | 0.7 | 0.7 | 0.66 | 0.66 | 0.66 | 0.69 | 0.66 | 0.67 |
| Water Factor | gal/sack | 4 | 4 | 4.9 | 4.9 | 4.9 | 5.0 | 4.2 | 4.9 |
| Air Factor | percent | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 |
| w/cm | | 0.35 | 0.35 | 0.43 | 0.43 | 0.43 | 0.44 | 0.38 | 0.43 |
| Cement | lbs/cu. yd. | 708 | 705 | 531 | 439 | 528 | 382 | 635 | 428 |
| Silica Fume | lbs/cu. yd. | 44 | | 33 | | | | | |
| Metakaolin | lbs/cu. yd. | | 47 | | | 36 | | | |
| Fly Ash | lbs/cu. yd. | | | | 125 | | | 159 | |
| GGBFS | lbs/cu. yd. | | | | | | 206 | | 230 |
| Coarse Aggregate | lbs/cu. yd. | 1909 | 1909 | 1800 | 1800 | 1800 | 1860 | 1800 | 1811 |
| Fine Aggregate | lbs/cu. yd. | 952 | 955 | 1269 | 1258 | 1271 | 1156 | 904 | 1085 |
| Water | lbs/cu. yd. | 265 | 265 | 243 | 243 | 243 | 262 | 303 | 282 |
| AEA-MBVR | oz./cu. yd | 4 | 4 | 4 | 4 | 4 | 1.5 | 4 | 4 |
| Water Reducer | oz./cu. yd. | 16 | 16 | 4 | 3 | 3 | 7 | 56 | 6 |
| 7-day $F_c$ | psi | 5350 | 6220 | 3520 | 3260 | 3230 | 4340 | 6380 | 5570 |
| 7-day E | ksi | 3650 | 4210 | 3290 | 2000 | 2800 | 4440 | -- | -- |
| 14-day $F_c$ | psi | 6830 | 7110 | 4770 | 3830 | 3870 | -- | -- | -- |
| 28-day $F_c$ | psi | 6890 | 6870 | 5480 | 4500 | 4210 | 5400 | 7430 | 6980 |
| 28-day E | ksi | 4450 | 3970 | 4020 | 3920 | 3520 | 4740 | 4210 | 4090 |
| 28-day $F_c$: 7-day $F_c$ | | 1.29 | 1.10 | 1.56 | 1.38 | 1.30 | 1.24 | 1.16 | 1.25 |
| 28-day Permeability | Coulombs | 678 | 527 | 693 | 6430 | 2919 | -- | -- | -- |
| Estimated Cost | $/yd³ | 72.50 | 70.50 | 64.50 | 57.00 | 63.00 | 62.50 | 66.00 | 65.50 |
| Relative Cost of CM | $/yd³ | 32.48 | 30.55 | 24.36 | 17.24 | 22.98 | 20.06 | 24.61 | 22.46 |
| Estimated HPC Performance Grades | FT | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| | SR | 2 | 2 | 2 | 1 | 1 | 2 | 1 | 2 |
| | AS | 2 | 2 | 2 | 2 | 2 | 2 | 1 | 2 |
| | CP | 3 | 3 | 3 | 0 | 1 | 2 | 1 | 1 |
| | CS | 2 | 2 | 2 | 1 | 1 | 2 | 2 | 2 |
| | SD | 2 | 0 | 3 | 2 | 2 | 1 | 1 | 2 |
| | ME | 2 | 1 | 1 | 1 | 1 | 2 | 1 | 1 |
| | SH | 1 | 1 | 1 | 2 | 1 | 2 | 1 | 1 |
| | SU | 1 | 1 | 2 | 2 | 2 | 2 | 1 | 2 |
| | WK | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |

# 6. RECOMMENDATIONS

The result of this study is a series of recommendations to improve the quality of concrete in the Pennsylvania transportation infrastructure. The recommendations have the immediate impact of simultaneously reducing the cost of concrete and extending the life of pavements, bridges, and other structures. In addition, the recommendations provide guidelines to improve the design process and the quality of the constructed infrastructure.

## 6.1. SPECIFIC RECOMMENDATIONS FOR THE COMMONWEALTH

### Recommendation I

The first item that should be addressed in the specifications is the variety of issues that lead to the high cementitious contents in portland cement concrete. The shrinkage cracking of pavements and bridge decks, the corrosion of reinforcing steel, and the cost of concrete are directly related to the volume of cementitious material and mortar used in concrete. The use of high volumes of portland cement (a) increases the concrete costs, (b) increases shrinkage cracking, (c) increases the permeability, and (d) increases the heat of hydration of the concrete mixture. The following changes should be made to the specifications:

(1) Concrete mixtures do not require the current levels of cementitious contents (Publication 408, Section 704, Table A) to meet 28-day structural design compressive strengths. The cement contents in Table A should be reduced to reduce shrinkage and the rapid strength gain exhibited by some mixture designs for AAA and AA concrete mixtures. The minimum cementitious content of AAA concrete should be 335 kg/m$^3$ (6 sks/yd$^3$) and the maximum should be reduced to 420 kg/m$^3$ (7.5 sks/yd$^3$). In addition, the "Cement Factor" should include the mass of all cementitious material (see recommendation 6).

(2) Table B of Section 704 in Publication should be changed to bring the 28-day structural design compressive strength in compliance with the minimum mixture design compressive strength. These values should be the same. This was one of the recommendations of the "Wilber Smith Study" (Babaei et al. 1996) and is a conclusion of this study as well. This provision

unnecessarily raises the cementitious content of concrete. The cost of this provision to the Commonwealth is approximately $5,000/lane mile in construction costs and a decrease in the life expectancy of the highway from cracked bridge decks and pavements.

(3) Fly ash, ground granulated blast furnace slag, silica fume, and natural pozzolans should be treated as cementitious materials. The limit of 15 percent fly ash as cementitious materials should be removed in all references and the specification should permit at least 25 percent fly ash. Concrete producers may choose to design a concrete mixture that contains 25 percent fly ash, 5 percent silica fume and 70 percent portland cement to deliver concrete that has low permeability, greater than 28 MPa compressive strength, low shrinkage, and resistance to both ASR and sulfate. This mixture would cost less than most AAA mixtures in many areas of the Commonwealth.

(4) Voids less than 1 mm in diameter should comprise more than 4 percent of the concrete volume and they should have a spacing factor equal to or less than 0.20 mm (0.008 in.).

(5) The minimum coarse aggregate content of A, AA, and AAA concrete should be increased to reduce the shrinkage of concrete. The current 0.37 $m^3/m^3$ should be raised to 0.41 $m^3/m^3$. Many mixture designs throughout the Commonwealth already exceed this limit. This must be implemented with the reduction in cementitious content provisions in 1,2, and 3.

Considering all of the above items, Table 4 of Section 704 in Publication 408 should be changed to the following:

**Table 11. Proposed Table A in Publication 408.**

| **Table A – Cement Concrete Criteria** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| CLASS OF CONCRETE | USE | CEMENT FACTOR* ($kg/m^3$) | | MAXIMUM WATER CEMENT RATIO (kg/kg) | MINIMUM COMPRESSIVE STRENGTH (MPa) DAYS | | | **MINIMUM** COARSE AGGREGATE SOLID VOLUME ($m^3/m^3$) |
| | | Min | Max. | | 3 | 7 | 28 | |
| AAA | Bridge Deck | **335** | **420** | 0.43 | -- | **21** | **28** | **0.41** |
| AA | Paving | **335** | **420** | 0.47 | -- | **18** | **24** | **0.41** |
| AA | Structures and Misc. | **335** | **420** | 0.47 | -- | **18** | **24** | **0.41** |
| A | | **310** | **420** | 0.50 | -- | **16** | **21** | **0.41** |
| C | | **200** | **340** | 0.66 | -- | 10 | **14** | ------ |
| H.E.S. | | **340** | 446 | 0.40 | 21 | -- | 24 | **0.41** |

*** Cement factor is equal to the mass of all cementitious materials in the mixture (e.g., portland cement, ground granulated blast furnace slag, fly ash, silica fume, natural pozzolan)**

The potential financial impact of these changes together is approximately \$6.50/$m^3$ (\$5/$yd^3$). In addition, each measure improves the quality of the concrete delivered to the Commonwealth and will extend the life of pavements, bridge decks, and structures. This is a cumulative savings of approximately \$15,800 per lane mile or \$635,000 on a ten-mile-long, four-lane divided highway. These estimates do not include the life extending cost savings to the Commonwealth. This savings is approximately \$2000/$m^2$ for bridge decks or approximately \$960,000 over the life of a 30-m-long (100 ft.), 4-lane concrete bridge.

## Recommendation 2

The specification should require additional measures of performance characteristics of the approved mixture designs. The department currently measures only air content, compressive strength and slump for approval of mixture designs. The current specification encourages high early-strength concrete and does not measure the parameters related to concrete performance. The following changes should be implemented to improve the long-term performance of concrete:

(1) The permeability of concrete should be measured for all AA and AAA concrete mixtures using AASHTO T277. This test is appropriate for the approval of mixture designs, but not for project quality control or payment. The criticisms related to this test can be addressed by testing concrete containing only air-entraining admixtures and water- reducing admixtures. If additional admixtures are used, these should not be contained in the concrete tested for mixture design approval. The test was developed to show the effect of portland cement, w/cm ratio, and supplemental cementitious materials. As such, testing concrete with only air-entraining admixtures and water-reducing admixtures eliminates the effects of salt-based admixtures that are associated with false negative tests, yet provides the appropriate performance characteristics of the cementitious system.

(2) The shrinkage potential of concrete mixtures should be measured for all AA and AAA concrete mixtures using ASTM C157 or the AASHTO Provisional Test procedure using cracking rings. Either of these tests is appropriate for the approval of mixture designs, but not for project quality control or payment.

(3) The compressive strength ratio (28-day/7-day) should be computed for all concrete mixtures. All type A and AA concrete mixture designs should meet a minimum value of 1.15 and all AAA concrete mixture designs should meet a minimum value of 1.33. This performance measure is appropriate for the approval of mixture designs, but not for project quality control or payment.

(4) The modulus of elasticity of concrete should be measured at 28 days for A, AA and AAA concrete mixture designs. This measure would be used for informational purposes, and it would allow structural engineers to compute deflections and stiffness with greater accuracy. It should not be used for project quality control or payment.

(5) The scaling resistance and abrasion resistance of type AA and AAA concrete should be measured for mixture design approval. The documented value of these tests would allow engineers or districts to specify these performance characteristics for particular applications. These tests are appropriate for the approval of mixture designs, but not for project quality control or payment.

(6) Mixture designs should be approved every 2 years. Districts are using mixture designs that were approved in 1991. The aggregate, cement, pozzolan, and admixture industries change on a regular basis. Many of these mixtures have changed admixtures and cement sources without going through reapproval.

(7) The scaling resistance, permeability, and compressive strength development are closely related to the curing of concrete. As such, concrete pavements and bridge decks should be cured for longer periods of time to provide better long-term performance of the concrete surface. Bridge decks should be cured for 10 days and all other pavements and structural concrete should be cured a minimum of 7 days.

The testing and documentation of performance characteristics is the only way to move forward and use a more performance-based specification. The combination of all these tests would cost approximately \$1000 per mixture design. This would add \$0.01 to \$0.20 / $m^3$, depending on the location in the Commonwealth and the volume of concrete purchased over 2 years. As part of this recommendation, Table B should be added to Publication 408, Section 704 (see Table 12).

**Table 12. Proposed Table B for Publication 408.**

| Table B – Cement Concrete Criteria | | | | |
|---|---|---|---|---|
| CLASS OF CONCRETE | USE | MINIMUM 28-DAY $f'_c$ / 7-DAY $f'_c$ RATIO | Minimum Moist Curing Time (Days) | MAXIMUM PERMEABILITY (Coulombs)* |
| AAA | Bridge Deck | 1.33 | 10-days | 1500 |
| AA | Paving | 1.15 | 7-days | 2500 |
| AA | Structures and Misc. | 1.15 | 7-days | 2500 |
| A | | 1.15 | 7-days | 4000 |
| C | | 1.15 | --- | ---- |
| H.E.S. | | ---- | --- | 4000 |

## Recommendation 3

The use of supplemental cementitious materials and admixtures within the specifications should be changed to reflect the state of the art in concrete technology and mixture designs. To implement this item, the department should make the following changes to the specification:

1) The recommended quantities of supplemental cementitious materials to mitigate the effects of ASR should be changed to the following:
    a. Use of 20-30 percent low calcium fly ash, or
    b. Use of 35-50 percent ground granulated blast furnace slag, or
    c. Use of 5-10 percent silica fume or metakaolin, or
    d. Reduction of the total of soluble alkalis from the portland cement below 2.5 $kg/m^3$ for elements not directly exposed to water, or below 1.8 $kg/m^3$ for structures directly exposed to water.

While the current specification takes appropriate steps to mitigate ASR, the current standards may increase the cementitious content, thereby increasing the probability of shrinkage cracking, heat of hydration-related problems and surface deterioration. This change has the effect of reducing the cost of concrete by $\$2.87/m^3$ (same as in recommendation 1).

2) There are chemical admixtures that could substantially improve the performance of concrete in highway structures. Shrinkage reduction admixtures, such as Tetraguard and Eclipse, corrosion inhibiting admixtures such as DCI and Ferroguard, and calcium silicate-based waterproofing admixtures, such as Ipanex and Xypex, should be investigated for life-cycle cost effectiveness and field performance.
3) The DM-4 limit on 55MPa (8,000 psi) concrete compressive strength should be raised to 80 MPa (11,500 psi) and the maximum span for prestressed concrete girders should be raised from 45 m (150 feet) to 60 m (200 feet). This would allow competition in the market for long-span bridge structures, provide another design alternative, and potentially reduce the cost of some long-span bridges. This would not flood the department with long-span, high-strength concrete girders, but would allow designers and contractors to consider them in the construction of bridges in the future. The design, production, and transportation of long-span concrete girders may be competitive in selective markets or particular jobs, but will not replace the need for steel in most long-span and curved applications.

## Recommendation 4

The major concrete components in structural design and pavements should be designed according to the recommendations above, including the revised 704 Table A and the proposed 704 Table B. Optional requirements could be added by the engineer, depending on the elements exposure to the elements, e.g., sulfate concentration, reactive aggregates, requirements would be Appendix A or the grade in the optional column):

**Table 13. Amendment to Publication 408.**

<table>
<tr><th>Concrete Item</th><th>Concrete Grade</th><th>Optional Requirements* (depending on exposure)</th></tr>
<tr><td>Bridge Deck</td><td>AAA<br>(CP Grade 3)</td><td rowspan="3">SR Grade 2<br>SH Grade 2<br>SU Grade 2<br>AS Grade 3<br>FT Grade 2<br>AB Grade 2</td></tr>
<tr><td>Pavement/Curbing</td><td>AA<br>(CP Grade 2)</td></tr>
<tr><td>Approach Slabs</td><td>AA<br>(CP Grade 2)</td></tr>
<tr><td>Beam Seats</td><td>AA</td><td rowspan="12">SU Grade 2<br>AS Grade 2<br>FT Grade 2<br>CP Grade 2</td></tr>
<tr><td>Pedestals</td><td>A</td></tr>
<tr><td>Piers</td><td>A</td></tr>
<tr><td>Abutments</td><td>A</td></tr>
<tr><td>Wing walls</td><td>A</td></tr>
<tr><td>Parapets</td><td>AA</td></tr>
<tr><td>Diaphragms</td><td>AA</td></tr>
<tr><td>Abutment Backwalls</td><td>AA</td></tr>
<tr><td>Retaining Walls</td><td>A</td></tr>
<tr><td>Footings</td><td>A</td></tr>
<tr><td>Precast Box Culverts</td><td>A</td></tr>
<tr><td>Pier Protection Walls</td><td>AA</td></tr>
<tr><td>Prestressed beams</td><td>As per design drawings<br>CS Grade 2 or 3</td><td>AS Grade 2<br>ME Grade 2 or 3<br>SD Grade 2<br>CC Grade 2<br>CP Grade 2</td></tr>
</table>

*Table 4.1 Legend:

SR: Scaling Resistance
SH: Shrinkage
SU: Sulfate Resistance
FT: Freeze Thaw Resistance
AB: Abrasion Resistance
AS: Alkali Silica Resistance
ME: Modulus of Elasticity
SD: Strength Ratio
CC: Creep Coefficient
CP: Chloride Penetration

## Recommendation 5

There is a need for a continuing educational component related to the durability of pavements and structures. This has been a long-term initiative advocated by the Federal Highway Administration and civil engineering professional groups. Engineers involved in specifying materials, project management, structural design, and construction inspection often do not have sufficient background in construction materials or value engineering related to life-cycle design. The following items should be implemented to improve the quality of concrete structures and pavements:

1) Engineers should obtain 40 hours of continuing education related to the selection of construction materials and new technologies related to construction materials. This training should take place over a 3-year period.
2) Engineers should learn to use a decision tree to provide a list of exposure conditions and a list of performance requirements for the specified concrete (see Appendix A).

# 6.2. HIGH PERFORMANCE CONCRETE ACTION PLAN FOR IMPLEMENTATION

A summary of the recommendations, industry comments, and economic impacts is presented in Appendix D. As the department moves forward with the implementation of high-performance concrete concepts and practices, there is a logical progression of activities that should be undertaken. There are three stages of implementation to gain the maximum benefits of current and future concrete technology.

## Stage 1

The first stage is to change the current specification to allow concrete producers, materials engineers and contractors to produce high-quality concrete. The current specification has accumulated many provisions that are contradictory to the department's goals for long lasting concrete. While the original intention of each of these individual provisions was to address particular technical concerns, over the years these provisions have created an overly prescriptive specification that produces average to marginal concrete quality. The first action the department can take is to implement all items in recommendation 1, item 1, or recommendation 3 immediately. This is summarized by the following items:

- Reduce the minimum volume of cementitious materials required in all classes of concrete.
- Allow fly ash to be used as an equal mass replacement for cementitious materials.
- Increase the minimum coarse aggregate requirement and eliminate the maximum coarse aggregate requirement.
- Eliminate the numeric difference between the 28-day structural design compressive strength and the minimum 28-day compressive strength of a concrete mixture.
- Change Table 4 of Section 704 in Publication 408 to resemble Table 11 in this report.
- Require 4-percent entrained air in hardened concrete.
- Change the use of pozzolans to improve ASR resistance.

## Stage 2

The second stage of the action plan should be to document the long-term and in-situ performance of HPC. This requires that the department implement studies of the technical, economic, and industrial impacts of using performance-based concrete mixture designs. This is the provisional use of all aspects of recommendation 2 and the remaining items of recommendation 3. A project such as the proposed I-99 initiative should be undertaken to document the long-life benefits of HPC to the department. This stage is summarized in the following items:

- Document the performance characteristics of concrete mixtures at the mixture design approval stage (permeability, shrinkage potential, ASR, scaling and sulfate resistance, etc.)
- Use binary and ternary blends of cementitious materials to improve the durability of concrete elements.

- Use chemical admixtures and advancements in concrete technology to extend the life of concrete elements and structures.
- Develop economic models that demonstrate the true long-term cost of HPC measures to the department.
- Use higher strength concrete in prestressed concrete girders to demonstrate the technical and economic potential of long span and alternative shape girders, e.g., U-beams.

## Stage 3

The nature of concrete materials and the durability of concrete structures have greatly evolved in the past 15 years. This is primarily due to the Strategic Highway Research Program and the intense state and industry research effort to obtain more efficient and long lasting concrete structures. The typical undergraduate education for civil and construction engineers includes a course in strength of materials, an introductory course in all engineering materials, and one or two courses in reinforced concrete structural design. At some universities, an advanced course in material science may also be part of the curriculum. This is not enough practical knowledge to specify, inspect, and evaluate the department's concrete structures. The third stage of implementation is to implement recommendation 5, which would offer continuing educational training to engineers and contractors. This education should focus on the material aspects of concrete and the variables that affect long-term durability and the maintenance of the infrastructure. This education should be a joint effort between universities and industry and should provide the engineers with continuing education unit credit or professional development credit within their performance evaluations. After this educational component is implemented, the department could implement a performance- based concrete specification by implementing recommendation 4.

# APPENDIX A
# ENGINEERING GUIDE TO SPECIFYING DURABILITY PERFORMANCE GRADES

# ENGINEERING GUIDE TO SPECIFYING DURABILITY PERFORMANCE GRADES

<table>
<tr><td rowspan="4">FT<br>Freeze<br>Thaw<br>Durability</td><td rowspan="4">Is the concrete exposed to freezing and thawing environments?</td><td rowspan="3">Yes</td><td rowspan="3">Is the member exposed to deicing salts?</td><td rowspan="2">Yes</td><td rowspan="2">Will the member be saturated during freezing?</td><td>Yes.<br>Use FT-Grade 3</td></tr>
<tr><td>No.<br>Use FT-Grade 2</td></tr>
<tr><td colspan="3">No. Use FT- Grade 1</td></tr>
<tr><td colspan="5">No. FT grade should not be specified.</td></tr>
<tr><td rowspan="4">SR<br>Scaling<br>Durability</td><td rowspan="4">Is the concrete exposed to deicing salts?</td><td rowspan="3">Yes</td><td rowspan="3">Is the exposure a direct application of salt?</td><td rowspan="2">Yes</td><td rowspan="2">Will the member be subjected to surface loadings?</td><td>Yes.<br>Use SR-Grade 3</td></tr>
<tr><td>No.<br>Use SR-Grade 2</td></tr>
<tr><td colspan="3">No. Use SR - Grade 1</td></tr>
<tr><td colspan="5">No. SR grade should not be specified</td></tr>
<tr><td rowspan="4">AB<br>Abrasion<br>Durability</td><td rowspan="4">Is the concrete exposed to surface abrasion?</td><td rowspan="3">Yes</td><td rowspan="3">Is the member subjected to other than tire wear?</td><td rowspan="2">Yes</td><td rowspan="2">Will the member be exposed to tire studs or chains?</td><td>Yes.<br>Use AB-Grade 3</td></tr>
<tr><td>No.<br>Use AB-Grade 2</td></tr>
<tr><td colspan="3">No. Use AB- Grade 1</td></tr>
<tr><td colspan="5">No. AB grade should not be specified.</td></tr>
<tr><td rowspan="4">AS<br>Alkali Silica<br>Reaction<br>Durability</td><td rowspan="4">Does the concrete contain reactive aggregates?</td><td rowspan="3">Yes</td><td rowspan="3">Is the concrete exposed to moisture?</td><td rowspan="2">Yes</td><td rowspan="2">Will the member be saturated during freezing?</td><td>Yes.<br>Use AS - Grade 3</td></tr>
<tr><td>No.<br>Use AS - Grade 2</td></tr>
<tr><td colspan="3">No. Use AS - Grade 1</td></tr>
<tr><td colspan="5">No. AS grade should not be specified.</td></tr>
</table>

# ENGINEERING GUIDE TO SPECIFYING DURABILITY PERFORMANCE GRADES

<table>
<tr><td rowspan="4">CP<br>Chloride Penetration Durability</td><td rowspan="4">Is the concrete exposed to chloride salts or soluble sulfates environments?</td><td rowspan="3">Yes</td><td rowspan="3">Is the member exposed in a potentially moist environment?</td><td rowspan="2">Yes</td><td rowspan="2">Will the member be saturated during freezing?</td><td>Yes.<br>Use CP-Grade 3</td></tr>
<tr><td>No.<br>Use CP-Grade 2</td></tr>
<tr><td colspan="3">No. Use CP- Grade 1</td></tr>
<tr><td colspan="5">No. CP grade should not be specified.</td></tr>
<tr><td rowspan="4">CS<br>Compressive Strength</td><td rowspan="4">Is the concrete structural or a pavement?</td><td rowspan="3">Yes</td><td rowspan="3">Is the member a slender column or prestressed beam?</td><td rowspan="2">Yes</td><td rowspan="2">Is the member optimized for high strength?</td><td>Yes. Specify a compressive strength within CS - Grade 3</td></tr>
<tr><td>No. Specify a compressive strength within CS - Grade 2</td></tr>
<tr><td colspan="3">No. Specify a compressive strength within CS - Grade 1</td></tr>
<tr><td colspan="5">No. Specify a minimum of 21 MPa (3000 psi)</td></tr>
<tr><td rowspan="4">SD<br>Strength Development</td><td rowspan="4">Will the concrete go into service after a minimum of 7 days after being cast?</td><td rowspan="3">Yes</td><td rowspan="3">Will the member benefit from long-term strength gain?</td><td rowspan="2">Yes</td><td rowspan="2">Is thermal cracking a potential in the member?</td><td>Yes.<br>Use SD-Grade 3</td></tr>
<tr><td>No.<br>Use SD-Grade 2</td></tr>
<tr><td colspan="3">No. Use SD- Grade 1</td></tr>
<tr><td colspan="5">No. SD grade should not be specified.</td></tr>
<tr><td rowspan="4">ME<br>Modulus of Elasticity</td><td rowspan="4">Is there a structural need for stiffness?</td><td rowspan="3">Yes</td><td rowspan="3">Is there a particular benefit to a higher than normal stiffness?</td><td rowspan="2">Yes</td><td rowspan="2">Is high stiffness critical to the structural design?</td><td>Yes.<br>Use ME - Grade 3</td></tr>
<tr><td>No.<br>Use ME - Grade 2</td></tr>
<tr><td colspan="3">No. Use ME - Grade 1</td></tr>
<tr><td colspan="5">No. ME grade should not be specified.</td></tr>
</table>

# ENGINEERING GUIDE TO SPECIFYING DURABILITY PERFORMANCE GRADES

| | | | | | | |
|---|---|---|---|---|---|---|
| **SH**<br>Shrinkage | Is the concrete exposed to moisture, chloride salts or soluble sulfates environments? | Yes | Is the member constructed without joints? | Yes | Is the member designed to be watertight or crack free? | Yes.<br>Use SH-Grade 3 |
| | | | | | | No.<br>Use SH-Grade 2 |
| | | | | No. Use SH- Grade 1 | | |
| | | No. SH grade should not be specified. | | | | |
| **SU**<br>Sulfate Resistance | Is the concrete exposed to more than 0.10 percent soluble sulfates? | Yes | Is the member exposed to more than 0.20 percent soluble sulfates? | Yes | Is the member exposed to wet-dry cycles? | Yes. Specify a compressive strength within SU - Grade 3 |
| | | | | | | No. Specify a compressive strength within SU - Grade 2 |
| | | | | No. Use SU - Grade 1 | | |
| | | No. SU grade should not be specified. | | | | |
| **TS**<br>Tensile Strength | Does the design depend on concrete to carry tension? | Yes | Does the structural performance rely on tensile strength? | Yes | Is there a design need for TS > 6 MPa? | Yes.<br>Use TS-Grade 3 |
| | | | | | | No.<br>Use TS-Grade 2 |
| | | | | No. Use TS - Grade 1 | | |
| | | No. TS grade should not be specified. | | | | |
| **WK**<br>Workability | Is there a congestion need because of formwork or reinforcing constraints? | Yes | Is there a need for flowing concrete or congestion in more than one area? | Yes | Is there a need for concrete to flow horizontally more than 2 meters? | Yes.<br>Use WK - Grade 3 |
| | | | | | | No.<br>Use WK - Grade 2 |
| | | | | No. Use WK - Grade 1 | | |
| | | No. Use WK - Grade 1 | | | | |

# APPENDIX B

# COST IMPLICATIONS OF SPECIFICATION CHANGES

## COST IMPLICATIONS OF SPECIFICATION CHANGES

Air entrained concrete: A 2.5 percent reduction in strength in AA or AAA concrete results from requiring 6.5 percent air rather than the recommended 6 percent air content. Concrete mixture designs must include approximately 15 kg/m$^3$ (25 lbs/cubic yard) more portland cement to compensate for this strength reduction.

$$\cos t = \frac{15kg}{m^3} \times \$0.0825 \Big/ kg - \frac{12kg}{m^3} \times \$0.0132 \Big/ kg = \frac{\$1.07}{m^3} = \frac{\$0.80}{yd^3}$$

- Fly ash equal mass replacement: A 25 percent replacement of portland cement with fly ash in AAA concrete currently results in a 10 percent increase in the mass of cementitious material and a 21 percent increase in the volume of paste in a concrete mixture. The proposed changes would allow an equal mass replacement.

**portland cement** **fly ash**

$$current \cos t = -\left\{\left[\frac{132kg}{m^3}\right] \times \$0.0825 \Big/ kg\right\} + \left\{\left[\frac{219kg}{m^3}\right] \times \$0.033 \Big/ kg\right\} = \left(\frac{\$3.66}{m^3}\right)$$

$$proposed \cos t = -\left\{\left[\frac{132kg}{m^3}\right] \times \$0.0825 \Big/ kg\right\} + \left\{\left[\frac{132kg}{m^3}\right] \times \$0.033 \Big/ kg\right\} = \left(\frac{\$6.534}{m^3}\right)$$

$$proposed\ \cos t savings = \left(\frac{\$3.66}{m^3}\right) - \left(\frac{\$6.534}{m^3}\right) = +\frac{\$2.87}{m^3} = +\frac{\$2.16}{yd^3}$$

- Using the ACI statistical method of specifying minimum compressive strength: While this will vary for from producer to producer, the typical standard deviation within concrete mixtures from a single good-quality plant is approximately 3.4 MPa (500 psi). Using equations 1 and 2 in section 4.6, the average compressive strength would drop approximately 5.5 MPa (800 psi) and still be well above the 28 MPa (4000 psi) required for structural design. This change would reduce the cementitious materials in AAA concrete mixture by 30 kg/m$^3$ (50 lbs/cubic yard) to compensate for this strength reduction.

$$\cos t = \frac{30kg}{m^3} \times \$0.0825/_{kg} - \frac{25kg}{m^3} \times \$0.0132/_{kg} = \frac{\$2.14}{m^3} = \frac{\$1.60}{yd^3}$$

(portland cement) (replacement aggregate)

- The increase of coarse aggregate content has the effect of reducing shrinkage in concrete pavements and bridge decks.

$$\cos t = \frac{86kg}{m^3} \times \$0.013/_{kg} - \frac{86kg}{m^3} \times \$0.008/_{kg} = \frac{\$0.43}{m^3} = \frac{\$0.32}{yd^3}$$

(reduction of fine aggregate) (addition of coarse aggregate)

# APPENDIX C
# ENVIRONMENTAL FACTORS FOR PENNSYLVANIA

# ENVIRONMENTAL FACTORS FOR PENNSYLVANIA

## HISTORIC AVERAGE RELATIVE HUMIDITY IN PENNSYLVANIA BY MONTH (MORNING AND AFTERNOON)

| DATA THROUGH 1993 | JAN | | FEB | | MAR | | APR | | MAY | | JUN | | JUL | | AUG | | SEP | | | | OCT | | NOV | | DEC | | ANN | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | M | A | M | A | M | A | M | A | M | A | M | A | M | A | M | A | M | A | M | A | M | A | M | A | M | A | M | A |
| ALLENTOWN, PA | 43 | 43 | 76 | 61 | 76 | 58 | 75 | 53 | 75 | 50 | 77 | 52 | 79 | 53 | 82 | 53 | 86 | 56 | 88 | 57 | 86 | 56 | 81 | 59 | 79 | 63 | 80 | 56 |
| ERIE, PA. | 28 | 28 | 77 | 72 | 78 | 71 | 77 | 66 | 75 | 61 | 76 | 62 | 79 | 63 | 80 | 64 | 82 | 64 | 82 | 65 | 77 | 63 | 75 | 68 | 76 | 72 | 78 | 66 |
| HARRISBURG, PA | 49 | 48 | 72 | 58 | 71 | 55 | 72 | 52 | 70 | 49 | 74 | 52 | 77 | 53 | 79 | 52 | 83 | 55 | 85 | 56 | 82 | 54 | 77 | 57 | 73 | 58 | 76 | 54 |
| HARRISBURG INTL APT | 51 | 50 | 72 | 58 | 71 | 55 | 72 | 52 | 71 | 50 | 74 | 52 | 77 | 53 | 79 | 53 | 83 | 55 | 85 | 56 | 82 | 55 | 77 | 57 | 73 | 58 | 76 | 55 |
| PHILADELPHIA, PA | 34 | 34 | 73 | 59 | 71 | 55 | 72 | 52 | 71 | 49 | 75 | 52 | 76 | 53 | 79 | 54 | 81 | 55 | 83 | 56 | 83 | 54 | 78 | 56 | 74 | 59 | 76 | 55 |
| PITTSBURGH, PA | 33 | 33 | 76 | 65 | 75 | 62 | 75 | 57 | 73 | 50 | 76 | 52 | 79 | 52 | 82 | 54 | 86 | 56 | 86 | 57 | 82 | 54 | 79 | 62 | 77 | 67 | 79 | 57 |
| AVOCA, PA | 38 | 38 | 75 | 65 | 75 | 63 | 74 | 58 | 72 | 52 | 76 | 52 | 82 | 55 | 83 | 56 | 87 | 58 | 88 | 61 | 84 | 59 | 79 | 64 | 77 | 67 | 79 | 59 |
| WILLIAMSPORT, PA | 48 | 48 | 76 | 62 | 76 | 58 | 77 | 54 | 75 | 49 | 80 | 51 | 84 | 54 | 87 | 55 | 90 | 58 | 92 | 59 | 88 | 57 | 82 | 61 | 78 | 63 | 82 | 57 |

## AVERAGE MONTHLY MAXIMUM TEMPERATURES (1961-1990)

| | #YR | JAN | FEB | MAR | APR | MAY | JUN | JUL | AUG | SEP | OCT | NOV | DEC | ANNUAL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ALLENTOWN, PA | 30 | 34.3 | 37.7 | 48.8 | 60.4 | 71.3 | 80.0 | 84.5 | 82.3 | 75.1 | 63.8 | 51.8 | 39.2 | 60.8 |
| ERIE, PA. | 30 | 32.5 | 33.7 | 43.6 | 54.5 | 65.9 | 75.3 | 79.9 | 78.5 | 72.1 | 61.0 | 49.4 | 37.8 | 57.0 |
| HARRISBURG, PA | 30 | 35.9 | 39.2 | 50.3 | 62.0 | 72.5 | 81.2 | 85.8 | 83.8 | 76.3 | 64.7 | 52.6 | 40.6 | 62.1 |
| HRSBRG INTL APT | 30 | 35.9 | 39.2 | 50.3 | 62.0 | 72.5 | 81.2 | 85.8 | 83.8 | 76.3 | 64.7 | 52.6 | 40.6 | 62.1 |
| PHILADELPHIA, PA | 30 | 37.9 | 41.0 | 51.6 | 62.6 | 73.1 | 81.7 | 86.1 | 84.6 | 77.6 | 66.3 | 55.1 | 43.4 | 63.4 |
| PITTSBURGH, PA | 30 | 33.7 | 36.9 | 49.0 | 60.3 | 70.6 | 78.9 | 82.6 | 80.8 | 74.3 | 62.5 | 50.4 | 38.6 | 59.9 |
| AVOCA, PA | 30 | 31.8 | 34.5 | 45.5 | 57.8 | 69.3 | 77.5 | 81.8 | 79.7 | 72.4 | 61.0 | 48.8 | 36.6 | 58.1 |
| WILLIAMSPORT, PA | 30 | 33.3 | 36.6 | 47.7 | 60.1 | 71.1 | 79.0 | 83.1 | 81.2 | 73.8 | 62.4 | 49.9 | 37.9 | 59.7 |

## AVERAGE MONTHLY MINIMUM TEMPERATURES (1961-1990)

| | #YR | JAN | FEB | MAR | APR | MAY | JUN | JUL | AUG | SEP | OCT | NOV | DEC | ANNUAL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ALLENTOWN, PA | 30 | 18.8 | 20.9 | 29.9 | 38.8 | 49.3 | 58.8 | 63.6 | 62.0 | 54.2 | 42.5 | 34.2 | 24.4 | 41.5 |
| ERIE, PA. | 30 | 18.2 | 17.9 | 28.1 | 37.7 | 47.8 | 57.6 | 62.6 | 61.8 | 55.9 | 45.6 | 37.0 | 25.2 | 41.3 |
| HARRISBURG, PA | 30 | 21.2 | 23.3 | 32.0 | 41.2 | 51.1 | 60.6 | 65.6 | 64.3 | 56.5 | 44.6 | 36.1 | 26.6 | 43.6 |
| HRSBRG INTL APT | 30 | 21.2 | 23.3 | 32.0 | 41.2 | 51.1 | 60.6 | 65.6 | 64.3 | 56.5 | 44.6 | 36.1 | 26.6 | 43.6 |
| PHILADELPHIA, PA | 30 | 22.8 | 24.8 | 33.2 | 42.1 | 52.7 | 61.8 | 67.2 | 66.3 | 58.7 | 46.4 | 37.6 | 28.1 | 45.1 |
| PITTSBURGH, PA | 30 | 18.5 | 20.3 | 29.8 | 38.8 | 48.4 | 56.9 | 61.6 | 60.2 | 53.5 | 42.3 | 34.1 | 24.4 | 40.7 |
| AVOCA, PA | 30 | 17.5 | 19.0 | 28.3 | 38.1 | 48.3 | 56.8 | 61.6 | 60.0 | 52.8 | 42.1 | 33.9 | 23.4 | 40.2 |
| WILLIAMSPORT, PA | 30 | 17.1 | 19.2 | 28.5 | 38.1 | 47.9 | 56.6 | 61.5 | 60.5 | 53.1 | 41.6 | 33.6 | 23.9 | 40.1 |

## AVERAGE MONTHLY LIQUID PRECIPITATION, in inches (1961-1990)

| | #YR | JAN | FEB | MAR | APR | MAY | JUN | JUL | AUG | SEP | OCT | NOV | DEC | ANNUAL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ALLENTOWN, PA | 30 | 3.16 | 2.95 | 3.28 | 3.52 | 4.20 | 3.75 | 4.14 | 4.28 | 3.93 | 2.94 | 3.88 | 3.49 | 43.52 |
| ERIE, PA. | 30 | 2.22 | 2.28 | 3.00 | 3.24 | 3.44 | 4.09 | 3.43 | 4.06 | 4.39 | 3.77 | 4.02 | 3.59 | 41.53 |
| HARRISBURG, PA | 30 | 2.84 | 2.93 | 3.28 | 3.24 | 4.26 | 3.85 | 3.59 | 3.31 | 3.51 | 2.93 | 3.52 | 3.24 | 40.50 |
| HRSBRG INTL APT | 30 | 2.84 | 2.93 | 3.28 | 3.24 | 4.26 | 3.85 | 3.59 | 3.31 | 3.51 | 2.93 | 3.52 | 3.24 | 40.50 |
| PHILADELPHIA, PA | 30 | 3.21 | 2.79 | 3.46 | 3.62 | 3.75 | 3.74 | 4.28 | 3.80 | 3.42 | 2.62 | 3.34 | 3.38 | 41.41 |
| PITTSBURGH, PA | 30 | 2.54 | 2.39 | 3.41 | 3.15 | 3.59 | 3.71 | 3.75 | 3.21 | 2.97 | 2.36 | 2.85 | 2.92 | 36.85 |
| AVOCA, PA | 30 | 2.10 | 2.15 | 2.55 | 2.97 | 3.65 | 3.98 | 3.79 | 3.32 | 3.31 | 2.79 | 3.06 | 2.51 | 36.18 |
| WILLIAMSPORT, PA | 30 | 2.54 | 2.76 | 3.19 | 3.23 | 3.86 | 4.32 | 3.98 | 3.39 | 3.39 | 3.30 | 3.73 | 3.03 | 40.72 |

## AVERAGE MONTHLY SNOWFALL, in inches (1961-1990)

| | #YR | JAN | FEB | MAR | APR | MAY | JUN | JUL | AUG | SEP | OCT | NOV | DEC | ANNUAL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ALLENTOWN, PA | 50 | 8.4 | 8.9 | 5.8 | 0.7 | T | 0.0 | 0.0 | 0.0 | 0.0 | 0.1 | 1.3 | 6.1 | 31.3 |
| ERIE, PA. | 39 | 22.8 | 16.1 | 10.4 | 2.7 | 0.0 | T | 0.0 | T | T | 0.3 | 10.3 | 22.9 | 85.5 |
| HARRISBURG, PA | 52 | 9.7 | 9.2 | 6.1 | 0.5 | T | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 2.0 | 6.8 | 34.3 |
| HRSBRG INTL APT | 54 | 9.4 | 9.3 | 6.5 | 0.5 | T | T | 0.0 | T | 0.0 | 0.0 | 1.9 | 6.6 | 34.2 |
| PHILADELPHIA, PA | 51 | 6.4 | 6.4 | 3.7 | 0.3 | T | T | 0.0 | 0.0 | 0.0 | 0.0 | 0.6 | 3.4 | 20.8 |
| PITTSBURGH, PA | 41 | 11.3 | 9.3 | 8.7 | 1.7 | 0.1 | T | T | 0.0 | T | 0.4 | 3.3 | 8.3 | 43.1 |
| AVOCA, PA | 38 | 11.2 | 10.8 | 9.4 | 3.1 | 0.1 | 0.0 | 0.0 | T | T | 0.2 | 3.3 | 8.8 | 46.9 |
| WILLIAMSPORT, PA | 49 | 10.4 | 10.1 | 8.2 | 1.2 | 0.0 | T | 0.0 | T | 0.0 | 0.1 | 3.0 | 8.2 | 41.2 |

## AVERAGE DAYS WITH MEASURABLE (>= 0.01") PRECIPITATION (1961-1990)

| | #YR | JAN | FEB | MAR | APR | MAY | JUN | JUL | AUG | SEP | OCT | NOV | DEC | ANNUAL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ALLENTOWN, PA | 50 | 11 | 10 | 11 | 12 | 12 | 11 | 10 | 10 | 9 | 8 | 10 | 11 | 125 |
| ERIE, PA. | 40 | 18 | 15 | 15 | 14 | 12 | 10 | 10 | 11 | 11 | 13 | 17 | 19 | 165 |
| HARRISBURG, PA | 13 | 11 | 10 | 11 | 13 | 13 | 11 | 10 | 9 | 9 | 9 | 10 | 10 | 125 |
| HRSBRG INTL APT | 15 | 10 | 10 | 11 | 12 | 13 | 11 | 10 | 9 | 9 | 9 | 10 | 10 | 125 |
| PHILADELPHIA, PA | 53 | 11 | 9 | 11 | 11 | 11 | 10 | 9 | 9 | 8 | 8 | 9 | 10 | 117 |
| PITTSBURGH, PA | 41 | 16 | 14 | 16 | 14 | 13 | 11 | 11 | 10 | 10 | 10 | 13 | 16 | 153 |
| AVOCA, PA | 38 | 12 | 11 | 13 | 12 | 13 | 12 | 11 | 11 | 10 | 10 | 12 | 13 | 140 |
| WILLIAMSPORT, PA | 49 | 12 | 11 | 13 | 13 | 13 | 12 | 11 | 11 | 10 | 10 | 12 | 12 | 141 |

# APPENDIX D

# SUMMARY OF RECOMMENDATIONS, COMMENTS, AND IMPACTS

# SUMMARY OF RECOMMENDATIONS, COMMENTS, AND IMPACTS

| CONCRETE EVALUATION | | | | |
|---|---|---|---|---|
| SECTION 4 | | | SECTION 5 | SECTION 6 |
| Existing Specification | Proposed Specification | Material Cost<br>*Bridge cost<br>*Life Cycle Savings | Industry Comments | Recommendations |
| **Freeze-Thaw Resistance** | | | | |
| • 6% air content in fresh concrete<br>• 3.5-8% entrained air in hardened concrete | • 6% air content in fresh concrete<br>• 4 - 8% entrained air in hardened concrete<br>• 80% durability factor for concrete bridge decks | $\$0.00/m^3$ ($\$0.00/yd^3$)<br>*Bridge $0.00<br>Life-Cycle Extension 5-10 years $160,000↓ | No comments | • Increase hardened concrete entrained air from 3.5 to 4%.<br>• Require a maximum spacing factor of 0.20mm (0.008") for hardened concrete.<br>• Require AAA concrete mixture designs to have an 80% durability factor. |
| **Scaling Resistance** | | | | |
| No current specification | • Require 7-day moist curing of concrete exposed to deicing salts<br>• Do not apply deicing salts in the first year. | $+\$8.14/m^3$ ($\$6.23/yd^3$)<br>*Bridge $1200↑<br>Life-Cycle Extension 5-10 years $60,000↓ | No comments | • Require 7-day moist curing of concrete exposed to deicing salts |

*Based on 30.5 meter (100 feet) structure length, 4-lane bridge, 7.6 meter (25 feet) abutments; 30-year average deck life, and $500,000 initial construction cost.

| SECTION 4 | | | SECTION 5 | SECTION 6 |
|---|---|---|---|---|
| Existing Specification | Proposed Specification | Material Cost<br>*Bridge cost<br>*Life Cycle Savings | Industry Comments | Recommendations |
| **Abrasion Resistance** | | | | |
| No current specification | • Use 28 MPa (4000 psi) compressive strength mixtures in acceleration and deceleration zones that are on steep grades or on interstate highways. Extend moist curing for 7 days in these zones. | +$8.14/m$^3$ ($6.23/yd$^3$)<br>*not applicable<br>Life-Cycle Extension Not applicable | No comments | • Train engineers to recognize abrasion potential areas and specify resistant concrete only when specifically needed. |
| **Alkali Silica Reaction** | | | | |
| Use fly ash or ggbfs to mitigate ASR; however, only reduce the cement content by 15% when using fly ash | Allow an equal mass replacement of fly ash to mitigate ASR | -$2.87/m$^3$ ($2.16/yd$^3$)<br>*Bridge $2336↓<br>Life-Cycle Extension 5-10 years $160,000↓ | No comments | • Allow an equal mass replacement of fly ash to mitigate ASR |
| **Chloride Penetration** | | | | |
| No standard or specification | Use AASHTO T277 in the mixture design approval process without potentially inhibiting admixtures | $0.00/m$^3$ ($0.00/yd$^3$)<br>*Bridge$00↑<br>Life-Cycle Extension 25-30 years $500,000↓ | May increase the cost of generating an initial mixture design. | • Require AASHTO T277 to be reported for all new and reapproved concrete mixture designs. |

*Based on 30.5 meter (100 feet) structure length, 4-lane bridge, 7.6 meter (25 feet) abutments; 30-year average deck life and $500,000 initial construction cost.

| CONCRETE EVALUATION | | | | |
|---|---|---|---|---|
| SECTION 4 | | SECTION 5 | | SECTION 6 |
| Existing Specification | Proposed Specification | Material Cost<br>*Bridge cost<br>*Life Cycle Savings | Industry Comments | Recommendations |
| **Compressive Strength** | | | | |
| Concrete compressive strengths are typically specified 3.4 MPa (500 psi) higher than the required minimum structural strength. There is no statistical deviation or monitoring. Minimum cement contents are specified. | Minimum compressive strength should be equal to the minimum structural compressive strength. This is true is nearly every other state. Variations should be determined according the statistical methods described in ACI 318 or AASHTO guidelines. The minimum cement contents should be lowered in line with industry standards and the supplementary cementitious materials (fly ash, ggbfs, silica fume, etc.) should be included in the minimum amount of cementitious materials. | -$2.14/m$^3$ ($1.60/yd$^3$)<br>*Bridge $1730↓<br>Life-Cycle Extension $0.00 | The statistical strength approach is the industry standard and is welcome in PSDOT specs.<br><br>Higher strengths would be welcome by the prestressed industry.<br><br>PADOT should change the penalty clause that discounts poor concrete, instead of requiring the contractor to replace concrete that is not durable.<br><br>The performance base on strength is welcome, as is the less prescriptive language on mixture design. | • Implement the recommendations.<br>• Change the penalty clause to require the removal of poor concrete. |
| **Strength Ratio** | | | | |
| No current standard or specification | Encourage 25-35 percent strength increase between 7 and 28 days | $0.00/m$^3$ ($0.00/yd$^3$)<br>*Bridge $00↑<br>Life-Cycle Extension 5-10 years $160,000↓ | The classes should be used carefully with prestressed concrete, since release strength typically controls. Unless true tensile strength can be used for release, 28-day strength is wasted | • Implement the recommendations for A, AA, and AAA concrete. |

*Based on 30.5 meter (100 feet) structure length, 4-lane bridge, 7.6 meter (25 feet) abutments; 30-year average deck life and $500,000 initial construction cost.

| CONCRETE EVALUATION | | | | |
|---|---|---|---|---|
| SECTION 4 | | | SECTION 5 | SECTION 6 |
| Existing Specification | Proposed Specification | Material Cost<br>*Bridge cost<br>*Life Cycle Savings | Industry Comments | Recommendations |
| **Modulus of Elasticity** | | | | |
| No current standard or specification | No proposed changes | $0.00/m^3$ ($0.00/yd^3$)<br>*Bridge $00↑<br>Life-Cycle Extension $0. | No comments | • No recommendations |
| **Shrinkage** | | | | |
| Specification specifies a minimum CAF of 0.61 and minimum cement content for AAA concrete of 385 kg/m^3 (6.75 sacks/yd^3). | Increase the coarse aggregate content of the concrete mixtures and reduce the cement content requirements. Require moist curing during hot or dry weather | -$2.57/m^3 ($1.92/yd^3)<br>*Bridge $369↓<br>Life-Cycle Extension 15-20 years $320,000↓ | Aggregate content must be weighed with w/cm ratio. The water content (w/cm ratio) should be increased if more coarse aggregate is required.<br><br>This is a reasonable provision. | • Implement recommendations. Do not raise w/cm ratios. |
| **Sulfate Resistance** | | | | |
| Use Type II cement in areas with known sulfate exposures | Allow greater amounts of class F fly ash for sulfate resistance without increasing the total cement content | -$2.87/m^3 ($2.16/yd^3)<br>*Bridge $1440↓<br>Life-Cycle Extension 5-10 years $160,000↓ | No comments | • Implement recommendations |

*Based on 30.5-meter (100 feet) structure length, 4-lane bridge, 7.6 meter (25 feet) abutments; 30-year average deck life and $500,000 initial construction cost.

| CONCRETE EVALUATION | | | | |
|---|---|---|---|---|
| SECTION 4 | | | SECTION 5 | SECTION 6 |
| Existing Specification | Proposed Specification | Material Cost<br>*Bridge cost<br>*Life Cycle Savings | Industry Comments | Recommendations |
| **Tensile Strength** | | | | |
| No current standard | No change. Tensile stresses could be monitored for prestress, but no specification change is required | $\$0.00/m^3$ ($\$0.00/yd^3$)<br>*Bridge \$00↑<br>Life-Cycle Extension \$0 | The $3\sqrt{f_c'}$ is too restrictive for allowable tensile stress. It prevents prestress beams from being optimized for compressive strength. | • Allow higher tensile stresses at release times. |
| **Workability** | | | | |
| Contractor specifies slump within limits | Maximum limit of 125mm (5 in.) should be specified unless there is proof of congestion. In congested areas, higher slumps should be permitted using superplasticizer. | $\$0.00/m^3$ ($\$0.00/yd^3$)<br>*Bridge \$00↑<br>Life-Cycle Extension \$0 | No comments | • Implement recommendations |
| **Creep Coefficient** | | | | |
| Uses AASHTO Design Guidelines | No design changes, however allow empirical data for creep | $\$0.00/m^3$ ($\$0.00/yd^3$)<br>*Bridge \$00↑<br>Life-Cycle Extension \$0 | No comments | • No recommendations |

*Based on 30.5meter (100 feet) structure length, 4 lane bridge, 7.6 meter (25 feet) abutments; 30-year average deck life and \$500,000 initial cost of construction.

# REFERENCES

# REFERENCES


American Concrete Institute. *Manual of Concrete Practice*. Farmington Hills, MI: ACI, 1999.

Babaei, K. and R. L. Purvis. "Prevention of Cracks in Concrete Bridge Decks – Summary Report." Wilber Smith Associates, PennDOT Project Report 89-01, March, 1996.

Bartos, Peter and Marianne Grauers. "Self-Compacting Concrete." *Concrete for the Construction Industry* 33 (4): 9-13 (1999).

Carlson, Roy W. "Drying Shrinkage of Concrete as Affected by Many Factors." *ASTM Proceedings*. 38 (II): 419-438 (1938).

Khayat, K. H. "Workability, Testing, and Performance of Self-Consolidating Concrete. " *ACI Materials Journal* 96 (3): 346-353 (1999).

Sohanghpurwala, A. A. and W. T. Scannell. "Verification of Effectiveness of Epoxy-Coated Rebars." ConCORR Inc., PennDOT Project Report 94-05, November, 1998.